Reliability of Standard Penetration Test to Determine Bearing Capacity of Fine Grained Soil: A Case Study in Addis Ababa Soils

Beza Gebremelak

A Thesis Submitted to

School of Earth Sciences

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

ADDIS ABABA UNIVERSITY
Addis Ababa, Ethiopia

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To My Father

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This is to certify that the thesis prepared by Beza Gebremelak, entitled: *Reliability of Standard Penetration Test to Determine Bearing Capacity of Fine Grained Soil: A Case Study in Addis Ababa Soils* and submitted in partial fulfillment of the requirements for the Degree of Master of Science (Engineering Geology) complies with the regulations of the University and meets the accepted standards with respect to originality and quality.

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Chair of School or Graduate Program Coordinator Date

Abstract

Reliability of Standard penetration Test to Determine Bearing Capacity of Fine Grained Soil: A Case Study in Addis Ababa Soils

Beza Gebremelak Gebrewold

Standard penetration test (SPT) is widely practiced in Ethiopia to define fine grained soils' strength parameter for bearing capacity analysis in foundation design. However, there is no published correlation between the in-situ measured SPT data and soil design parameters to define the reliability of the method. In this research, it is attempted to measure the reliability of bearing capacity estimation from SPT data of fine grained soil using linear regression analysis. SPT data were collected from newly measured SPT data, 22 boreholes in Addis Ababa, with a record of N-value, hammer and anvil type, penetration interval, hammer blow rate, drill rod type, rod length, and drill rig type of to define the bearing capacity of fine grained soil. Besides, corresponding samples were also obtained to determine undrained shear strength and other physical and engineering properties of soil in the laboratory. Initial efforts were made to establish a correction factor of SPT blow N-values to different N-values; N_{55} N_{60} and N_{70}. Besides, further correlation and comparison were made between SPT N-values and undrained compressive strength (C_u) considering other parameters such as natural water content (W_n), liquid limit (LL) and plasticity index (PI). Results have shown that SPT N-values are strongly correlated to the undrained compressive strength within the range of limited estimation error. It is also noted that there is mild correlation between two parameters based on the coefficient of regression. The correlation factors are also compared with the previously established coefficients for fine grained soil to show their relevance. As a result, empirical correlation between soil test results and bearing capacity design values are established. Further, correlation values were established between C_u and the in-situ measurement data to define the degree of uncertainty.

KEYWORDS: Standard penetration test, Bearing capacity, Undrained shear strength, N-value,

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TABLE OF CONTENTS

List of Figure

List of Plate

List of Table

CHAPTER 1 INTRODUCTION

1.1 General

Geotechnical investigation provide sufficient information regarding far site suitability, design criteria, and possible construction problems. Both laboratory and in-situ (field) testing are routinely used to obtain information about engineering properties of soils and rocks. During the field exploration phase, in the case of laboratory testing is not conducted, field test will be needed to determine the soil strength parameters. One of the most common field test that is used in geotechnical investigation is standard penetration test (SPT) (Bowles, 1997).

Standard penetration test is the most common in-situ test method that widely used in ground investigation. It can provide much of the information required during a site investigation as compared to other field techniques because of its simplicity, quick estimation and relatively inexpensiveness. It is conducted in the vertical direction at the bottom of borehole using the drilling rig and its accessories made up of split-spoon sampler, hammer, and drill rods. This test provide a measurement of the resistance of the soil, interms of hammer blow counts (N-value). The N-values are used to estimate the consistency, strengh and in some cases, the compressibilty of the soil (Clayton, 1995).

The purpose of SPT is determine the resistance of soil to the penetration of the standard-size sampler. Its done to characterize the shear strength of soil by counting the number of hammer blows that are required to penetrate a given depth. SPT mainly involves in determining soil properties as well as estimating the bearing capacity of soils. The technique is suitable for most soil types except gravel and is usually performed using a conventional geotechnical drill rig (Aggour and Radding, 2001). It can estimate the relative density and compressibility for granular soils, since it is originally developed for coarse-grained soils; but these days, it has been also widely applied for fine-grained soils in estimating the unconfined compressive strength (q_u), undrained shear strength (C_u), and coefficient of volume compressibility (m_v) in Sirvikaya et al., (2009, cited in Frazad and Behzad, 2011).

Since SPT is highly dependent upon the equipment and operator performing the test, it is often susceptible for errors. Numerous authors have researched on the accuracy and validity of SPT for many years. Most of them have learned the test need a standard or correction on the basic things; includes hammer efficiency, split spoon sampler design, length of drill rod, diameter of borehole and over burden pressure. (Bowles, 1997).

Although questions have been rased on the consistency of standard penetration test method applying in fine grained soils but still it is widely used in the world including Ethiopia. Efforts are always made in order to minimize or avoid uncertainties by dealing with possible source of errors and correlating values with soil strength parameters in order to get a better result.

1.2 Problem Statement

Now a days Ethiopia, particularly Addis Ababa is undergoing a transformation in the construction industry. In order to strengthen its infrastructure and supply affordable housing and power to its population a proper geotechnical investigation should be practiced for safe and long lasting engineering structures. However, in many constructions a standard construction procedure has not been practiced even if the government declared a construction law. One of the vital work in building construction is foundation investigation and it is not getting the necessary attention it needed. Determination of bearing capacity of a particular building is one of the major task in foundation investigation. Due to lack of understanding and for the sake of money saving most contractors are carelessly practiced foundation investigation and some of them are not experienced at all.

The SPT method has been widely practiced in Ethiopia, due to its cost effectiveness and quick estimation. In fact, this test is conventionally measured using different kinds of hammers, drill rigs, energy delivery systems with different degrees of efficiency, drill rod lengths, hammer blow rates, borehole fluids, and sampling tubes. Thus this can affect the test result. On the other hand, the SPT N-value may vary significantly with seasonal fluctuations in the water table of cohesive soil (Aggour and Radding, 2001). Therefore, the values may fall short of providing information on the characteristics of the fine grain soil, mainly its strength. Consequently, this getting rise an question around the ability of the test to consistant result.

This is also the problem in Addis Ababa. Several geotechnical firms used SPT method for estimating bearing capacity despite the question of reliability and compatibility for fine grained soil. In this study, it is planned to define the degree of reliability of SPT method in estimating the bearing capacity of fine grained soil. It is basically intended to correlate the

SPT results with its undrained shear strengh and identify the main source of errors during testing data analysis and interpretations as well as to evaluate any related uncertainties.

1.3 SPT Practice in Ethiopia

Internationally, standard penetration test has significant differences between the drilling techniques, SPT equipment and test procedures. The major influences on the result of SPT are method of drilling, hammer mechanism, borehole fluid, borehole diameter, rod stiffness, split-spoon geometry and method of testing (Clayton, 1995).

In Ethiopia currently several geotechnical firms outshine related with the rise of construction in the country. Majority of the contractor's interest to make geotechnical investigation for safe and economical foundation design. This investigation provided the ultimate bearing capacity and tolerable limit of settlement of the foundation material (soil / rock). Both analytical and emiperical (semi emiperical) methods are widely used for the determination of bearing capacity. Most geotechnical engineers in ethiopia have been prefer standard penetration test since it is simple and quick estimation for foundation design. Meyerhoff SPT equation is used for bearing capacity calculation with 25mm settlement. SPT N-value is standardazed by applying different correction factors (hammer, sampler, borehole, rod and depth correction). According to previous suggustion the SPT N-values to be standardized to N_{55}, N_{60} and N_{70} (Aggour and Radding, 2001). Most companies have been applied N_{55} and N_{70}. for the ajustment of SPT depending on the SPT equipment and drilling rigs.

As stated in Ethiopian building code standards (EBCS-7, 1995), during conducting SPT test some features shall be considered. This includes lifting method, shoe, mass of the falling weight, the drop height, the diameter of the casing and the rods, groundwater condition, the nature of the ground. The test is conducted with different drilling rig types, hammer types (donut, safety and automatic hammer), drilling system, and hammer lifting mechanism (cathead and automatic release system).

Ethiopia hasn't establish its own SPT standard and as a result companies are adopting different standards from other country practice. The standards widely applied in Ethiopia are American standard (ASTM D1586: 2008) and Britain standard (BS 1377: 1975). Applying other's country SPT standards which have different soil types, climate and hydrogeology leads to untrustworthy result. In order to reduce the significant variability of the SPT N-value due to the large variation in energy delivered and other factors, it has been recommended that the N-value be standardized to a specific value through the use of correction factors.

1.4 Objectives

1.4.1 General Objectives

The general objective of this research work is assessing the reliability of standard penetration test to estimating bearing capacity of fine grained soil.

1.4.2 Specific Objectives

- Describe the engineering properties of the fine grained soils
- Identify the possible source of errors during conducting the test.
- Standardization of SPT N-values by applying different correction factor.
- Correlate SPT N-value with undrained shear strength.
- Correlate SPT N-value with PI, PL, LL and W_n
- Proposed empirical equations.
- Compare predicated undrained shear strength with measured undrained shear strength of fine grain soil values.
- Compare with previous correlation.
- Recommend important measures that to be taken minimize the errors in SPT data collection.

1.5 Methodology

The objectives of the study were accomplished following an appropriate and systematically organized work. The research work was started on literature review followed by field work which includes in-situ testing, and sample collection. Prior to make an interpretation and result analysis a detail laboratory work had been done.

In order to effectively and systematically accomplish the research's objectives the following detail methodology were adopted.

In the desk study the following work were undertaken

- Literatures were reviewed that related to the research topic including journals, books, published and unpublished technical papers.

- Different types of maps have been reproduced and georeferenced in ArcGIS project for this research work. Some of the maps are geological map, engineering geological maps and city maps of the study area.
- Developed the geo-database in microsoft access and excel that got from field work.

The primary tasks were done in the field includes, recording N-values of SPT, geotechnical logging, taking disturbed and undisturbed samples for laboratory testing on active sites in Addis Ababa. Standard penetration tests were conducted at 15 cm interval and the split-barrel sampler was penetrated 45cm in to the soil at the bottom of the bore hole and recorded only the last count (30 cm) to obtain the N-value.

Along with sample collection, the major source of errors have been identified while conducting SPT. The test was executed using different rig types with two SPT hammer types; donut and automatic hammer. The samples were collected from different sites of Addis Ababa to conduct the following tests in the laboratory;

- Liquid limit
- Plastic limit
- Grain size analysis
- Natural moisture content
- Unit weight and
- Unconfined compressive strength test

Once the results of in-situ and laboratory testing are completed:

- The soil engineering properties are defined by comparing and relating different parameters.
- Characterization of the soil by integrating the results obtained from geotechnical field investigations and laboratory result analysis.
- Interpretation of the results of the tests and the classification in parallel to determine, the index properties of the soil by correlating in-situ N-values and unconfined compressive strength or undrained shear strength derive from q_u.
- Comparing with previous correlation.

1.6 Significance of the Study

This research work was conducted to define examine the capability and consistency of standard penetration test for fine grain soils and to identify the major source of errors in SPT. The findings of the research will provide information for foundation designers to ensure about SPT method that give representative result or not in determining bearing capacity.
Simultaneously it is tried to answer the question that arising around its reliability SPT method for estimating bearing capacity of fine grain soil. Thus this study will contribute in the efforts to develop a guide line for SPT data collection and estimation of bearing capacity of soil.

1.7 Scope and Limitations of the study

To study the reliability of SPT test in determination of bearing capacity of fine grain soil, many SPT measurements at different depth were carried out. Many samples were collected to determine experimentally the properties of soil. There are different parameters were taken into consideration such as SPT number (N-value), undrained shear strength parameters (C_u), liquid limit (LL), plastic limit (PL), plasticity index (PI) and moisture content (W_n). In this study, the relationships between SPT N-values and undrained shear strength have been discussed to make sure that the reliability of the SPT result.

Since there is a shortage of financial support it is difficult to conduct drilling operation solely for this project. Therefore, the only way to proceed the study is to cooperate with few companies to collect data on their active drilling sites which are not well enough. Consequently, it affected the study to deal with various source of errors which is related to rig machines, sampler and SPT equipment. The study has also been limited to assessing the consistency of standard penetration test of fine grain soils, limited samples, field test and laboratory test and the tests were conducted on limited number of drilling rig type and SPT equipment.

Consequently further research is required to assess the consistency in-terms of hammer efficiency including different SPT tools and drilling rig type and also the fine grain soils collaborate with other compressive strength test, investigating with a large scale.

1.8 Scheme of Presentation

This research is composed of six chapters and the scheme of presentation is as follows:
Chapter 1 comprises the introduction to the research in general which basically is the background, problem statement, objective and significance of the research, methodology, application of the research and limitations and scope of research.

Chapter 2 presents the literature review. The literature review comprises a brief description of previous works relevant to the present research it includes history and background SPT, Standard penetration test practice, adjustment of measured SPT N-values, and SPT limitation.

Chapter 3 is describing a brief description of the study area including geographical location, climate, geology, physiographic and drainage conditions, soils, hydrogeology and seismology of the research area.

Chapter 4 deals with the SPT measurement details and laboratory analysis. It consists, SPT equipment, test procedure and standardization of field N-values.

Chapter 5 presents the data analysis and interpretation of the test results, correlation between different parameters and comparison of the test results, and comparison with existing correlation.

Chapter 6 presents the overall conclusions and recommendations that can be made out of the present research work.

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CHAPTER 2 LITERATURE REVIEW

2.1 Introduction

In this chapter, a brief literature review on standard penetration test has been presented. The information which compiled here was selected to be directly related to the scope of this particular study. More detailed information obtained from the literature on standard penetration test can be found in detail in Construction Industry Research Information Association (CIRIA) Report No.143, *SPT Methods and Use* 1995, which mainly describe about the standard penetration test practice and influence of equipment and ground condition on penetration resistance. Bowles also describes SPT test procedures and correction factors in *Foundation Analysis and Design* presented in 1997. The other literature that reviewed is *Standard Penetration Test Correction* presented by Aggour and Radding (2001), which mainly describe about the correction factors that mainly applied in the adjustment of standard penetration test.

2.2 History and Definition of Standard Penetration Test

Standard penetration test is an in-situ dynamic penetration test that measures the resistance of the soil to penetration of the sampler. The SPT is a simple and rugged test suitable for most soil types and is usually performed using a conventional geotechnical drill rig (Sabatini and Bachus, 2002).

As stated in American Society for Testing Materials (ASTM D1586: 2008) manual, this method performed from the base of borehole using split- spoon sampler which consists a driving shoe, a split- barrel of circular cross-section and a coupling. SPT describes a procedure for using a split-barrel sampler to obtain representative samples of soil for identification purposes and other laboratory tests.

According to Aggour and Radding (2001), the earliest credits for the SPT are attributed to Mohr and also to Terzaghi. The standard penetration test, developed around 1927, is currently the most popular and economical means to obtain subsurface information (Bowles, 1997). The split soon sampler was first introduced by Harry Mohr (a subsidiary of the Raymond concrete pile Co.) and the other by Sprague and Henwood. The sample tubes has a 2-in diameter and were driven into the bottom of the borehole by repeated blows of a weight lifted by two men by hand, without the use of any powdered winch equipment (Clayton, 1995).

The system to drive the SPT sampler into the soil, known as the drive-weight assembly, basically consists of the hammer, hammer fall guide, anvil, and a hammer release system. There are different types of hammers applied for the standard penetration test; safety and donut hammer type, those also used different hammer lifting systems; the trip, automatic, or semiautomatic system, where the hammer is lifted and allowed to drop unimpeded. The other one is the cathead release system; it is a method of raising and dropping the hammer that uses a rope slung through a center crown sleeve or pulley on the drill rig mast and turns on a cathead to lift the hammer (Day, 2006).

2.3 Standard Penetration Test Practice

2.3.1 National standards and test procedures

The SPT has been used in almost every part of the word. The Standard penetration test was first “standardized” by the American Society for Testing Materials in the USA in 1958 with periodic revisions to date (Clayton, 1995). The method was further standardized with the most recent update in 1999. The test introduce in to Britain standard (BS 1377) in 1975 as the ‘Determination of the penetration resistance using the split spoon sampler’.

Although many countries have their own standard, internationally ASTM standard is widely used. According to ASTM (D1586), SPT is utilize using a split spoon sampler which has 51 mm external diameter, 35 mm inside diameter and 45 cm to 65 cm length (see Fig. 2.1). The split spoon sampler is connected to a string of drill rods and is lowered in to the bottom of the borehole which was drilled and cleaned in advance. As shown in Fig. 2.2 the drill rods marked in three successive 15 cm increments so that the advance of the sampler under the impact of the sliding hammer weighing 63.5 kg having a free fall length of 760 mm and count the number of blows applied in each 150 mm. The first 150 mm so considered to be a seating drive. The sum of the number of blows required for the second and third 150 mm increment of penetration is used as N-value. The SPT commonly terminates when the number of blows to drive any of the 150 mm segment exceeds 50 or when no more depth was attained with 10 consecutive blows. The test procedure is repeated at every 1.5 m or at every stratum change. At the end of each SPT operation, the sampler tube is removed and disassembled to collect representative disturbed sample for further laboratory tests (Bowles, 1997).

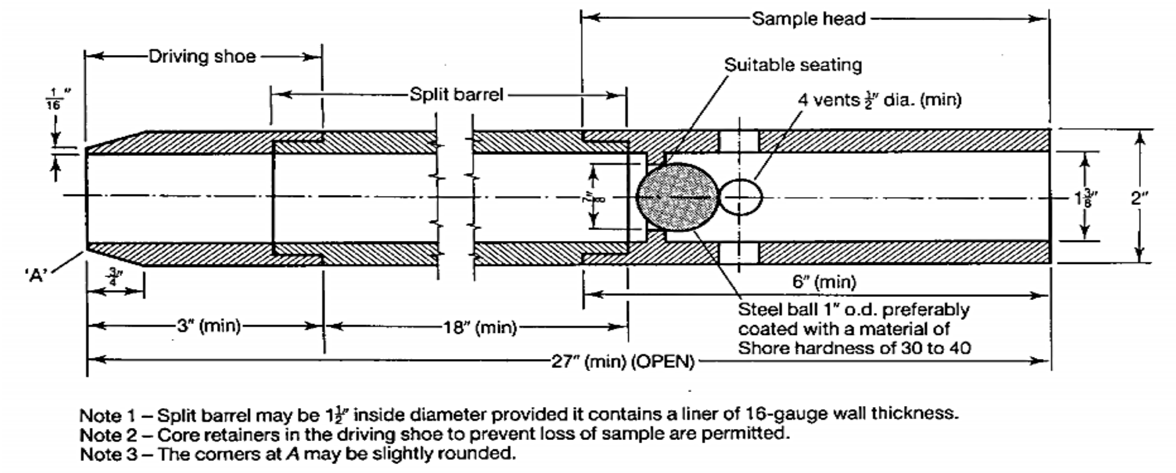

Fig. 2.1 ASTM, Split spoon sampler

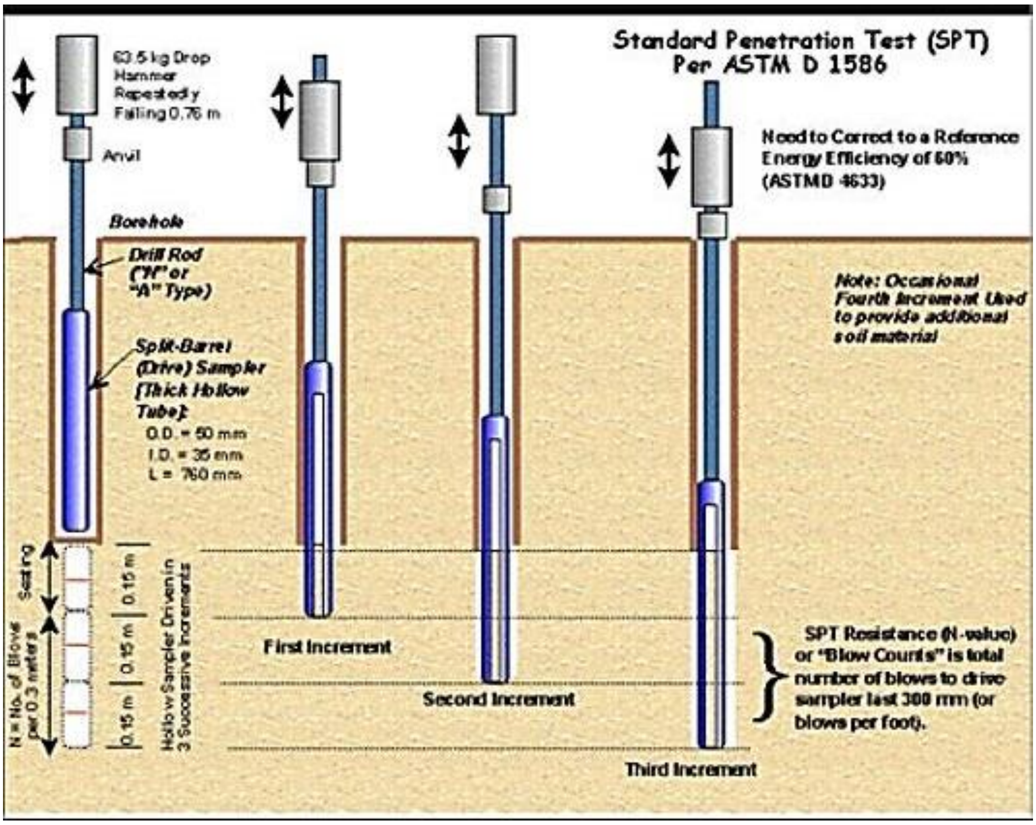

Fig. 2.2 SPT Procedure as per ASTM D1586

2.3.2 Corrections applied for SPT N-Values

The standard penetration test has been conducted conventionally using different kinds of hammers, drill rig types, drill rod lengths and types, hammer blow rates, different energy delivery systems with different degrees of efficiency, different borehole fluids, and different kinds of sampling tubes (Aggour and Radding, 2001). In order to reduce the significant variability associated with the SPT N-value, it was needed to adjust by applying different correction factors.

Various correction factors that commonly applied to field N-values, these includes the energy efficiency, split spoon configuration, borehole diameter, overburden pressure, rod length, blow rate and anvil correction factors (Aggour and Radding, 2001), recognize that most significant factor affecting the measured N-value as the amount of energy delivered to the drill rods. They indicated that the energy delivered to the rods during an SPT test can vary from about 30% to 80% of the theoretical maximum depending on the type of hammer system used and made a summary of energy efficiencies as predicted by a number of researchers, indicated that the energy transfer ratio for safety hammers with cathead and rope hoisting mechanism can vary considerably. The range of reported values is from 30% to 96%. For automatic trip hammers, the range is smaller, with a low of 60% and a high of 90%.

The SPT hammer drops a distance of 760 mm per blow to drive the split spoon sampler into the soil however only a portion of the applied energy transfers through rod in to the soil depending on hammer efficiency. The hammer efficiency depends on the type of hammer and local practice. The efficiency of the hammer can be evaluated as the ratio of kinetic energy (KE = ½ mv^2) to potential energy of the falling weight (PE = mgh), which are function of the falling mass (m), velocity (v), gravitational acceleration (g), and height of drop (h) (Bowels, 1997).

$$Energy\ efficency = {}^{E_r}/_{E_{rb}} \qquad \textbf{Eq. 2.1}$$

$$\text{where } \mathrm{E_r} = \text{energy ratio}, \mathrm{E_{rb}} = \text{standard energy ratio}$$

Table 2.1 Suggestions for the value of the standard energy ratio E_{rb} (Bowels, 1997)

E_{rb}	Reference
50 to 55 (use 55)	Schmertmann (1983)
60	Seed et al. (1985): Skempton (1986)
70 to 80 (use 70)	Riggs (1986)

When the length of the drill rod is less than 10m, energy is reflected back in the rod reducing the energy transmitted to the split spoon penetrating the soil. This warrants the use of a rod correction factor for rod length below 10 m. Early split spoon configurations had a constant inner diameter of 35 mm, while the current standard ASTM D1586 split spoon has a variable internal diameter to accommodate an internal liner. As the size of borehole increases, the effective stress acting on the soil at the base of the borehole decreases making it easier on the split spoon sampler to penetrate the subsoil. This effect is not significant for boreholes having a diameter less than 115 mm. Soils of the same density will give smaller N-values if p'_o is smaller (as near the ground surface). Oversize boreholes on the order of 150 to 200mm will also reduce N-value (Bowels, 1997).

The anvil can vary in shape, size and weight. The amount of energy transferred to the drill rods depends on the weight of the anvil and blow count frequency that applies for soils are also different.

$N_{cor} = C_N xNx\eta_1 x\eta_2 x\eta_3 x\eta_4$ *(Suggested by Bowles (1997*))…………………..… **Eq. 2.2**

$N_{cor} = C_N xNx\eta_1 x\eta_2 x\eta_3 x\eta_4 x\eta_5 x\eta_6$ *(Suggested Mostafa and Mahamoud (2013))* ... ...**Eq. 2.3**

Authors have been proposed various correction factors; it is applied to field SPT N-values. Different researchers suggest different values of correction factors, and it is listed in Table 2.2.

Table 2.2 Summary of correction factors by different author (Aggour and Radding, 2001)

	Seed (1984) per McGregor and Duncan (1998)	Robertson & Wride (1997)	Bowles (1997)	Skempton (1986)
Hammer Type η_1= Er/Era				
Automatic Pulley/Safety Donut	1.67 1 0.75	0.8 –1.5 0.7 –1.2 0.5 –1.0	1.14 –1.43 1 –1.14 0.64	-
Rod Length Correction, η_2				
>30m 10-30m 6-10m 4 -6m 3-4m 0-3m	1 1 1 1 1 0.75	>1 1 0.95 0.85 0.75 -	1 1 0.95 0.85 0.75 0.75	1 1 0.95 0.85 0.75 0.75
Sampler Correction, η_3				
No liner With liner: loose sand With liner: dense sand, clay	-	1.1-1.3 1 1	1 0.9 0.8	1.2 1 1
Borehole Diameter Correction, η_4				
Parameter 60-120mm 150mm 200mm	-	 1 1.05 1.15	N4 1 1.05 1.15	 1 1.05 1.15
Anvil correction η_5				
Small Large Safety	0.85 0.7 0.9	-	-	0.7-0.8 0.6-0.7 0.7-0.8
Corrections for Blow Rate η_6				
>20 <20	10-20b/m - 0.95 10-20 b/m - 1.05	-	-	-

2.3.3 SPT limitations

The SPT should not be relied on in soils containing coarse gravel, cobbles, or boulders, because the sampler can become obstructed, resulting in high N-values. The test should not be relied on for cohesionless silts because dynamic effects at the sampler tip can lead to erroneous strength and compressibility evaluations. The test also has little meaning in soft and sensitive clays (Kulhawy and Mayne (1990, as cited in Sabatini and Bachus, 2002).

The limitation of SPT in fine grained soils is due to the variability of drilling rig type, drilling system, test procedures and equipment's. Some of the limitation of standard penetration test is related with drilling process that disturb the ground at the base of borehole, Prone to errors by drillers, device imposes very complex strain paths to the soil and no theory at present is capable of predicting what are the most influential factors affecting the N-value (Clayton, 1995).

Many engineers have experience using SPT for foundation design purposes; even though the standard accepted correlations are often based on limited laboratory reference tests. Additionally, variability associated with hammer types used (donut, safety, and automatic) and specific testing errors result in relatively poor correlations for evaluating performance properties for design, especially for cohesive soils. The test does provide a rough index of the relative strength and compressibility of the soil in the vicinity of the test (Sabatini and Bachus, 2002).

According to Clayton (1995), the result of a standard penetration test is influenced by three main groups of factors; drilling technique, SPT test equipment and test procedures. Drilling techniques variation produce large difference in SPT result in this respect the driller plays the important role while the way that uses the tools and it is a great influence on penetration resistance. The sources of errors in carrying out SPT tests are listed in Table 2.3.

Many researchers were investigating on the performance of the SPT for evaluating the sources of uncertainty including Schmertmann (1975) and Kulhawy and Mayne (1990), are mentioned that sources of uncertainty which includes sources depending on encountered soil, due to presence of water, sources related to equipment and its maintenance and site investigation procedure (Rabiel and Albata, 2012).

Table 2.3. Common source of errors while carrying out SPT tests Kulhawy and Mayne, 1990).

Cause	Effects	Influence on SPT-N value
Reducible sources related to equipment and its Maintenance		
Hammer weight inaccurate	Hammer energy varies	Increases/Decreases
Lack of hammer free fall because of ungreased sheaves, new stiff rope on weight, more than two turns on cathead, on complete release of rope each drop	Hammer energy reduced	Increases
Borehole diameter		
Sampler		
Rod Length		
Careless blow count	Inaccurate results	Increases
Reducible sources with careful site investigation Procedure		
Hammer strikes drill rod collar eccentrically	Hammer energy reduced	Increases
Careless measure of hammer drop	Hammer energy varies	Increases

Use of bent drill rods	Inhibited transfer of energy of sampler	Increases
Inadequate cleaning of hole	SPT is not made in original in-situ soil. Therefore, spoils may become trapped in sampler and be compressed as sampler is driven, reducing recovery	Increases
Failure to maintain adequate head of water in borehole	Bottom of borehole may become quick and soil may rinse into the hole	Decreases
Errors by drillers water head, depth measurement errors		
Use of non-standard sampler	Corrections with standard sampler not valid	Increases/Decreases
More than two turns on cathead		
Incomplete release of rope in each drop		
Sources due to presence of water		
Pore pressure generation		
Moisture-sensitive behavior		
Sources depending on encountered soil		
Vertical Stress		
Coarse gravel or cobbles in soil	Sampler becomes clogged or Impeded	Increases

2.4 Standard Penetration Test and Fine Grained Soils

SPT is one of the relatively cost-effective and yet informative field techniques most commonly used in subsurface probing. According to Sirvikaya & Toğrol (2009, as cited in Frazad and Behzad, 2011), the SPT was originally designed to determine the relative density of cohesionless soils, but it has been applied to fine-grained soils to estimate engineering properties such as undrained compressive strength (q_u), undrained shear strength (C_u), and coefficient of volume compressibility (m_v). However, its applicability for fine-grained soils is still argued.

In geotechnical engineering practice the bearing capacity of soil has been determined from in-situ tests like standard penetration tests, cone penetration tests and plate load tests. The in-situ bearing capacity of soils can be reliably determined using the plate loading test results; however, they are difficult, time consuming and expensive. The SPT can be performed quickly and are economical in comparison to other field or laboratory tests. Several SPT-based methods can be used to estimate the variation of bearing capacity with respect to depth (Mohamed and Vanapalii, 2012).

As stated in Day (2006), the bearing capacity of cohesive soils may be stiffened or softened depending on an increase or decrease of their moisture contents. It became dried-out during the summer and getting wet in rainy season. It can cause heave (upward movement) of lightly loaded foundations and the calculation of bearing capacity has for short-term condition (total stress analyses) and long-term condition (effective stress analyses). Total stress analyses that use the undrained shear strength of the plastic soil and should be determined from field tests, such as the vane shear test, or in the laboratory from unconfined compression tests. If the undrained shear strength is approximately constant with depth, then C_u=C (cohesion) and f (angle of friction) =0. These types of shear strength tests are often referred to as undrained shear strength tests because there is no change in water content of the soil during the shear portion of the test.

According to Terzaghi and peck (1967, as cited in (Clayton, 1995). reports that penetration resistance in cohesive soil is broadly a function of C_u but the relationship is controlled by various factors like plasticity, sensitivity, and fissuring.

Schmertmann (1979) attempted to investigate the derivation of soil resistance in clay soil, at least 70% of the soil resistance can be derived from side shear, the remainder coming from end bearing capacity. Since end bearing capacity is determined by undisturbed undrained shear strength, and side shear by remolded strength a modest sensitivity of 10 will increase a C_u/N-value from 5 for insensitive clay to 13.5 if the clay is sensitive. An estimate of the influence on the C_u/N-value ratio can be obtained for the insensitive London clay by examine the value of this ratio when remolded samples are used to determine undrained shear strength. The compressibility of clay cannot be expected to have any significant influence on SPT penetration resistance.

The SPT produced dynamic failure condition therefore compressibility or penetration resistance correlations will depend up on the broad relationship between the undrained strength of the material and its stiffness, which occurs as a result of the influence of void ratio upon the two valuables. Different researcher indicated on their studies the effect of soil type on the SPT N-value result related to its stiffness. Clayton (1995), mentioned different researcher works around soil type and its effect for example; studies made by Bosscher and Showers (1987), indicates the effect of soil type on the amount of energy losses during SPT operations with stiffer soils subject to higher losses which in turn, falsely increases blow counts. Yokel (1989) studied the mechanism of energy transfer in SPT sampler and concluded that the amount of energy actually used to penetrate the sampler depends on the stiffness of soil such that it decreases with increase in number of blows. Lee et al. (2010), analyzed the reaction of SPT sampler as it strikes soils of different stiffness. They came to conclusion that two different secondary impacts occur depending on soil penetration resistance where in soils with $N < 25$, additional sampler penetration is to be expected due to rebound impact while for $N > 50$; the secondary impact does not contribute to further driving of the sampler hence larger N-values.

2.5 Correlation with Undrained Shear Strength

In geotechnical engineering, problems can be solved directly by taking the N-value result and proceed directly to the required quantity in design calculation it includes allowable bearing capacity, settlement, liquefaction potential of sand and indirectly uses the test result to derive geotechnical parameters it includes undrained shear strength of clay, effective angle of friction of sand, unconfined compressive strength of clay, coefficient of volume compressibility (Alan and John, 2010).

In cohesive soils, SPT N-values are usually used to correlate with undrained shear strengths several authors suggest different correlations. First study to determine the relationship between q_u - N (SPT) was done by Terzaghi & Peck (1967). Their study was done on a variety of fine- grained soils that only examined q_u and N-value which did not considered other parameters.

Frazad and Behzad (2011) summarized various researcher work; Stroud (1974), examined the different relationships and used the shear strengths that were obtained from UU(unconfined undrained test) but then results of Sowers (1979), were shown that C_u increases with increasing in plasticity index. Sirvikaya & Toğrol (2002), made a wider study on different fine-grained soils using results of UCS experiment and presented a new correlation. Hettiarachchi & Brown (2009), assumed SPT sampler as an open-end pile and presented a correlation using energy balance method based on N_{60}.

Bowles (1997), gives an empirical correlation between N_{cor} and unconfined compressive strength that can be estimated by the following equation:

$q_u = kxN_{cor}$ ……………………………………………..…………………………..**Eq. 2.4**

Where, k = proportional factor, k =12 has been recommended by Bowles

In line with this, Stroud (1974, as cited in Clayton, 1995) had reported good correlations for insensitive fissured over consolidated clays between N-value and C_u. Stroud suggested that N- value in cohesive soils is not only a function of undrained shear strength but also it has a relationship with plasticity index.

$C_u = f_1 * N_{60}$ ……………………………………………………………………………………………**Eq. 2.5**

Values of f_1 depend slightly up on the plasticity so the clay if the clay is sensitive; this method will also yield an underestimate of undrained shear strength.

Table 2.4 Summarized empirical correlation of C_u versus N-values suggested by different researcher (fine grain soils)

Terzaghi & Peck (1967)	6.25N
Bowles (1997)	$6N_{cor}$
Sivikaya &togrol (2002) (UCS)	4.32N
Hettiarachichi & Brown (2009)	$4.1N_{60}$
Sirvikaya (2009)(UCS)	$C_u = 2.41N - 0.82W_n + 0.14LL + 1.44PI$ $C_u = 3.24N_{60} - 0.53W_n - 0.43LL + 2.14PI$
Frazad & Behzad (2011)(UCS)	$C_u = 1.5N - 0.1W_n - 0.9LL + 2.4PI + 21.1$ $C_u = 2N_{60} - 0.4W_n - 1.1LL + 2.4PI + 33.3$

2.6 Previous Works

Many researches have been done around reliability of standard penetration test estimating the engineering properties of fine grained soils and determination of bearing capacity.

Mustafa and Mohammed (2013), attempted to measure the reliability of using standard penetration test in predicting some properties, such as Atterberg limits and shear strength parameters of silty clay with sand soils. The results of the research indicated that the shear strength of soil affects SPT number. However Atterberg limits LL, PL and PI has no effect on SPT result. Frazad and Behzad (2011) examined the SPT ability to predict undrained shear strength of fine-grained soil using the multi linear regression analysis. Results showed that considering other parameters such as natural water content (w), liquid limit (LL) and plasticity index (PI), in addition to N (SPT), increase the correlation coefficient of estimation.

Abdulrazzaq and Hameed (2011) this research considers the most famous methods to evaluate the bearing capacity from the SPT using FD interpolation method program must be used with caution since it is not a replacement of sound hand calculations associated with engineering judgment and experience.

There are no plentiful studies on SPT method reliability in Ethiopia there are only few recent studies are conducted in the methodology of assessing the bearing capacity of the Addis Ababa soils. These related works are a master thesis research works by Lamesgin Melese (2014) and Woyenishet Tadesse (2015).

Lamesgin Melese (2014) focused on comparing the different bearing capacity calculation methods and signifies that the bearing capacity equation and SPT method gave similar results for red clay soil, in Addis Ababa. However, the bearing capacity of dark clay soil that rather showed higher results, when calculated using empirical method than the others; while the other methods gave more or less similar results. Weinshet Tadesse (2015) attempted to compare the results of bearing capacity calculation using SPT method to that of the laboratory test results in common types of soil in Addis Ababa and her results also show that the both approaches gave more or less the results.

2.7 Evolution of Methodology for the Present Study

A literature review encompassing data to obtain an overview of the current information within the knowledge base regarding the relationship of SPT N values and shear strength to determine bearing capacity. Generally from literature review it has been learned that even if standard penetration test widely practice in geotechnical investigation, it is affected by various factors related with test equipment, test procedures and other factors. Many researchers identified the main influence that need to be corrected. Consequently correction factors have been developed including sampler correction, energy correction, borehole diameter correction and rod length correction. Recent published papers (Mustefa and Mahmoud (2013) and McGregor and Duncun (1998)) has increase the number of correction factor to six including anvil and blow count corrections but still needed improvement in the standardization including other factors.

Hence, such test method (SPT) should be investigated, adequately identified the major source of errors while carrying out SPT tests prior to foundation investigation. In Ethiopia SPT method is a common practice in foundation engineering in the determination of bearing capacity despite the reliability of tests. Thus, to study the reliability of the test in fine grain soils the present research problem was conceived. The present study was conducted on SPT method reliability in Addis Ababa on where the soil samples were taken. During the present study the following were conducted.

Field testing, borehole logging and soil samples for selected laboratory testing's and to have a correlation between in-situ test (SPT) and Laboratory test (UCS). Consequently a comparison was made with the parameters and previous correlations. Lastly, based on the results of analysis and interpretations, conclusions and recommendations have been suggested.

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CHAPTER 3 DESCRIPTION OF THE STUDY AREA

3.1 Introduction

The study area was defined in terms of soils and geology, physiography and drainage, climate, hydrogeology and seismicity. Common factors required for the prediction of soil type are rock type, climate, and topography (Roy, 2006).

3.2 Location of the Study Area

Addis Ababa (the study area) is located in the central part of Ethiopia. It is situated in the western margin of the Main Ethiopian Rift and represents a transition zone between the Ethiopian Plateau and the rift with poorly defined escarpment. It is located between 986000 N to 100500 N and 463000 E to 481000 E (Fig. 3.1). Its maximum elevation is at the northern tip around 3000 m and decreasing towards the Ethiopian rift valley, south direction, which is measured about 2000 m. The city covers a total area of about 540 square kilometers.

3.3 Topography and Drainage

Topography controls the rate of weathering by partly determining the amount of available water for each zone of weathering. Precipitation will tend to run off hills and accumulate soils in valleys and hollows (Blight (1997 as cited in Hana Tibebu, 2008)).

Terrain investigation provides a basis for identifying the mode of occurrence of soil formations and classifying them by origin. Landform and stream forms and pattern are the basic interpretative factors for terrain exploration. Soils are sub classified on the basis of their mode of occurrence, which refers to the landform or surface expression of a deposit, or its location relative to the regional physiography. Ethiopia can be divided in to four major physiographic regions widely known as the Western plateau, Southern plateau, the Main Ethiopian Rift and Afar Depression (Mengesha Tefera et al., 1996).

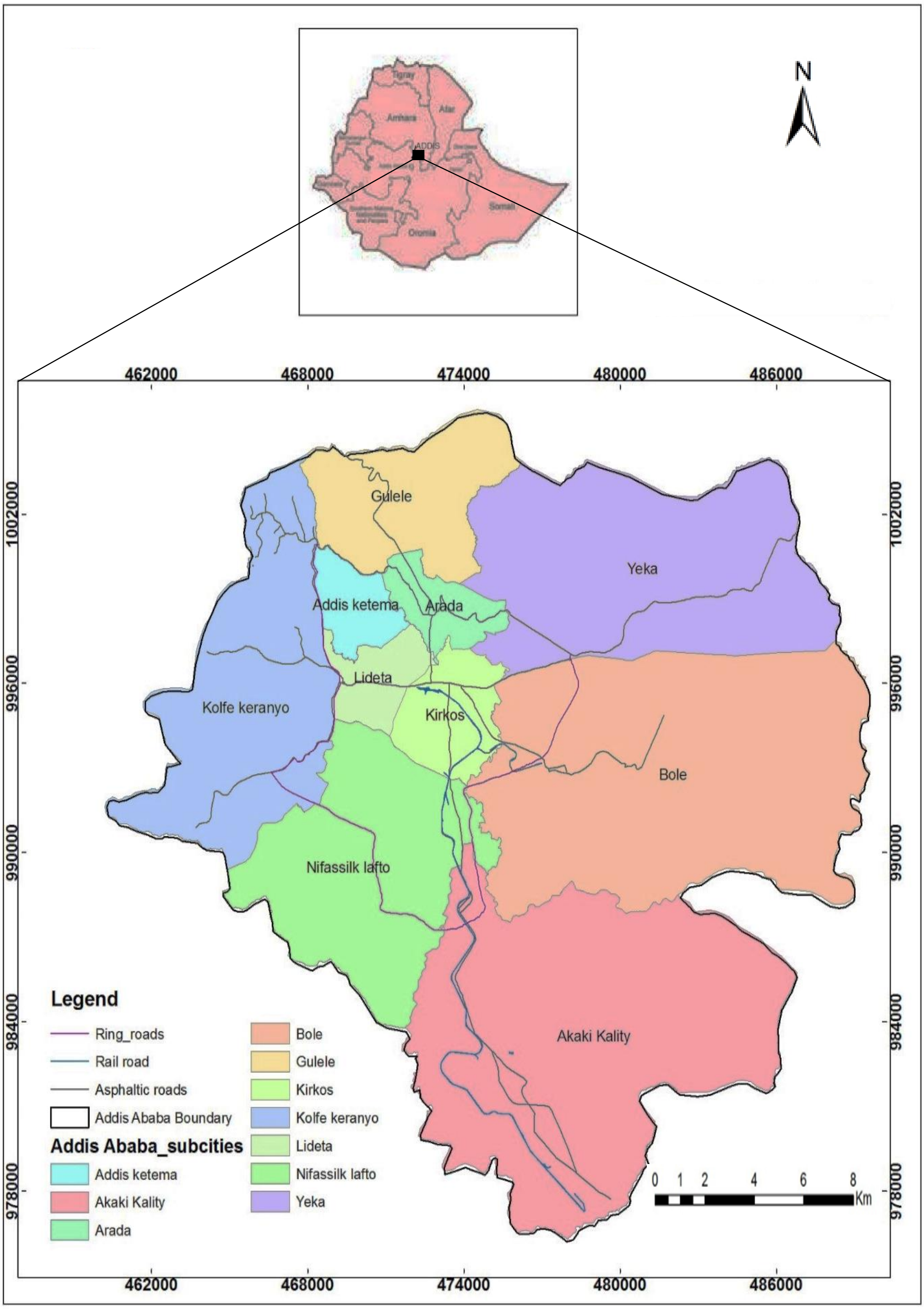

Fig 3.1 Location map of the study area

Addis Ababa is located on a plateau with an elevation ranging from 2000 to 2800 m a.s.l. on the shoulder of the western Main Ethiopian Rift escarpment. The south eastern and the north western parts of Addis Ababa are above 2000 m a.s.l. Addis Ababa is located over a hanging valley plain surrounded by high standing volcanic mountains, namely Mt Entoto in the north, Mt Menagesha in the west, Mt Wechacha in the south west, Mt Furi in the south and Mt Yerere in the south east. Internal morphology of Addis Ababa is characterized by undulating ridges and valleys which can be marked as one travels from west to east across the city. The central part of Addis Ababa is relatively flat and undulating topography. The presence of domes and river valleys create this undulating topography. Further southwards, the topography becomes very gentle and a very wide area falls under a smaller elevation range of 1960 m and 2160 m a.s.l (Kebede Tsehayu & Tadesse H. Mariam, 1990).

In the study area the drainage shows dendritic pattern (Fig.3.2). These drainages are denser to the southern part. The drainage pattern is governed by the geology and physiographic setup to the area. The major rivers which crossing the city are Kebena and Akaki.

Generally both parameters, topography and drainage condition of the study area, also played a major role on the color and distribution of the soils. In relatively gentle and steep slopes of the northern, northeastern and northwestern parts of the city, light to yellowish brown soils are common. These areas are well grained in favor of the topography. The dark grey soils are dominate in central part of addis ababa and low lying areas of the city where the surface drainage is poor (Kebede Tsehayu & Tadesse H. Mariam, 1990).

3.4 Climate

The two components of climate which governs the rate and types of soil formation are precipitation and temperature. Ethiopia is classified into five climatic zones which includes include "Kur" (Alpine), above 3000 m mean sea level; "Dega" (Temperate), 2300 m to about 3000 m; "Weina Dega" (Sub-tropical), 1500 m to about 2300 m; "Kolla" (Tropical), 800 m to about 1500 m and "Bereha"(Desert), less than 800 m. The study area is largely characterized by a represented by sub-tropical or “Weina Dega”(EMA,1981).

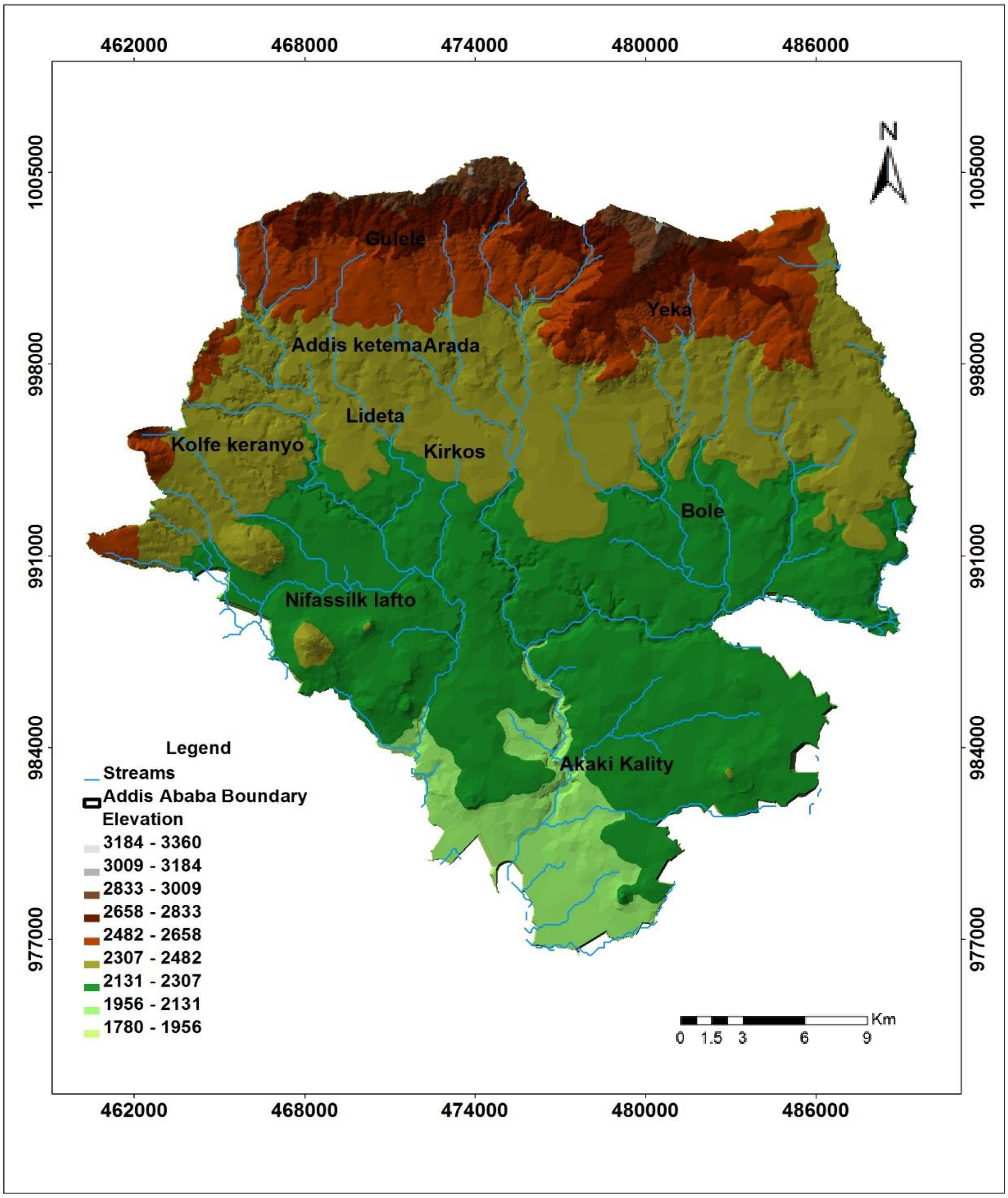

Fig 3.2 Physiographic and drainage map of the study area

3.4.1 Rainfall

Rain fall variations have a major impact in the water table fluctuations. This cause difference in the soil (cohesive soil) moisture content which have influence on the engineering properties of soil in terms of strength and degree of firmness. Even the field tests conducted in dry season and rainy season may get different result even in the same site due to the raising and lowering of water level. Hence, prior of rainfall variations helps knowledge understand its impact on ground water table fluctuation for appropriate design and safety of engineering structures (Daniel Gemechu, 1977).

The variation in the seasonal distribution of rainfall in Ethiopia can be attributed by the reference to the position of the Inter-Tropical Convergence Zone, the relationship between upper and lower air circulation, the effects of topography and the role of local convection currents and the amount of rainfall (Daniel Gemechu, 1977). Addis Ababa is provided with on balance 1089 mm of rainfall per year or 90.8 mm per month. Wet climate in which the rainy season prevails is from June to September.

In Addis Ababa rainfall intensity variation is attributed to differences in topography. The high elevated areas such as the Entoto receive relatively greater precipitation than lowland areas around Bole and Akaki. High rainfall data is recorded in the month of August whereas in the month of January and February no rainfall data is recorded. This shows that there is variation in the amount of rainfall within Addis Ababa with difference in altitude.

The mean monthly and annual mean rainfall of National Meteorological Services Agency (NMSA) data are shown in Table 3.1.

Table 3.1 Mean annual and monthly precipitation data of Addis Ababa (2010 to 2014)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Mean annual
2010	1.3	3.0	1.9	5.1	21.4	20.4	37.2	41.5	27.4	2.7	1.7	1.9	13.8
2011	0.1	0.0	2.2	1.7	7.4	10.6	14.0	7.2	8.0	1.3	4.8	0.0	4.8
2012	0.0	0.0	1.2	0.6	7.0	8.9	15.2	14.1	5.2	2.0	2.4	0.5	4.8
2013	0.2	0.2	0.3	1.0	6.4	5.3	13.7	9.4	8.6	9.9	3.8	0.2	4.9
2014	0.6	1.1	4.7	6.0	10.4	8.4	8.2	-	-	-	-	-	5.6

(Source: National Meteorological Services Agency)

3.4.2 Temperature

Temperature variability become an effect in the weathering of rocks and disintegrated in to soil. Weathering is increase in warmer temperature. Addis Ababa had thick weathered profile especially in flat and gentle lands. Although temperature has no direct influence on the engineering properties of soils, its long term effect of accelerating weathering of near surface. The overall temperature in Ethiopian highlands is lower than those in tropical lowlands. The average fall in temperature is 0.6^0C for every 100 m rise in elevation. The average temperatures are typically tropical and fluctuate by 5^0C between the coldest and warmest months (Grifffiths, 1972 as cited in Habtamu Solomon, 2011).

The maximum temperature is expected to be between March and May and the minimum between July and September. The mean annual temperature recorded at Addis Ababa ranges from a maximum of 27 °C to a minimum of 12.1 °C. The compute average maximum and minimum temperature is present in table below.

Table 3.2 Temperature variation in Addis Ababa

Year		**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**	**Mean Annual**
2010	Max	28.3	29.4	30.4	30.0	24.7	21.5	18.4	18.9	20.9	24.6	25.6	26.4	24.9
	Min	12.5	14.9	14.3	15.7	15.4	13.7	13.1	13.2	12.6	10.7	10.8	11.7	13.2
2011	Max	12.4	12.4	14.4	15.1	13.7	12.8	11.6	11.3	10.9	10.4	12.1	11.1	12.4
	Min	28.4	32.5	31.4	33.0	26.9	23.7	21.2	20.0	21.7	25.2	24.9	27.9	26.4
2012	Max	29.4	32.8	32.8	31.2	29.5	24.5	20.2	20.9	22.3	25.3	26.5	28.2	27.0
	Min	11.7	12.5	14.4	13.5	13.6	11.9	11.7	11.0	11.3	10.4	11.3	12.1	12.1
2013	Max	29.8	32.7	31.4	31.4	28.1	23.7	19.6	19.8	22.0	22.9	24.7	26.4	26.0
	Min	13.4	14.0	14.1	13.3	13.8	11.7	10.8	10.2	11.2	11.4	12.2	10.7	12.2
2014	Max	28.7	29.7	29.3	28.3	24.7	23.8	20.3	-	-	-	-	-	26.4
	Min	14.0	12.2	14.2	14.4	14.0	12.7	12.6	-	-	-	-	-	13.4

(Source: National Meteorological Services Agency)

3.5 Hydrogeology

Several different flows with unconformity favored the occurrence of multilayered aquifer which is manifested by several contact springs mainly concentrated on the northern part. Most of the aquifers are confined below the clay and hence storage coefficient is very low. The ground water potential of the areas around shola, Bole and the surroundings is very low. The eastern part is covered by a very thick black cotton soil which is characterized by a very low Infiltration. Around Filwoha, Ghion hotel, Stadium and Legehar the ground water is exposed at very shallow depth. Fractured volcanic rocks are the main contribution in the potential groundwater of Addis Ababa. The intense fracturing combined with the jointing of the rock favored to be a good aquifer. Ignimbrite and basalts are better aquifers because of intensive fracturing caused by tectonism and jointing by the cooling effect (Kebede Tsehayu & Tadesse H. Mariam, 1990).

The major aquifer in Addis Ababa is basalts, rhyolites, trachyte, scoria, trachy basalts, welded tuffs, and unwelded tuffs. The main aquifers can be categorized in to three groups which includes shallow aquifer of the weathered volcanic rocks and alluvial sediments along the river courses, deep aquifer of the fractured volcanic rocks that traps fresh ground water and thermal aquifer along Filuha fault (Tamru Alemayehu et al,. 2006).

3.6 Seismicity

Different literature and historical records is confirmed that earthquakes have occurred in Ethiopia many times in the past. Most shocks originate along the rift structures of red sea, Gulf of Aden and the Ethiopia rift. Currently hazard levels due to expected earthquake occurrences are available in the form of regional seismic zonation map of the country (RADIUS, 1999).

The seismic activity around Addis Ababa is related to the main Ethiopian rift. Hence Addis Ababa is placed in the western margin of rift, which is among the tectonically active areas of the world. Due to this fact, recurrence of earthquake poses a significant risk in the area. The level of hazard due to earthquake occurrence of a certain magnitude/intensity can be significantly affected by surface geology, which can vary appreciably within close proximities (EBCS-8, 1995).

According to the seismic hazard map presented in Ethiopian Building Code Standard (EBCS-8, 1995), the country subdivides in to five seismic zones. Depending on the local hazard the seismic zonation map of the Ethiopia shows the distribution of the expected hazard, which follows the physical boundaries of the Rift Valley itself. Areas, which are near the valley, experience more hazard than those located far away. According to the seismicity map of Ethiopia, Addis Ababa is lies in the second high risk zone.

Hence topography play an important role in amplifying seismic motion, Entoto relatively high intensities following earthquake that occurred outside Addis Ababa. Around Yeka, Abware and Filwoha which reported of high intensities fall along the Filwoha fault (Kebede Tsehayu & Tadesse H. Mariam, 1990).

According to Seismic zoning of Addis Ababa as per RADIUS Project (1999) the study area is located within a Peak Ground Acceleration (PGA) zone ranging from 0.13 to 0.5m/s^2 which is classified as seismically moderately vulnerable for potential damage.

Therefore, it advisable to check the stability of the engineering structures against seismic effects and appropriate seismic design considerations shall be taken into account during the design stage.

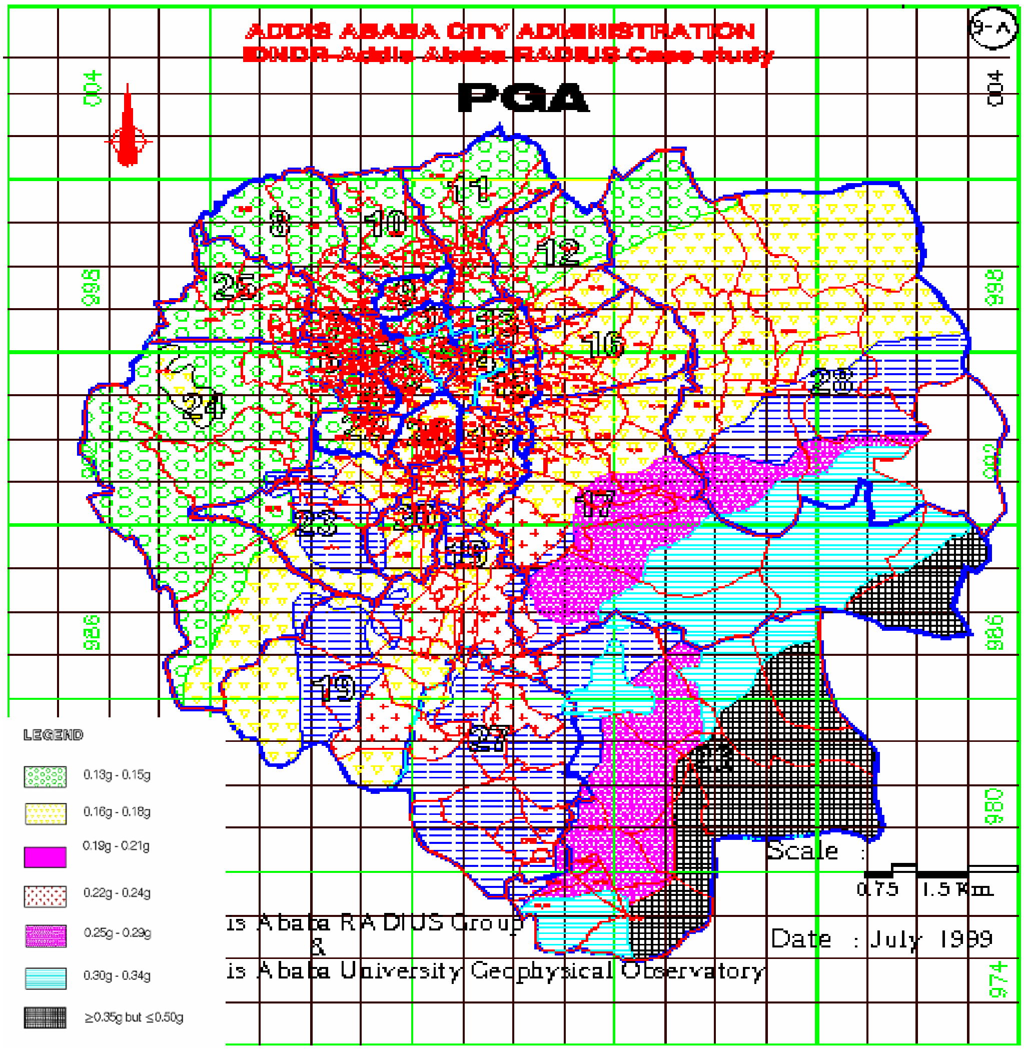

Fig 3.3 Seismic zoning of Addis Ababa as per RADIUS Project (1999).

3.7 Regional Geology and Tectonics

The Tertiary volcanic of central Ethiopians divided in three stages of volcanism and tectonism Zanettin (1997, cited in Asseged Getaneh, 2007). Where the first transitional (thiolettic) flood basalt followed by alkali rhyolite and ending with alkali basalt. At first the volcanism has occurred on a large elongated basin. The outer part of the basin was uplifted and deep eroded during this stage the volcanism dies out on the uplifted area.

The outcrops of Addis Ababa and its environs are entirely Cenezoic volcanics sequences with different types of recent sediments. The Tertiary Volcanics include basalts, ignimbrite, rhyolite, trachyte, tuff and occasional pumice. From stratigraphic point of view, the tertiary volcanic rocks are classified as Trap Series, Entoto Silicics, Addis Ababa Basalt and Younger Volcanics in chronological order from the oldest to the youngest.

The Addis Ababa area is characterized by tectonic and volcanic activities associated with the formation of the rift valley that commenced in Miocene following major uplifting called the Afro-Arabian dome during early Tertiary period. N and NE trending steep normal faults bounds the area in the east and west. Most of which have sub parallel trend as the main Ethiopian rift fault. Besides, the region is traversed by the major east-west trending Yerer-Ambo lineament which separates the southern Main Ethiopian Rift and the northern Afar Rift. The western marginal zone of the Main Ethiopian Rift is affected by the younger, N-S trending Wonji faults forming small horst and graben structures (Assegid Getaneh, 2007).

3.8 Geology of Addis Ababa

The geological map of Addis Ababa compiled by Hailieselasse Girmay and Getaneh Assefa (1989), Assegid Getahun (2007), Efrem Beshawered (2009) and WWDSE (2008). According to these previous geological studies, Addis Ababa is covered by volcanic rocks dominated in the lower part by basaltic lava flows (Addis Ababa basalt), followed by a pyroclastic sequence, mainly formed by ignimbrites (Addis Ababa Ignimbrite), followed by central composite volcanoes (Central Volcanoes unit), and finally small spatter cones and lava flows (Akaki unit) and superficial deposits.

According to Assegid Getahun (2007), Addis Ababa is covered by volcanic rocks and superficial deposits consisting of superficial deposits of mainly CLAY, Repi basalt unit (strongly jointed basalt topped by trachy basalt), Entoto mixed volcanic unit (Silicic volcanic rocks with minor sediments), Lower ignimbrite unit (Ignimbrites, un-welded tuffs, ash flows and trachyte) and Ignimbrites and associated pyroclastic unit of Yerer volcanic in the east (Fig. 3.4).

3.8.1 Addis Ababa basalt

Extensively crops out along Akaki, Kebena, and Dukem rivers at the east to southeastern part of Addis Ababa, and represents the oldest unit of the area. It consists of essentially sub-horizontal lava flows with thickness ranging from few meters up to 20m. Maximum exposed thickness was found east of Addis Ababa, along the Kebena River. Addis Ababa basalt is predominantly constituted by alkaline and olivine basalts with three main textural attributes, that is, porphyritic, aphyric, and sub-aphyric (Assegid Getaneh, 2007).

3.8.2 Addis Ababa ignimbrite

It is exposed close to Addis Ababa along the Akaki and Kebena rivers. It overlies the Addis Ababa basalt and locally covers the products of the composite central volcanoes of Wechecha and Furi. The sequence is constituted by different flow units, consisting of pale-green to pale-yellow welded and crystal rich ignimbrites (Assegid Getaneh, 2007).

3.8.3 Central volcanoes unit

Includes the Yerer volcano and the product of the two composite volcanoes wechecha and Furi west and southeast of Addis Ababa, respectively. Wechecha and Furi volcanoes are two large edifices composed by predominant trachyte with minor pyroclastics. Yerer represents the largest volcanic edifice in the region, with a relief of 1000m from the plain and 14km wide along east-west direction. Products mainly consist of trachytes, even if pyroclastics are widespread mainly in the central part eastern sector. The highest part of Yerer volcano was affected by a more recent volcanic activity that produces spatter cones and associated basalt (Assegid Getaneh, 2007).

3.8.4 Akaki unit

Crops out east of Addis Ababa and consists of scoria and spatter cones with associated tabular lava flows and phreato- magmatic deposits. Alluvial deposits covering these units consists of regolith, reddish brown soils, talus and alluvium with maximum thickness of about two meters (Assegid Getaneh, 2007).

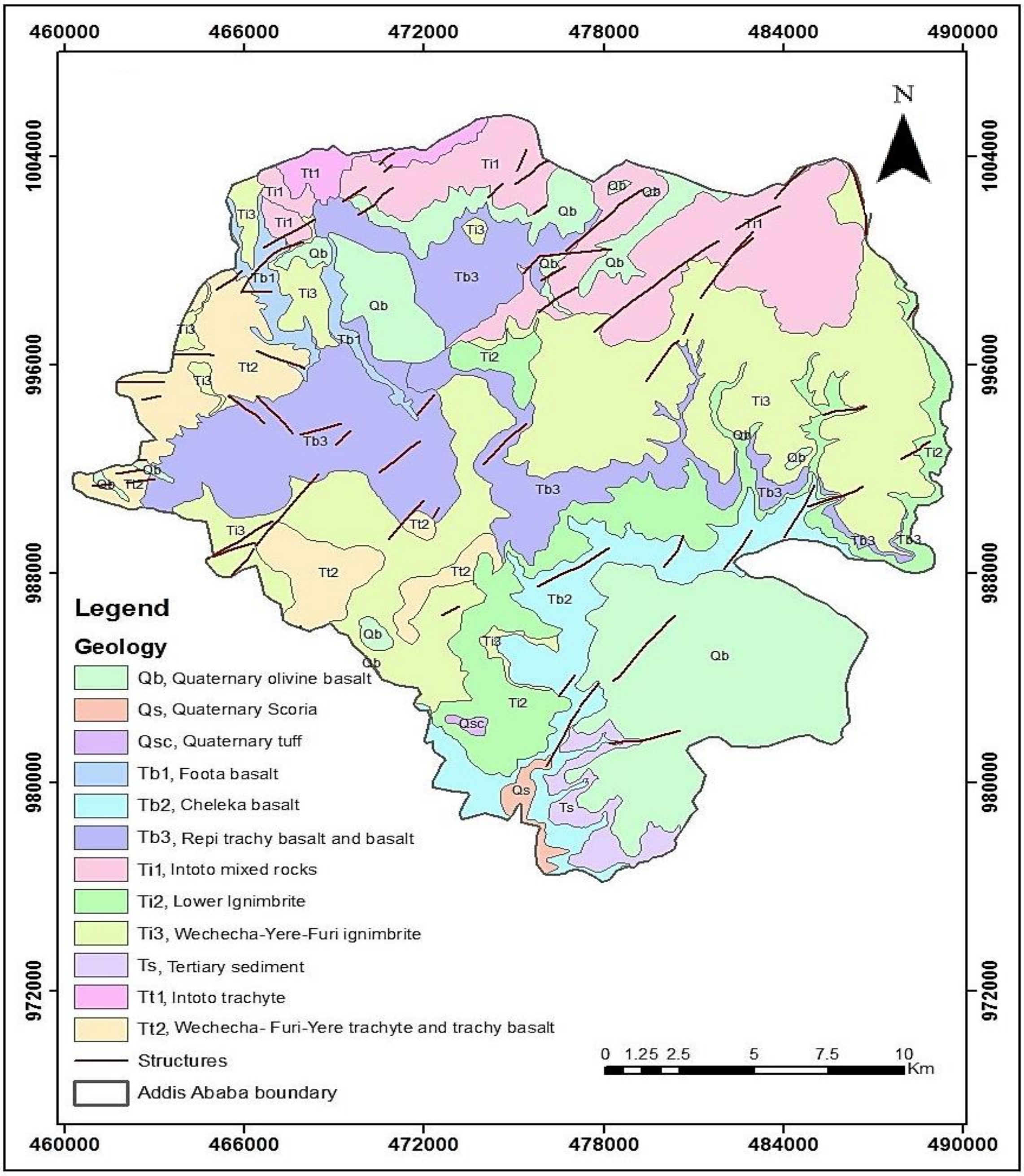

Fig 3.4 Geological map of Addis Ababa (Assegde Getaneh, 2007)

3.9 Soils in the Study Area

Casagrande (1948 as cited in Bell, 2006) advanced one of the first comprehensive engineering classifications of soil. In the Casagrande system, the fine grained soils are distinguished from coarse grained soil on the basis of particle size. The properties of clay soils are very dependent upon their moisture content and they can be much more complex, mineralogical, than more granular soils.

In general, according to the engineering geological map of Addis Ababa (Kebede Tsehayu & Tadesse H. Mariam, 1990), the soil units of Addis Ababa mainly consists of the residual, lacustrine, alluvial and colluviel soils. Soils that remain at the site of weathering are called residual soils. These soils retain many of the elements that the parent rocks comprise. The residual soils have resulted from in-situ weathering of parent rocks that are not subjected to transportation and are still in the place of their origin. These soils are located mainly in central part, Gulele and Kolfe regions. According to unified classification system (USCS) these soils grouped in lean clay (CL). This soil displays intermediate to high plasticity and they have low degree of swelling.

Alluvial soils are deposited in low land areas after transported by rivers and streams from their origin. Further, the colluvial and alluvial deposits are commonly observed at the foot of hills and river banks. The composition of these soils depends on the environment under which they were transported and is often different from the parent rock (Budhu, 2000). However, these types of soils are rarely exposed along Akaki river channel, around Entoto Mountain and beneath Yeka and Megenagna hills and also along Kebena River. This deposit consists stratified gravel and clay moved by streams from high to low elevation area. This type of soil classified according to (USCS) is plastic silt (ML) and degree of swelling is low (Kebede Tsehayu & Tadesse H. Mariam, 1990).

Lacustrine (Black cotton soil) soils are deposited in flat and low altitude areas where there is lake or small water body. The East and south part of the study areas like Bole, Megenagna CMC, Lideta, Kality and Mekanisa are covered by lacustrine sediments (Black cotton soil). It is possible that during Pleistocene these places were covered by water body. The name of the soil according to USCS is MH (rarely CL). These soils have extremely high plasticity and very high degree of swelling as compared to other soil types. The thickness of this soil is varies place to place (2 – 10m). Coluvial soils are deposited where either soils or disintegrated bed rock move down slope due to gravity. It is located at the foot slope of north eastern part of Entoto silicic and few places (Kebede Tsehayu & Tadesse H. Mariam, 1990).

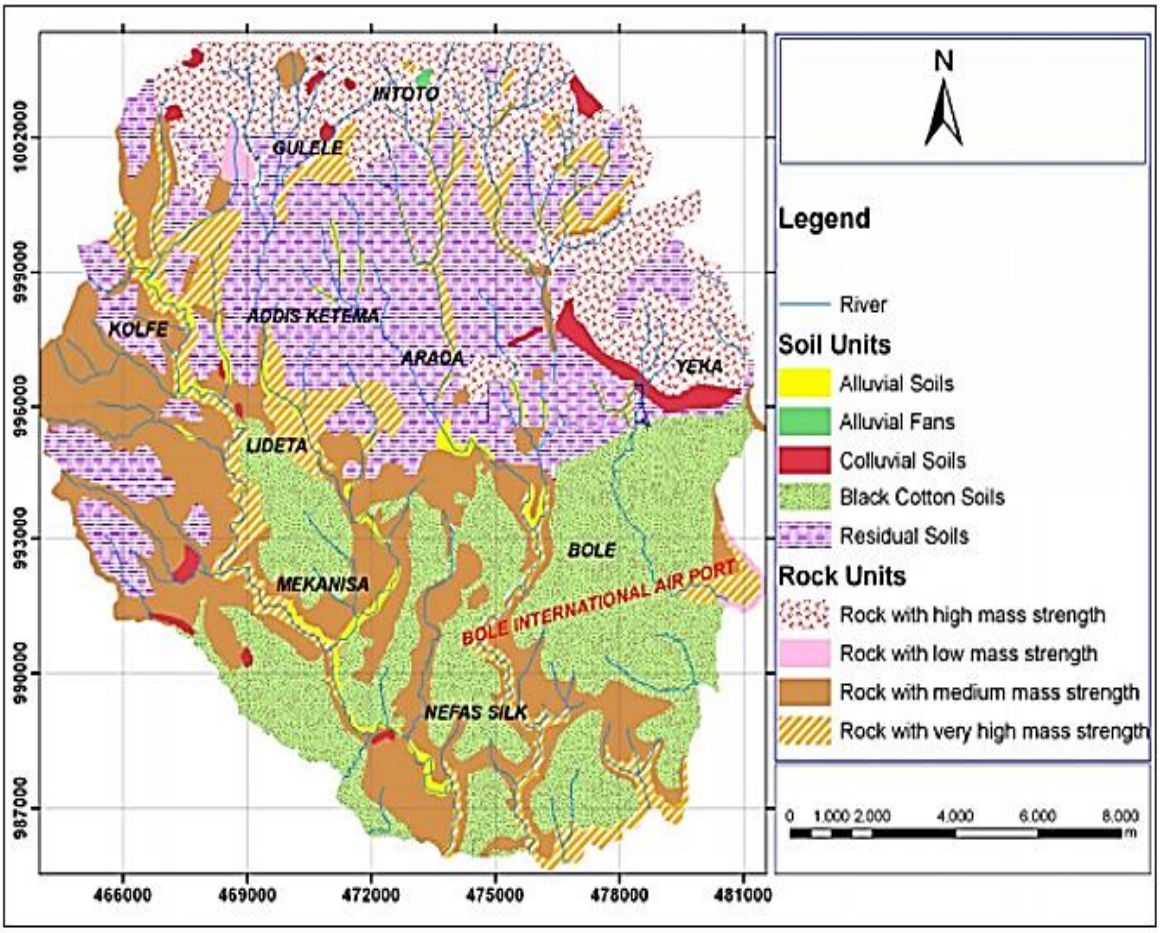

Fig. 3.1 Engineering Geological map of Addis Ababa (Kebede Tsehayu & Tadesse H. Mariam, 1990).

In most part of the study area clay soils have been dominantly exist combined with silts. A clay that found western and central part of Addis Ababa have red color whereas in the eastern and southern part have dark color clay. This variation is come from the way of formation mostly the red soils are residual soil which exist the original place of formation but the dark clay soil place in relatively flat area of the study area.

3.10 Fine Grained Soils in the Study Area

The fine grained soil distributions in the study area are classified based on their color, consistency and plasticity. These are dark clay, red clay and red silt soils.

The red clay soils found on the study area are characterized by firm to very stiff, medium to highly plastic silty clay. A minimum and a maximum of 6 and 30 blows per 300mm penetration were measured. The soil layer has firm to stiff consistency.

Plate 3.1 Red clay soil (Gulele area)

Dark clay soils found in the study area is characterized by firm to very stiff, medium to highly plastic clay (black cotton soil). A minimum and a maximum of 5 and 21 blows per 300mm penetration were measured. The soil layer has firm to stiff consistency.

Plate 3.2 Dark Clay soil around (Akaki area)

The red silt soils found in the study area are characterized by firm to very stiff, medium to highly plastic silty clay. A minimum and a maximum of 5 and 30 blows per 300mm SPT penetration were measured. The soil layer has firm to stiff consistency.

Plate 3.3 Red silt soil (Lamberet area)

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CHAPTER 4 SPT MEASUREMENT DETAILS AND LABORATORY ANALYSIS

4.1 Introduction

SPT is currently the most popular and economical means to characterize in- situ soil strength. It provided the resistance of the soils in the form of number of blows (N-value), from which the bearing capacities of foundation material can be calculated. It is made by dropping a free falling hammer onto the drill rods from a specific height to achieve the penetration of a standard sample tube to the soil (Mwajuma, 2015).

Although great effort has been put into standardizing the SPT procedure, variability is inherent in present procedures. The standard penetration resistance is, in fact, conventionally measured using different kinds of hammers, drill rig types, drill rod lengths, drill rod types, hammer blow rates, different energy delivery systems with different degrees of efficiency, different borehole fluids, and different kinds of sampling tubes. Thus, the test is performed by different equipment and testing procedures as well as different operators (Clayton, 1995).

Due to the above mentioned factors the consistency of standard penetration tests is questionable. This research was designed to measure its reliability for a fine grained soil in Addis Ababa. In doing so, SPT measurements were carried out at different depths in selected area of Addis Ababa where fine grained soils are prominent. Corresponding samples were also obtained to determine undrained shear strength and other physical and engineering properties of soil in the laboratory.

The SPT variables which are considered in this study are N-value, hammer and anvil type, penetration interval, hammer blow rate, drill rod type, rod length, and drill rig type. Accordingly, a total of twenty two borings were used in this study at different site around Addis Ababa as shown in Fig. 4.1. The boreholes were drilled for foundation investigation purpose by private geotechnical firms and representative fine grained soils have been investigated for this research work. Two different types of hammer were used in performing the SPT test (Donut and Automatic hammer).

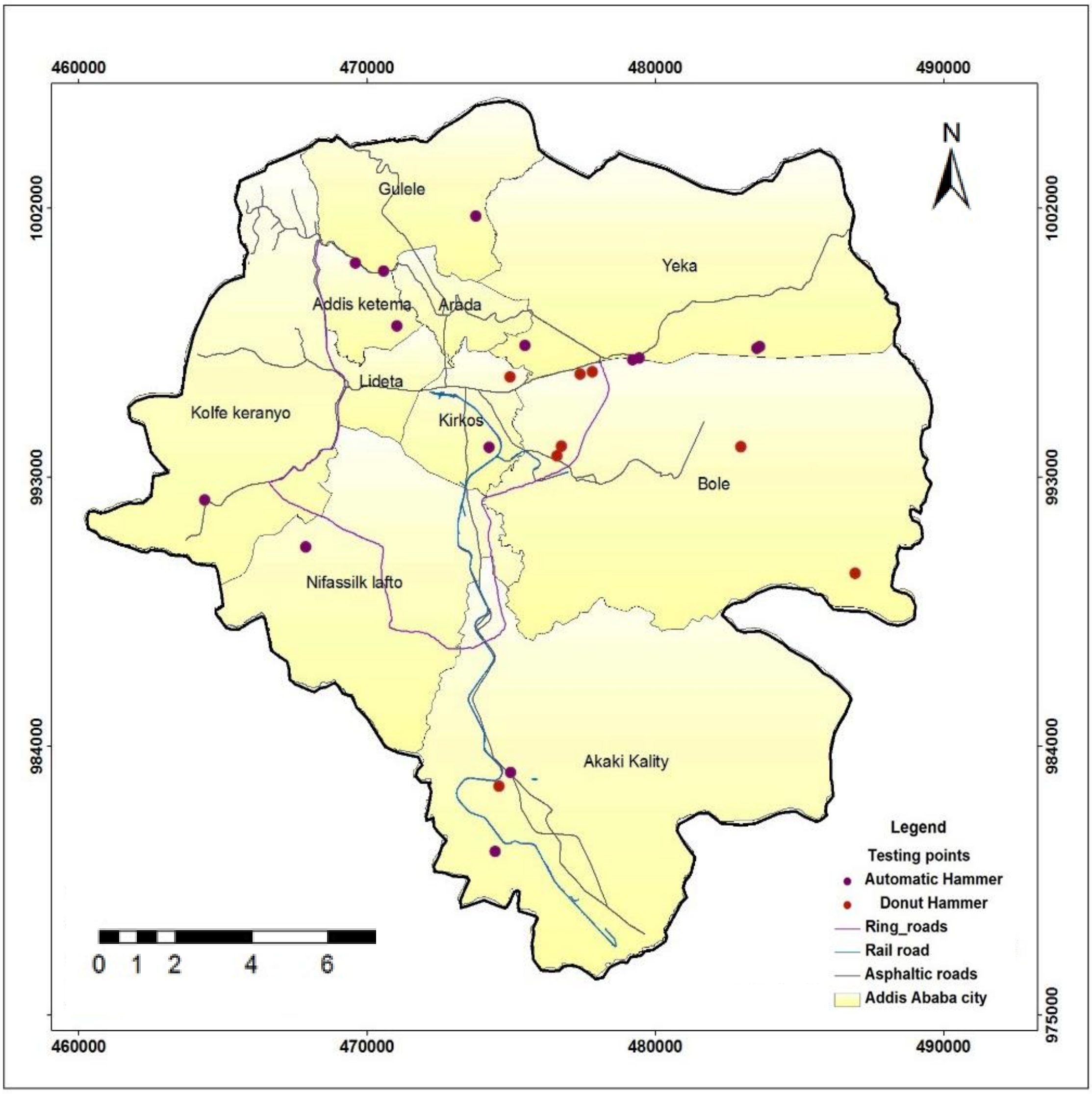

Fig. 4.1 Test point location

4.2 SPT Equipment Details

Standard penetration test tools composed of SPT hammer system (hammer, anvil and lifting head), split spoon sampler, and drill rod. Deploying inappropriate use and damaged tools have a great impact on the result of SPT blow count and on overall processes to determine the bearing capacity (Clayton, 1995). The different equipment used in this study has been described briefly as follows.

4.2.1 SPT hammer system

SPT hammer system is comprised of the hammer itself, the mechanism that lifts and drops the hammer, the anvil stem and anvil or drive- head (Aggour and Radding 2001). The use of

SPT has grown rapidly; three different types hammer systems have been developed: donut hammer, safety hammer, and automatic hammer. Herein the tests were conducted only in two of the hammer type system; donut and automatic hammers. The donut hammer system is the weight centered on a guide pipe which strikes an external anvil above the drill rods and the weight is raised and dropped by the operator and relatively its increase the risk of injury (Lamb, 1997). Unlike the donut type of hammer, the automatic hammer system is a hydraulically powered chain lift device; the only thing that operator does is to open the hydraulic valve in order to start the lifting and dropping sequences and weight which slides over the drill rods and impacts against an internal anvil (Riggs, 1986).

Due to its difficulty of measure the hammer weight in the field, the hammer weight data was collected from drilling rig manual and specifications. Most of the hammers were fall on the range between 62 kg to 63 kg qualified the ASTM standards. The drill rigs were used for this study are SH-700 and SH-100 Korea rigs mounted with an automatic hammer, HXY – 2BT and XY-200 spindle type China rigs with a donut hammer as shown in Fig. 4.2. The hammer drop system the donut hammer used the cathead and rope system whereas the semiautomatic or automatic hammers have a trip system.

The anvil is that portion of the hammer system through which the hammer energy is transmitted to the drill rods as it is impacted by the falling hammer. The hammer and anvil is designed for steel on steel contact when the hammer is dropped. During field observation the anvils have different shape and size.

The hammer type is the most influential due to the variability in energy delivered to the drill rods. Researchers have shown that energy transfer efficiency can be varies from 30% to 90% depending on the type of hammer used. Thus, different drill rig hammer systems give different N values. Hammer efficiency highly depends on equipment and operator skill. It has been found that an inverse relationship exists between the N measured and the efficiency of the hammer (Sherbiny and Salem 2013). These findings and the recognition of the direct impact of this inconsistency on geotechnical design.

Bowels (1997), listed the energy ratio of different counties practice, is between the range of 40% to 60% and 60% to 100% for donut hammer and automatic hammer, respectively. It is evident from the above mentioned values that energy efficiency for each type of hammer varies widely with the local practice in different parts of the world.

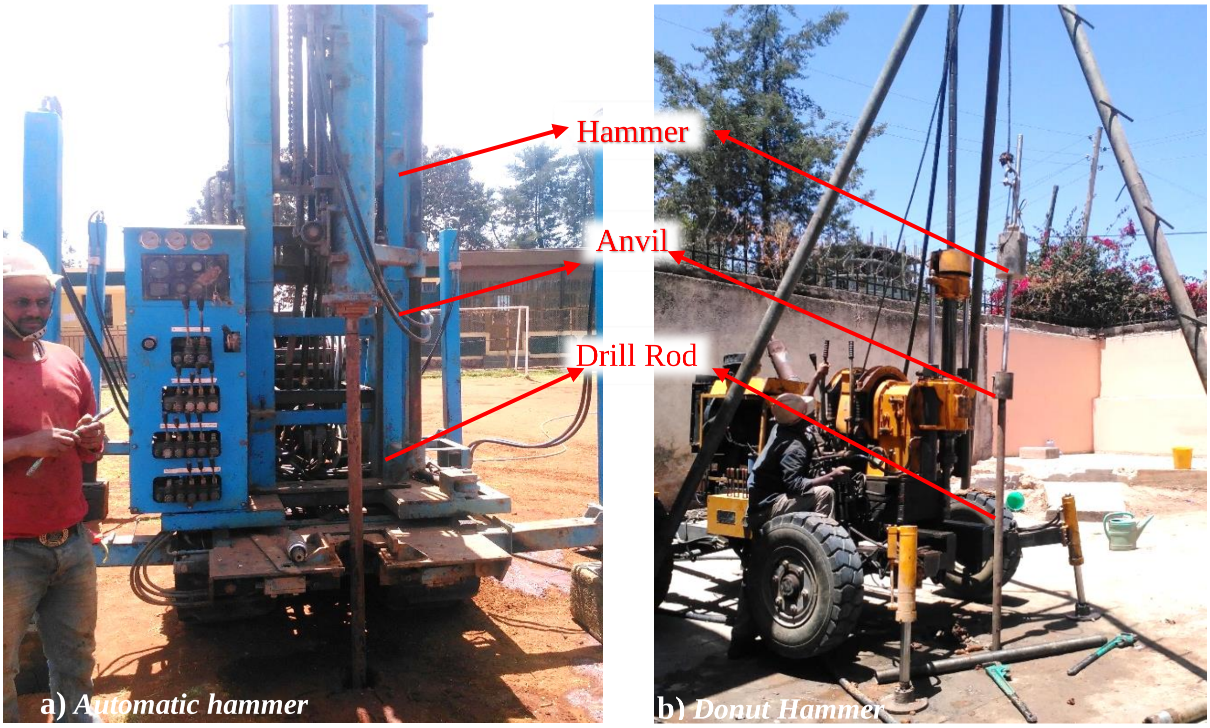

Plate 4.1 Drilling rigs with SPT hammers

In fact in this study, the actual energy delivered to drill rod is not measured and there are no available data on hammer efficiency of either type for local practice in Ethiopia. Thus, the energy ratio, from other countries practice, was adopted considering the drilling rig, hammer type and lifting mechanism. The information regarding the type of drill rigs and hammers used in the field testing program listed in Table 4.1.

The length of drill rod is one of the factors that affects the hammer efficiency, manual cathead hammers are influenced by short rod lengths and rod stiffness that may reduce the energy transferred. When the length of the drill rod is less than 10 m, energy is reflected back in the rod reducing the energy transmitted to the split spoon penetrating the soil (Bowles, 1995).

Table 4.1 Drilling rigs and hammer tested

BH No.	Rig type	Drilling method	Hammer type	Drill stem	Condition of rope	Hammer lifting system
BH01-BH06	Conventional, HXY – 2BT	Rotary core drilling	Donut Hammer	AWJ	Good condition	Cathead and rope system
BH07-BH08	XY- 200 spindle type China rigs	Rotary core drilling	Donut Hammer	AWJ	Good condition	Cathead and rope system
BH09-BH15	Wire line, SH-700	Rotary core drilling	Automatic Hammer	AWJ	-	Trip system
BH16-BH22	SH-100 Korea rigs	Rotary core drilling	Automatic Hammer	AWJ	-	Trip system

The SPT data were measured using donut hammers and automatic hammer systems as presented in table 4.2.

Table 4.2 SPT N-values with the corresponding depth of different hammer type

Donut hammer			
BH ID	**Depth**	**N_f**	**Lithological Description**
BH1	1.5	10.0	Firm, black to dark brown, highly plastic silty CLAY
	3.0	8.0	Firm, black to dark brown, highly plastic silty CLAY
	4.5	7.0	Stiff, reddish brown, highly plastic silty CLAY with some whitish pebbles
	6.0	18.0	
	7.5	50.0	
	9.0	10.0	Hard to stiff, reddish brown to light gray, non-plastic clayey SILT with vesicular basalt gravels
	10.5	7.0	
BH2	2.0	9.0	Dark gray, high plastic silty CLAY
	4.5	8.0	
	6.0	7.0	
	8.0	10.0	
BH3	1.5	7.0	Firm to stiff, light gray, highly plastic silty CLAY
	4.5	10.0	
	6.0	12.0	
	8.0	13.0	
BH4	1.5	6.0	Firm, dark grey, highly plastic silty CLAY
	3.6	7.0	Firm to stiff, light grey, highly plastic silty CLAY
	5.0	10.0	Firm to stiff, light grey, highly plastic silty CLAY
	7.5	16.0	
	9.6	18.0	

BH ID	Depth	N_f	Lithological Description
BH5	1.5	10.0	Firm, dark grey, highly plastic silty CLAY
	3.0	7.0	Firm to stiff, light grey, highly plastic silty CLAY
	5.0	9.0	
	9.0	8.0	
	10.6	7.0	
	14.8	30	
BH6	5.2	12.0	Firm to stiff, light grey, highly plastic silty clay
	7.6	16.0	
	9.8	20.0	
	11.2	20.0	
BH7	4.0	12.0	Stiff, light grey, highly plastic silty CLAY
	6.5	16.0	
	8.0	20.0	
BH8	3	5	Firm, light grey, highly plastic silty clay
	5.5	14	Stiff to very stiff, reddish brown clayey, low plastic clayey SILT
	7	14	
Automatic Hammer			
BH ID	**Depth**	**N_f**	**Lithological Description**
BH9	2.5	14.0	Reddish brown, highly plastic silty CLAY
	3.8	18.0	Reddish brown, stiff, highly plastic, silty CLAY
	5.6	20.0	
	7.0	25.0	
	8.5	11.0	
	11.0	50.0	Reddish brown, Dense, non-plastic Silty SAND
	12.5	50.0	
BH10	1.6	9.0	Stiff to very stiff, black, highly plastic CLAY
	3.0	12.0	Stiff to very stiff, brownish grey clayey, low plastic clayey SILT
	4.6	16.0	
	5.8	27.0	
	7.6	20.0	
	9.0	11.0	
	10.6	12.0	
BH11	1.6	50.0	Firm, brown, highly plastic silty CLAY
	5.1	5.0	Medium stiff to very stiff, reddish brown, low plastic clayey SILT
	6.6	6.0	
	8.1	11.0	
	10	17.0	
BH12	2.0	28.0	Red, stiff to very stiff, highly plastic, silty CLAY
	3.0	27.0	
	5.0	24.0	Reddish brown, moist, stiff, slightly gravelly clayey SILT
	6.5	9.0	
BH13	1.6	4.0	Dark, highly plastic CLAY
	3.0	6.0	
	4.5	9.0	
	6.0	9.0	

BH13	7.5	12.0	Dark, highly plastic CLAY
	9.0	13.0	
BH14	3.0	5.0	Light brownish grey highly plastic Silty CLAY few gravels
	4.5	6.0	
	6.0	8.0	
	8.0	9.0	
BH15	2.0	7.0	Reddish to yellowish brown Clayey SILT
	3.9	13.0	
	6.0	12.0	
	8.3	13.0	
BH16	2.0	7.0	Soft to medium stiff, dark grey, highly plastic silty CLAY
	5.0	6.0	Medium] stiff to stiff, grey to brownish grey, highly plastic silty CLAY/clayey SILT
	6.0	3.0	
	7.5	10.0	
	9.0	13.0	
BH17	2.0	12.0	Reddish brown, highly plastic, silty CLAY
	3.5	7.0	
	5.0	9.0	
	6.5	11.0	
BH18	5.0	15.0	Reddish brown, highly plastic, Clayey SILT
	6.6	17.0	
	8.0	10.0	
	9.6	19.0	
	11.0	19.0	
BH19	3.5	8	Reddish brown, highly plastic, Clayey SILT
	5	9	
	8	11	
BH20	1.5	8.0	Reddish brown, highly plastic, silty CLAY
	3.0	7.0	
	4.5	14.0	
	6.0	15.0	
	7.5	28.0	
	9.0	50.0	
BH21	1.6	5.0	Medium stiff, dark to light grey, highly plastic silty CLAY
	3.0	15.0	
	4.6	7.0	Medium stiff, dark to light grey, highly plastic silty CLAY
	6.0	7.0	Medium stiff, reddish brown, highly plastic CLAYEY SILT with few gravel
	7.6	6.0	
	9.0	7.0	
BH22	1.6	8.0	Medium stiff to very stiff, light brown, highly plastic silty CLAY
	3.0	7.0	
	4.6	10.0	
	6.0	50.0	
	8.0	12.0	Stiff to very stiff, reddish brown clayey SILT
	10.0	8.0	

4.2.2 Split spoon sampler

SPT split spoon sampler design to carry out the standard penetration test as specified in ASTM-D1586 (2008) and BS 1377 (1975) standards. The standard sample barrel is 2-inches wide in outside diameter. It consists of a sampler head, a split-barrel sampling tube, and a driving shoe. The sampler is attached to the bottom of a core barrel and lowered into position at the bottom of the borehole.

The sampling tube design could also affect the SPT blow count. As stated in Schmertmann (1978), the use of a liner in the sampling tube would cause an increase of the blow count as compared to the use of the same sampling tube without a liner. Early split spoon sampler had a constant inner diameter of 35 mm, while the current standard ASTM-D1586 (2008) split spoon has a variable internal diameter to accommodate an internal liner.

The split-spoon samplers used in this study having an average outer diameter of 50.8 mm and internal diameter of 35 mm and a length of 600 mm. All SPTs for this study were performed without split spoon liners. The driving shoes have the same diameter with the tube and its length was 76 mm. The outside diameter at the bottom was 19 mm. Some of the split-spoon had driven shoes that were worn (i.e., shoe tips were not sharpened) (Fig 4.3).

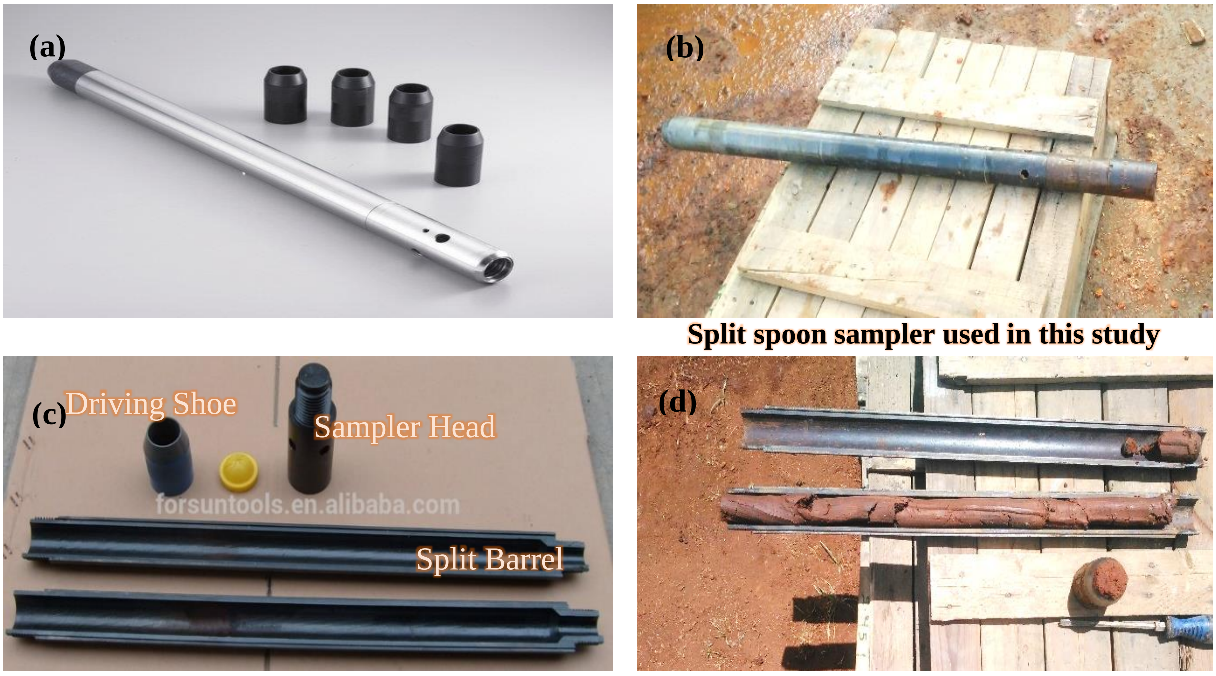

Plate 4.2 (a) & (c) Standard split spoon sampler, (b) & (d) Split spoon sampler used in this study

Plate 4.3 (a) Deformed driving shoe, (b) New driving shoe

4.3 SPT Procedure

SPT test procedure was comprised in four steps, the test hole, assembling equipment, penetration interval and handling samples.

Step 1 - Drill the hole to the desired sampling depth and clean out all disturbed material.

Step 2 - Attach the split-barrel sampler to the drill-rod and lower into the hole until it is reached on the undisturbed material then attached the drive weight assembly and lift the hammer approximately 76 cm and allow it to fall on the anvil delivering one seating blow then mark the drill rod in three successive 15 cm increments to observe penetration. The blow count data was recorded for each of three consecutive 15 cm increments of drive of the sampling spoon sampler.

Step 3 - Raise and drop the hammer by means of the lifting and dropping mechanisms. The hammer should be operated between 40 and 60 blows per minute and should drop freely. The blow rate is depending on the efficiency of the rigs, types of hammer and hammer lifting mechanisms. The recorded rate values of application for automatic hammers is on the range of 24 to 30 blows per minute (b/min) whereas donut hammer is on the range of 15 to 20 blow per minute (b/min). The blows were counted until the sampler has penetrated 45 cm into the soil at the bottom of the borehole or until refusal has occurred. Refusal is defined as the condition when 50 blows have been applied during any one of three 150 mm increments of drive. Finally the number of blows recorded for each 15 cm increment of penetration. However the first 15 cm of drive is rejected because it is considered as a seating drive. Thus the sum of the second and third blows is considered as “standard penetration resistance” or the “N-value”.

Step 4 - Finally the sampler will be lifted to the surface and remove any obvious contamination from the ends or sides and drain excess water. Carefully scrape or slice along one side to expose fresh material and any stratification and also recorded the length, composition, color, stratification and condition of sample. Remove sample and wrap it or seal in a plastic bag to retain moisture.

Plate 4.4 SPT test procedures: (a) drilled at target depth (b) assembled SPT Equipment (c) put marks on the rod three (d)&(e) conducting the test (f) Handling the samples.

4.4 Standardization of SPT N-values

Accurate measurement of the N-value and correcting those values with the appropriate correction factor is extremely important in engineering design. The observed SPT N-values as measured in the field are not reliable due to several factors such as the energy efficiency, split spoon sampler design, diameter of the borehole and rod length and other factors (Aggour and Radding, 2001). Thus, corrections are needed to apply in SPT blow counts in order to achieve a normalized value prior to use.

Correction factors have been proposed by various authors to account the above factors; Bowles (1997), listed five correction factors comprises hammer efficiency, sampler, rod length, and borehole diameter and depth correction. While recent papers (Mostefa and Mohammed, (2013) and McGregor and Duncan (1998)) has been applied seven correction factors includes anvil and blow rate correction. In this research, the following equation is employed to adjust the SPT values.

N_{cor} is calculated using Eq. 4.2. (McGregor and Duncan 1998):

$$N'_{cor} = C_N * N * \eta_1 * \eta_2 * \eta_3 * \eta_4 * \eta_5 * \eta_6 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \textbf{Eq 4.1}$$

Where

N'_{cor} = adjusted N

C_N = adjustment for overburden pressure $(\frac{P''_O}{P'_O})^{1/2}$ p'$_o$ (kPa)

P'_O = overburden pressure

P''_O = reference overburden pressure

η_1 = Energy correction E_r/E_{rb} (where E_r is average energy ratio that depends on the drill system and E_{rb} is the standard energy ratio).

η_2 = Rod length correction

η_3 = Sampler correction

η_4 = Borehole diameter correction

η_5 = Anvil correction

η_6 = Blow count frequency correction

There are several publications recommending that a standard energy ratio (E_{rb}) should be adopted for SPT investigations in to allow reproducible and consistent blow counts. Kovacs (1983) initially suggested that order 55% be adopted as the efficiency at which most drill rig systems operated at the time that empirical correlations were made. Seed (1985), suggested instead that 60% be used since it is associated with the safety hammer whereas, Bowles (1997), has recommended 70% be used (Aggour and Radding, 2001). These estimates are the basis for the proposed correction factors for hammer types. In this study it is adopted 70% as the standard energy ratio since it will greatly minimize field data (N-value) and it is more recent suggestion using current drilling equipment with a donut or an automatic hammer technology. In most cases, local practices were usually employed to estimate the energy efficiency; and the energy ratio often assumed 45% and 60% of donut hammer and automatic hammer, respectively. The values for the above factors used from Bowles (1997), listed in Table 4.5.

$$\text{Energy efficency} = {}^{\mathrm{E_r}}/_{\mathrm{E_{rb}}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{Eq 4.2}$$

where $\mathrm{E_r}$ = mean energy ratio = $\mathrm{E_{rb}}$ = standard energy ratio

The SPT is considered to be standard to the energy ratio E_r = 70%.

$$\text{Where, } E_r = \frac{\text{actual hammer energy to sampler, } \mathrm{E_a}}{\text{input energy, } \mathrm{E_{in}}} X\ 100$$

$$\text{Energy efficency } = {}^{45}/_{70} * 100 = 0.64\ (\mathit{Donut\ hammer})$$
$$= {}^{60}/_{70} * 100 = 0.84\ (\mathit{Automatic\ hammer})$$

Since the drilling rigs considered in this research are equipped with a donut and automatic hammer; the hammer correction factors of 0.64 and 0.84 are used, respectively. When the length of the drill rod is less than 3m, a considerable amount of energy is reflected back in the rod reducing the energy available for driving the sampling tube into the ground, thus it is recommended that the N_f values should be corrected for short lengths of rods. The rod length correction (η_2) was performed according to Table 4.3.

SPT N_f values are corrected if they are made in boreholes having diameter larger than 114mm.When boreholes diameter are larger than 114mm, measured N_f values are lower than they would be for a smaller diameter hole. All boreholes had a diameter between 60 - 120 mm; thus, the borehole diameter correction is 1.0. The split spoon had a constant diameter of 35 mm (without liner); thus, the split spoon configuration correction is 1.0.

When the hammer falls during the SPT testing, it strikes an anvil attached to the drill rod stem. The anvil can vary in shape, size and weight. The amount of energy transferred to the drill rods depends on the weight of the anvil. Therefore the values used for correction for donut and automatic hammer are 0.70 and 0.90 respectively. The last correction is blow count frequency this depending on hammer type and rig type. As mentioned previously the recorded rate values of application for automatic hammers is on the range of 24 to 30 blows per minute (b/min) whereas donut hammer is on the range of 15 to 20 blow per minute (b/min). Thus the correction factor that was used for both types of hammer was taken 1.05 (Table 4.4).

Table 4.3 SPT field correction factors (Bowles, 1997)

Hammer for η_1= Er/70				
Average Energy Ratio, Er				
	Donut		Safety	
Country	Rope-Pulley	Trip	Rope-Pulley	Trip/Auto
United States	**45**	-	70-80	80-100
Japan	67	78	-	-
United Kingdom	-	-	50	**60**
China	50	60	-	-
Rod Length Correction, η_2				
Length >10m	η_2 = 1.00			
6-10m	= 0.95			
4-6m	= 0.85			
0-4m	= 0.75			
Sampler Correction, η_3				
Without liner	**η_3=1.00**			
With liner: Dense sand, clay	= 0.80			
Loose sand	= 0.90			
Borehole Diameter Correction, η_4				
Hole diameter:	60-120mm	**η_4=1.00**		
	150mm	=1.05		
	200mm	=1.15		

Table 4.4 Anvil and blow rate correction (Seed (1984) per McGregor and Duncan (1998)).

Anvil Correction, η_5	
Small **Large** **Safety**	0.85 0.7 0.9
Corrections for Blow Rate (CBF), η_6	
>20 **<20**	10-20b/m - 0.95 10-20 b/m - 1.05

The final adjustment of measured SPT (N-values) is presented in the table 4.5 by applying various corrections as mention earlier.

Table 4.5 Adjusted measured SPT N-values

Hammer Type	BH ID	Test point (m)	Depth of Stratum (m)	N_f	γ (kN/m^3)	σ'_v (kN/m^2)	C_N	$\eta_1 = Er/E_{70}$	$\eta_1 = Er/E_{60}$	η_2	η_3	η_4	η_5	η_6	N_{70}	N_{60}
Donut	BH-1	6.00	6.00	18.00	14.30	85.80	1.00	0.64	0.75	0.95	1.00	1.00	0.70	1.05	**8**	**9**
	BH-2	4.50	4.50	14.00	20.10	90.45	1.00	0.64	0.75	0.85	1.00	1.00	0.70	1.05	**5**	**7**
	BH-3	4.50	4.50	10.00	20.80	93.60	1.00	0.64	0.75	0.85	1.00	1.00	0.70	1.05	**4**	**5**
	BH-4	3.00	3.00	7.00	18.00	54.00	1.00	0.64	0.75	0.75	1.00	1.00	0.70	1.05	**2**	**3**
	BH-5	10.00	10.00	30.00	18.00	180.73	0.73	0.64	0.75	1.00	1.00	1.00	0.70	1.05	**10**	**12**
	BH-6	7.60	7.60	16.00	18.24	138.62	0.83	0.64	0.75	0.95	1.00	1.00	0.70	1.05	**5**	**7**
	BH-7	7.70	7.70	21.00	18.00	138.60	0.83	0.64	0.75	0.95	1.00	1.00	0.70	1.05	**7**	**9**
	BH-8	5.50	5.50	14.00	19.00	12.05	1.00	0.64	0.75	0.85	1.00	1.00	0.70	1.05	**5**	**7**
Automatic	BH-9	7.00	7.00	25.00	17.98	125.86	0.87	0.86	1.00	0.95	1.00	1.00	0.70	1.05	**13**	**15**
	BH-10	5.90	5.90	14.00	17.57	103.65	0.96	0.86	1.00	0.85	1.00	1.00	0.70	1.05	**7**	**8**
	BH-11	5.00	5.00	5.00	17.44	87.20	1.00	0.86	1.00	0.85	1.00	1.00	0.90	1.05	**3**	**4**
	BH-12	3.50	3.50	27.00	21.58	75.53	1.00	0.86	1.00	0.75	1.00	1.00	0.90	1.05	**16**	**19**
	BH-13	4.50	4.50	9.00	17.25	77.63	1.00	0.86	1.00	0.85	1.00	1.00	0.90	1.05	**6**	**7**
	BH-14	3.50	3.50	5.00	17.47	61.13	1.00	0.86	1.00	0.75	1.00	1.00	0.90	1.05	**3**	**4**
	BH-15	4.50	4.50	13.00	16.66	74.97	1.00	0.86	1.00	0.85	1.00	1.00	0.90	1.05	**8**	**10**
	BH-16	4.50	4.50	6.00	18.28	82.26	1.00	0.86	1.00	0.85	1.00	1.00	0.90	1.05	**4**	**5**
	BH-17	3.50	3.50	7.00	17.82	62.37	1.00	0.86	1.00	0.75	1.00	1.00	0.90	1.05	**4**	**5**
	BH-18	6.50	6.50	17.00	19.10	124.17	0.88	0.86	1.00	0.95	1.00	1.00	0.90	1.05	**11**	**13**
	BH-19	3.50	3.50	8.00	15.82	55.37	1.00	0.86	1.00	0.75	1.00	1.00	0.90	1.05	**4**	**6**
	BH-20	2.90	2.90	7.00	17.00	49.30	1.00	0.86	1.00	0.75	1.00	1.00	0.90	1.05	**4**	**5**
	BH-21	4.55	4.55	7.00	18.76	85.36	1.00	0.86	1.00	0.85	1.00	1.00	0.90	1.05	**4**	**6**
	BH-22	4.5	4.5	12	17.83	80.24	1.00	0.86	1.00	0.85	1.00	1.00	0.90	1.05	**8**	**10**

4.5 Laboratory and Field Test Analysis

The field and laboratory tests have been used for determination engineering and physical properties of the fine grained soils. These tests include the unconfined compressive test, Atterberg limits, moisture content and standard penetration test. Most problems in construction involve either the strength of the in-situ soil or the compressibility of the soil.

In this study a total of twenty two undisturbed samples were taken (one per borehole) for conducted unconfined compressive test (UC) and disturbed samples for classification tests. The primary purpose of UC test is to determine the unconfined compressive strength (qu), which is then used to calculate the unconsolidated undrained shear strength of the clay under unconfined conditions. According to the ASTM D 2166 (2004), the unconfined compressive strength 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, whichever occurs first during the performance of a test.

In fine grained soils the penetration resistance is broadly a function of undrained shear strength. The undrained shear strength of soils is necessary for the determination of the bearing capacity of foundations and other civil structures.

C_u of a cohesive soil is equal to one-half the unconfined compressive strength (q_u) when the soil is under the f = 0 condition (f = the angle of internal friction). This is expressed as: using for determine unconfined compressive strength (q_u) and with full saturation of sample, C_u can be obtained from the Eq. 4.3 (Hara et al, 1974):

$C_u = 0.5q_u$ …...…………………………………………………………………….Eq 4.3

The test was performed according to ASTM D2166 (2004), the test were carried out on shelby tube samples recovered from the borings as essentially unaltered moisture content and especial attention was given during transportation and testing procedures in order to avoid inappropriate result.

On the other hand seive analysis, atterberg limits and moisture content were carried out from disturbed samples. Plasticity index (PI) and liquid limit (LL) can be used to determine the swelling characteristics of fine grain soil. Table 4.6 summarizes the representative SPT N-value (the test conducted while the undisturbed samples were taken) with engineering properties of fine grain soils.

Table 4.6 Geotechnical properties of collected soils

Soil Type	BH ID	Test Depth	N_f	Seive	LL	PL	PI	Y	W_n	q_u	C_u
Dark to light CLAY	BH-2	5	14	99	78	35	43	20	30	130	65
	BH-3	5	10	99	71	30	41	21	38	72	36
	BH-4	3	7	95	78	42	36	18	34	38	19
	BH-7	8	21	98	89	43	46	18	38	68	34
	BH-13	5	9	92	77	41	36	17	29	106	53
	BH-14	4	5	97	95	50	45	17	42	89	44
	BH-21	5	7	96	64	39	25	19	40	81	41
	BH-22	5	12	96	74	34	40	18	40	85	43
Red CLAY	BH-1	6	18	86	86	41	45	14	30	218	109
	BH-5	10	30	86	50	34	16	18	37	271	136
	BH-9	7	25	94	56	29	27	18	35	220	110
	BH-12	4	27	90	62	32	30	22	33	266	133
	BH-16	5	6	98	90	50	40	18	41	59	29
	BH-17	4	7	85	92	43	49	18	30	79	39
	BH-20	3	7	92	89	43	46	17	45	150	75
Red SILT	BH-6	8	16	95	92	43	49	18	38	65	33
	BH-8	6	14	95	57	29	28	19	35	176	88
	BH-10	6	14	92	40	28	12	18	36	199	99
	BH-11	5	5	93	60	29	31	17	49	68	34
	BH-15	5	13	98	68	35	33	17	41	83	41
	BH-18	7	17	92	56	32	24	19	33	143	71
	BH-19	4	8	85	52	33	19	16	31	105	53

As mentioned in the previous chapter, the type of fine grained soils that are encountered in the study area are red clay, dark clay and red silt soils. The soils' consistency is considered from the results of SPT blow count, based on the Terzaghi and Peck, (1967) and Bowles (1997) correlation.

Table 4.7 Consistency of fine grain soil

N (Terzaghi and Peck, 1967)	N (Bowles 1997)	Consistency	q_u(kN/m^2)
0-2	0-2	Very soft	<25
2-4	3-5	Soft	25-50
4-8	6-9	Firm	50-100
8-15	10-16	Stiff	100-200
15-30	17-30	Very stiff	200-400
>30	>30	Hard	>400

Based on field and laboratory test result, the fine grained soils are described below;

Accordingly, the dark grey soils recorded from 5 to 21 blows per 30 cm penetration and the corresponding C_u values yielded on the range of 19 kPa to 65 kPa; which implied the soil is firm to very stiff. Plasticity index and liquid limit values ranging from 25% - 46% and 64% - 95% respectively. It implies that the dark grey soil is medium to highly plastic. The percentage of the material finer than the 0.075 mm sieve ranges from 86% - 95%.

The red clay soils recorded from 6 to 30 blows per 30 cm penetration and the corresponding C_u values yielded on the range of 29 kPa to 136 kPa; which implied the soil is firm to very stiff. Plasticity index and liquid limit values ranging from 16% - 45% and 50% - 86%, respectively. It implies that the red clay soil is medium to highly plastic. The percentage of the material finer than the 0.075 mm sieve ranges from 81% - 99%.

The red silt soils recorded from 5 to 30 blows per 30 cm penetration and the corresponding C_u values yielded on the range of 33 kPa to 99 kPa; which implied the soil is firm to very stiff. Plasticity index and liquid limit values ranging from 12% - 52% and 28% - 92%, respectively. It implies that the red silt soil is low to highly plastic. The percentage of the material finer than the 0.075 mm sieve ranges from 77% - 90%.

4.6 Overall SPT Measurement Results

The tests were carried out in fourteen boreholes by automatic hammer and eight boreholes were tested by donut hammer. A total of 100 SPT blow counts at the twenty two sites were used for this research. N-values with the corresponding depth for each borehole of both hammer types are shown in figure 4.2.

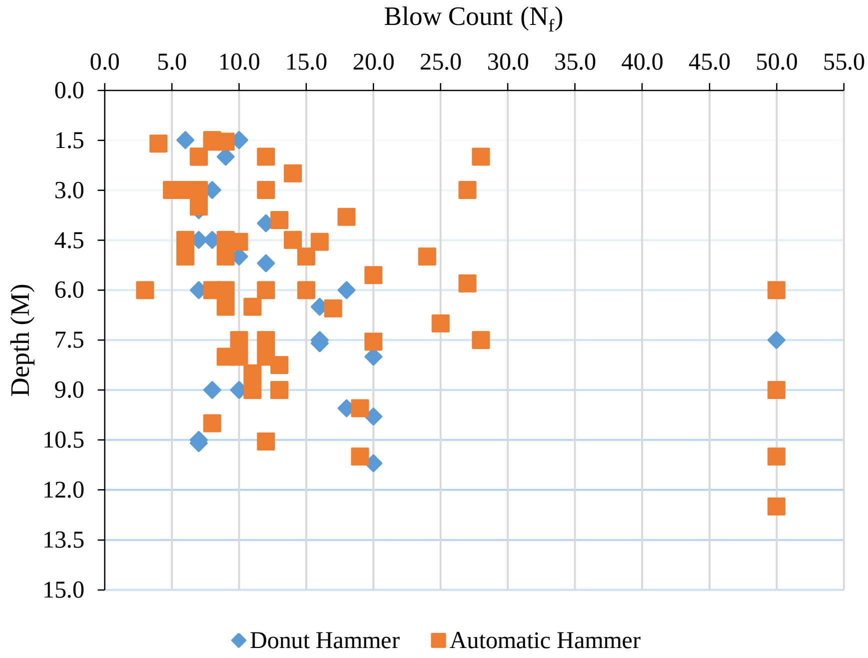

Fig. 4.2 SPT N-values distribution with depth

Fig. 4.2 showed that N-value distribution with the corresponding depth. Usually the consistency of soil increase with depth due to overburden pressure. The test is conducted with both donut and automatic hammer system. in both case, some values that got deep have low N-values and record high N-values near surface. This observation provides factual information that even though the SPT is a standardized test, the diversity of equipment (hammer system, rig type etc...) allowed to perform SPT can have a significant influence on the resulting SPT N-value.

Table 4.8 Summarized geotechnical properties of studied soil

Variable	Mean	SD.	S.Variance	Range	Min.	Max.	Sum	Count
N_f	13.27	7.29	53.16	25.00	5.00	30.00	292.00	22
N_{70}	6.32	3.46	11.94	14.00	2.00	16.00	139.00	22
W_n	35.94	5.14	26.40	20.45	28.55	49.00	790.59	22
PI	33.74	11.78	138.88	45.36	11.64	57.00	742.34	22
LL	67.18	17.67	312.06	67.00	28.00	95.00	1478.00	22
PL	36.08	6.51	42.39	21.64	28.36	50.00	793.66	22

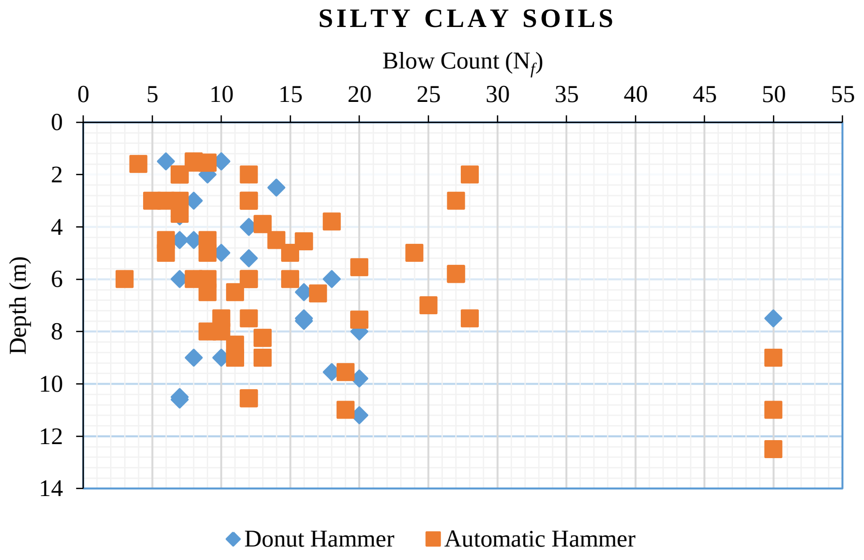

Fig. 4.3 SPT N-values with depth in silty clay soils

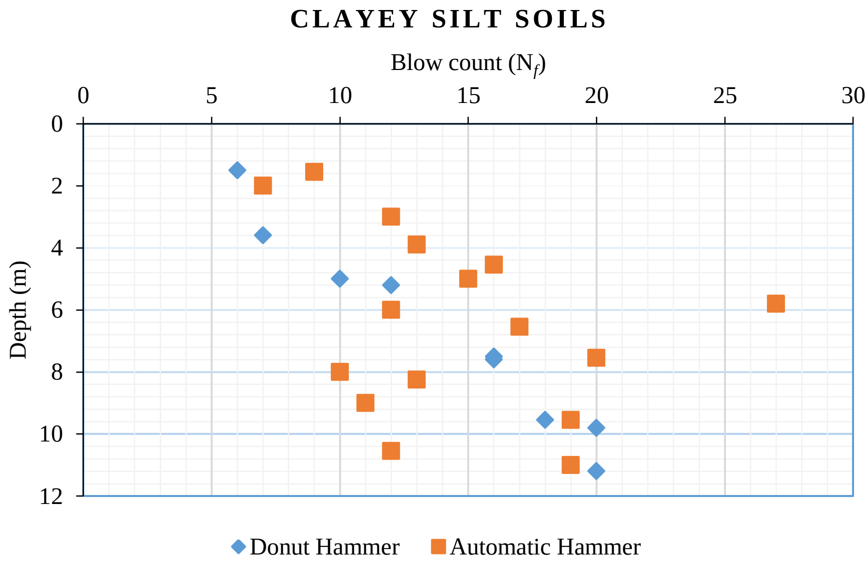

Fig. 4.4 SPT N-values with depth in clayey silt soils

As illustrated in the figures 4.3 & 4.4 the tests were conducted by donut and automatic hammer on both silty clay and clayey silt soils. The test that conducted in clay soil have different value the even count in the same depth and also recorded high N-value in the clay soils around 50, this is due to the data collected in dry season so it may be increase the stiffness of the soil. The consistency of the soil was not increase with depth whereas The N-values of silt soil recorded by both types of hammer were relatively increased with depth than clay soils. This indicated that SPT N-value high affected by clay soil than silt soil and the soil type also one of the factor affecting the N-value.

As mentioned earlier in this study, SPT measurements were carried out in twenty two boreholes with donut and automatic hammers. Researchers were indicated the result of a standard penetration test is influenced by drilling technique, SPT test equipment and test procedures. In this research work further, possible source of errors due to faulty equipment, procedure and other are also considered as follows.

- Used damage and old cutting shoes
  - The main advantage of cutting shoe is cutting and penetrating the soil while conducting the test. Thus it should be sharp and new to give an appropriate result. When the test is conducted in damage shoe the N-values result may increase and overestimate the strength of foundation material. In this study it has been observed some of the tests conducted in damaged and old cutting shoes; which caused the N-values increase the blow counts.

- Lack of hammer free fall
  - Due to the lack of free fall the hammer energy doesn't reach at the bottom of the testing depth. This is because of ungreased sheaves and as a result its increase SPT N-values. In this study also observed such type of problem in donut hammers due to the dropping/lifting mechanism (cathead and rope).

- Rusty cathead and low-speed cathead
  - During carried out the test, it was noted that majority of the donut hammers have rusty cathead and use low speed cathead. Such problems lead to decrease the hammer energy delivered to split spoon sampler tips. Consequently it increases the N-value.

- Hammer weight inaccuracy
  - In this study such type of errors was observed in donut hammer type. The hammer weight has above the standard weight given by ASTMD1586 (2008). This may decrease the N-value result.

- Presence of gravels and pebbles

- In this research, the presence of pebbles and gravels in fine grain soil were observed to affect the SPT measurement. It is also overestimating the consistency of the soil by increasing the blow counts.

- Stiffness of the soil
    - The consistency of soil is depends on the presence of water table or not around the foundation material. Hence, the present study was carried out in dry season, most of the borehole were dried. Some have a perched aquifer the N-value is very low.

Generally, SPT measurement helps try to identify the possible source of errors while conducting the test especially from SPT equipment like hammer types, hammer weight, split spoon sampler design and etc…The main source of errors that identified in this studies are, used damage and old cutting shoes, hammer weight inaccuracy, rusty cathead and low-speed cathead, presence of gravels and pebbles and stiffness of the soil. Most the identified errors may overestimate the strength of the soil by increasing the result of N-value. Others are underestimate the strength of soil by decreasing the N-values. Usually N-value increase when getting depth due to overburden pressure but as shown in depth distribution of N-value it's not observed such type of relationship due to the above reasons.

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CHAPTER 5 ANALYSIS AND INTERPRETATION

5.1 General

Despite the associated defect related to its less reproducibility, geotechnical engineers have established correlations with the SPT N-values for most of the soil geotechnical properties. It is known that there are well-established correlations, charts and empirical equations in relation to the physical properties, in-situ state of stress, strength, and stiffness of the cohesive soil to that of the SPT N-values. Though, it is quite unreliable to draw correlations for in-situ shear strength of fine grained soils from SPT N-values. There are also few attempts like penetration resistance in cohesive soil is broadly a function of undrained shear strength. C_u of a cohesive soil is equal twelve times of N-value (Terzaghi and peck, 1967), Schmertmann (1979) try to investigate the derivation of soil resistance in clay soil, at least 70% of the soil resistance can be derived from side shear, the remainder coming from end bearing capacity and Stroud (1979) indicated that undrained shear strength is four times of N-value.

Recent studies indicated that undrained shear strength of fine grain soil calculated from N-value with atterberg limit and moisture content shows better empirical correlation. In this chapter, then the correlations drawn from the SPT N-value to different geotechnical properties of soils will be discussed and will be further compared to the results of this research.

In correlation, it is known that it will be important to consider the correlation coefficient (R^2), and the best fitting plots between the results. According to Taylor (1990, as cited in Mostefa and Mohammad, 2013) the purpose of using this statistical method is to give us a statistic known as the correlation coefficient which is a summary value of a large set of data representing the degree of linear association between two measured variables. R^2 is a statistic that will give some information about the best fit of a relationship. In regression, the R^2 coefficient of determination is a statistical measure of how well the regression line approximates the real data points. The values of R^2, the relationship between any two parameters can be classified as presented in table 5.1.

Table 5.1 The values of correlation coefficient (Taylor, 1990)

R^2	Relationship
< 0.30	No correlation
0.30 – 0.499	Mild/slightly relationship
0.5 – 0.699	Moderate relationship
0.70 – 1.0	Strong relationship

Many graphs have been plotted to analyze and illustrate the relationships between N-values with depth, atterberg limits, moisture content and undrained shear strength. The relationship between these mentioned parameters have been presented as follows:

5.2 Depth Distribution of SPT N-value

Usually N-values increase with depth due to overburden pressure even though the SPT is a standardized test, the diversity of equipment and other source of errors allowed to perform SPT can have a significant influence on the resulting SPT N-value. SPT measurements were conducted in the borehole at depth where fine grained soils were encountered. Some local decreases with depth in the counts were also recorded at some of the boreholes, which is implying that the N-values affected by various factors that investigated during conducting the test includes; damage and old drive shoes, lack of hammer free fall, rusty cathead and low-speed cathead, hammer weight inaccuracy, presence of gravels and pebbles and stiffness of a soil.

5.3 SPT N-values with Atterberg Limit and Moisture Content

According to the procedures of SPT, it can be found that the test has carried out in the field on soil as it is (undisturbed condition). Whereas, the procedures of these laboratory tests, the Atterberg limits and moisture content of soil depend mainly on the physical and mechanical properties of soil particles and are carried out on disturbed samples. The relation between SPT N-value with plastic index and moisture content of soil was investigated by some researcher and the result was indicated atterberg limits and moisture content no significant effect on the SPT N-value.

Figure 5.1 shows the relationship between corrected N-value with LL, PL, PI, and W_n respectively. These variables were plotted with N_{70} to develop new correlations specific to fine grained soils.

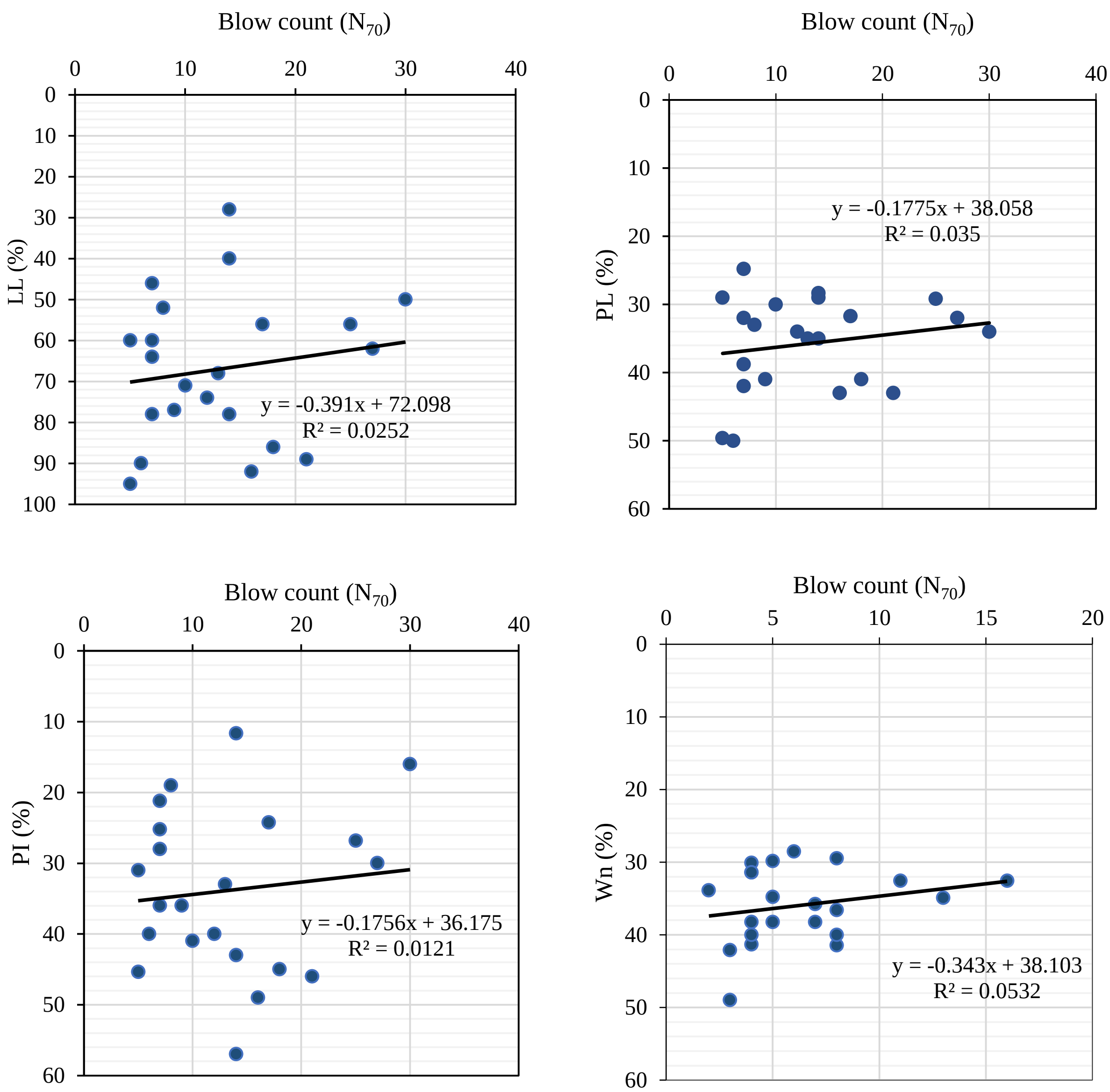


Fig. 5.1 Relation between N-values with Atterberg limits and moisture content

As shown in the above plots, the correlation coefficient of all variables (PI, LL, PL and W_n) falls in between 0.01 and 0.05. Also, considering the values of correlation coefficient ($R^2 < 0.30$), this indicate the results did not show any correlation. Normally moisture content is related with N-value. In this case the N-value collected in dry season. This may be due to SPT is conducted in the field insitu soil; however Atterberg and moisture content are conducted in the laboratory in disturbed soil. From these plots, it can be noticed that the atterberg limits and moisture content have no effect on SPT number (N_{70}).

5.4 SPT N-value and Undrained Shear Strength Correlation

Standard penetration test primary gives idea about shear strength of soil which it can be expressed in terms of shear strength parameters (cohesion & angle of internal friction). In fine grain soils, SPT N-values are usually used to correlate with undrained shear strengths.

This section examines the relationship between SPT N-value and undrained shear strength from a statistical point of view, taking account of test types and SPT corrections. In correlating shear strength to SPT N-value the major problem to the fact that undisturbed samples are recovered, N-values are typically not obtained. As such, some amount of interpolation is required to arrive at a relatively accurate estimate of N-value for a particular sample depth.

As stated in the previous chapter, a total of twenty two UCS test were conducted from undisturbed soil samples, from these tests undrained shear strength were calculated. The collected data have been presented in the following plots are divided in two, N_f vs C_u and N_{cor} vs C_u. with each group the data has been divided in to different hammer and soil type.

A relationship between measured N-values and undrained shear strength is presented in the following charts.

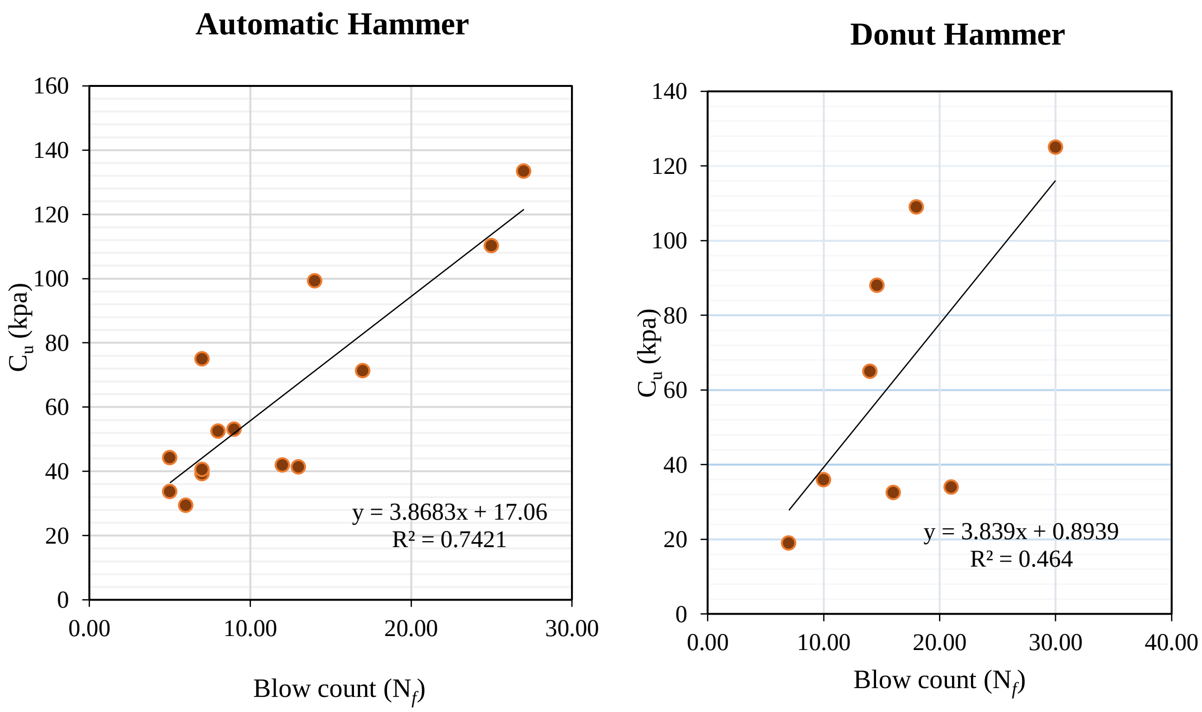


Fig. 5.2 Field N-value versus undrained shear strength

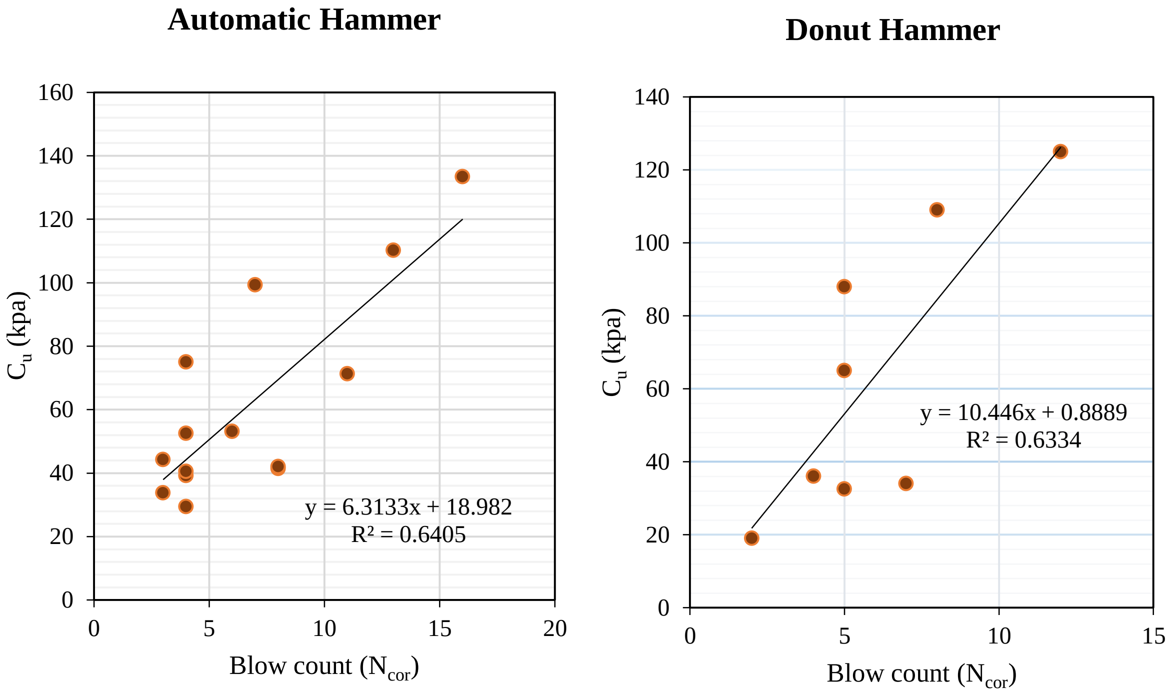


Fig. 5.3 Corrected SPT number versus undrained shear strength

The above plots shows the relationship between SPT N-values and undrained shear strength of fine grain soil with automatic and donut hammer types. In Fig. 5.2 the automatic hammer has a high correlation coefficient (R^2 = 0.74), this can be an indication of a strong relationship whereas the correlation coefficient of donut hammer test (R^2 = 0.46) it implies that the test has a mild relationship. This indicates that it is a fair correlation between N_f and C_u. Also, Fig. 5.3, the correlation coefficient of an automatic hammer is 0.64 and 0.63 for donut hammer. The relationships between N_{cor} and C_u in both cases are considered to a moderate relationship.

Mostly the donut hammer types the correlation is increases after applying the correction factors this implies hammer types highly affected the result of N-value compare to an automatic hammer. Generally this leads to conclude that the undrained shear strength of fine grain soil's estimation from SPT is moderately affected by the hammer type.

It can be noticed that N-value increases as undrained shear strength of fine grain soil increase. The increasing of shear strength parameters of soil is due to increasing in consistency which leads to increasing in N-value. Therefore SPT can predict the strength of fine grain soil in terms of undrained shear strength.

5.5 Data Analysis

The correlations were developed using simple linear regression and multi linear regression analysis. Linear regression analysis is trying to establish linear correlations between undrained shear strength and N-value, PI, LL, PL & W_n. In simple linear regression analysis trying to establish linear correlations between SPT N-value and C_u using the least square method whereas in multi linear regression analysis establish linear correlations in addition to N-value, LL, PL, PI and W_n were included. This analysis is developed by linear regression excel software (see Table 5.2).

Table 5.2 Summary Output regression analysis

Regression Statistics				
	N_{70}	*N_f*		
Multiple R	0.7816	0.776549		
R Square	0.610898	0.603029		
Adjusted R Square	0.591443	0.58318		
Standard Error	21.68659	21.896		
Observations	22	22		
	Coefficients	*Standard Error*	*Lower 95.0%*	*Upper 95.0%*
Intercept	13.65212	9.700173	-6.58208	33.88633
N_{70}	7.403143	1.321139	4.647296	10.15899
	Coefficients	*Standard Error*	*Lower 95.0%*	*Upper 95.0%*
Intercept	13.52698	9.871608	-7.06483	34.11879
N_f	3.612156	0.655333	2.245155	4.979156

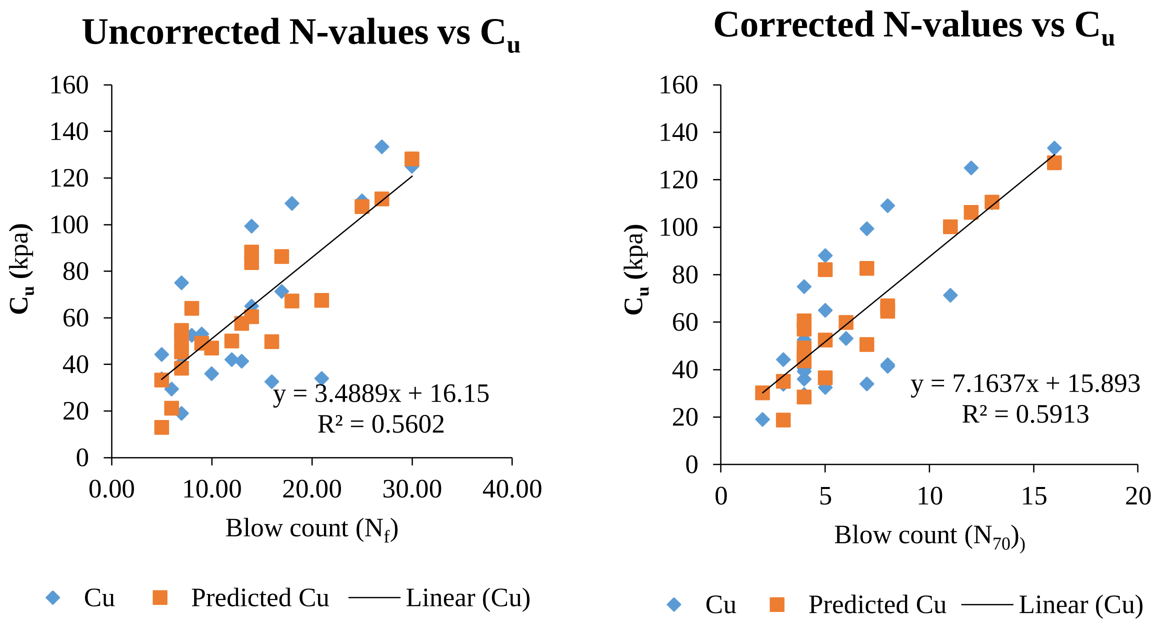


Fig. 5.4 N-values versus undrained shear strength

As shown in Fig 5.5 the correlation coefficient of N_f and N_{70} are (R^2 = 0.56 and 0.59) respectively. This can be an indication of a good correlation between the two correlated parameters. This means the relationships between N-value and C_u are considered to a moderately relationships. So, the undrained shear strength can be fairly predicted from corrected N-values.

Through the linear regression method, the following relationships have been obtained using SPT N-values; the accuracy of the correlation is quantified using the statistical characteristics (n, r^2 and S.D.).

$C_u = 3.49N_f + 15.92$ (n=22, R^2=0.56, S.D= 0.12) ………………………………………...Eq 5.1

$C_u = 7.16N_{70} + 15.89$ (n=22, R^2=0.59, S.D= 0.04) ……………………………………….Eq 5.2

In multi linear regression analysis in addition to N-value; W_n, LL, PL and PI are considered as independent parameters (see Table 5.3 and Fig. 5.7).

Table 5.3 Summary Output regression analysis

Regression Statistics	N_{70}	N_f
Multiple R	0.845381	0.844921
R Square	0.722988	0.713892
Adjusted R Square	0.645445	0.644573
Standard Error	20.23684	20.20464
Observations	22	22

	Coefficients	*Standard Error*	*Lower 95%*	*Upper 95%*
Intercept	95.75	41.21	8.80	182.71
N_{70}	6.12	1.32	3.33	8.91
PL	0.06	0.45	-0.88	1.01
LL	-0.66	0.30	-1.28	-0.04
W_n	-0.85	0.86	-2.67	0.97

	Coefficients	*Standard Error*	*Lower 95%*	*Upper 95%*
Intercept	101.57	40.46	16.22	186.93
N_f	3.00	0.65	1.64	4.37
PL	-0.28	0.44	-1.21	0.65
LL	-0.62	0.30	-1.24	0.01
W_n	-0.76	0.87	-2.59	1.06

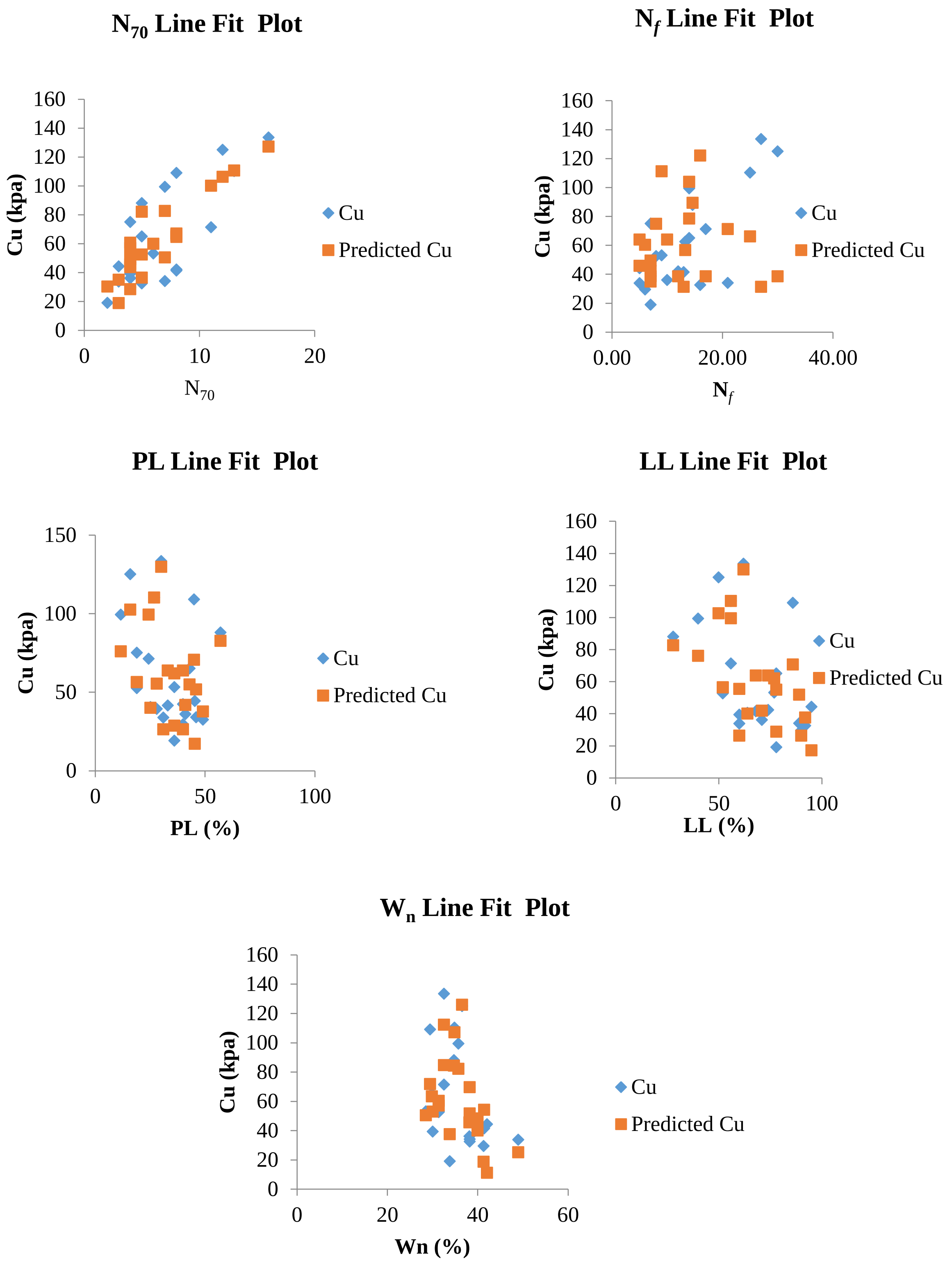


Fig. 5.5 Undrained shear strength versus PI, LL, PL& W_n

From the above analysis the following empirical equation is proposed for both field and corrected N-values.

$C_u = 3.00N_f - 0.76W_L - 0.28P_L - 0.62L_L + 101.57$ (n = 22, $R^2 = 0.71$) ……..……..Eq 5.3

$C_u = 6.12N_{70} - 0.85W_L + 0.06P_L - 0.66L_L + 95.75$ (n = 22, $R^2 = 0.72$)…………....Eq 5.4

All empirical correlation with regression analysis of data revealed that relationships in the second correlation have better than first. The first correlation coefficient: $R^2 = 0.56$ and $R^2 = 0.59$ for correlations based on N_f and N_{70} respectively, the second: $R^2 = 0.71$ and $R^2 = 0.72$ for correlations based on N_f and N_{70} respectively.

5.6 Comparison

To evaluate the capabilities of present proposed correlations, predicted C_u values versus measured C_u values were plotted, compared with existing studies and also comparison was made between SPT and bearing capacity equation by calculating the bearing capacity.

5.6.1 Comparison with predicted C_u with measured C_u value

As it is also noted in the previous analysis, the SPT N-values have moderately relationship with undrained shear strength, which can be reasonably applied to predict strength parameters from field measurements using corrected SPT number (N_{cor}). (Fig. 5.7 and 5.8).

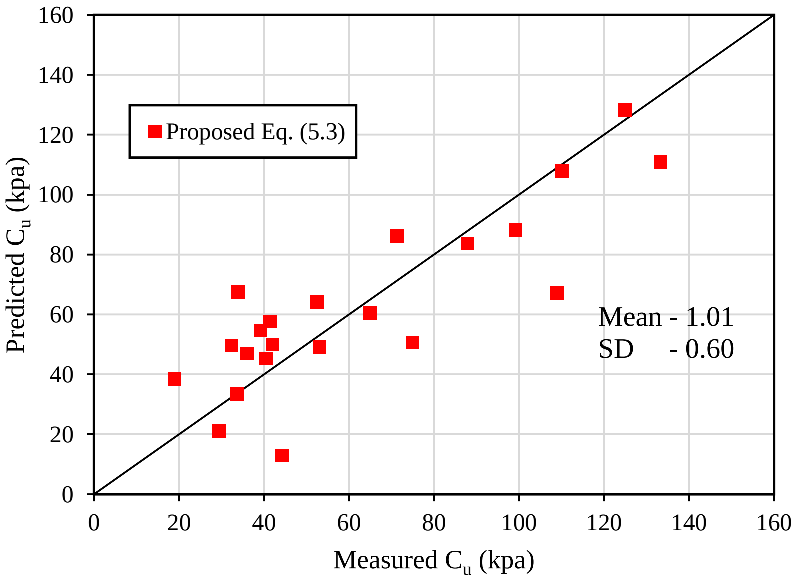


Fig. 5.6 Comparison of Predicted C_u from field N-value vs measured C_u from N_f

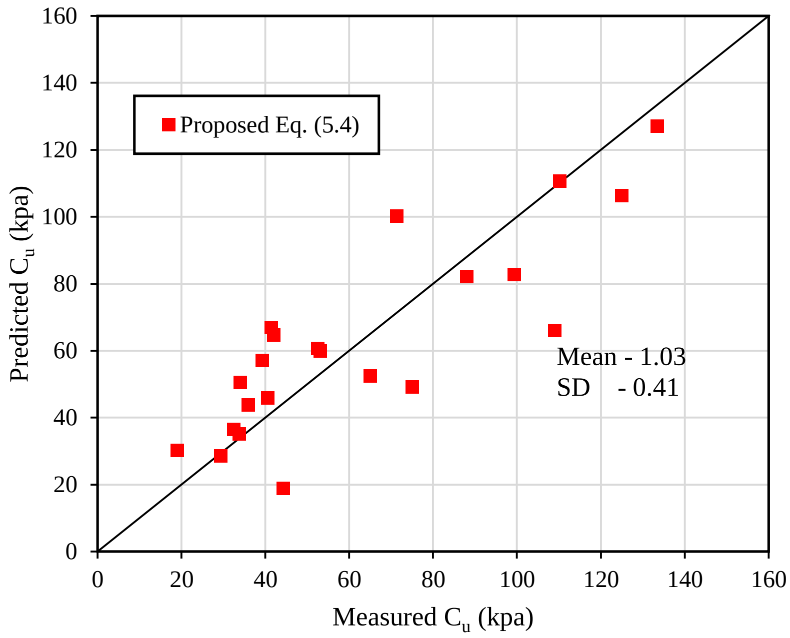


Fig. 5.7 Comparison of predicted C_u with measured C_u from N_{70}

The comparisons between the predicted undrained shear strength values using the proposed procedure and the measured undrained shear strength values are shown in the above plot. This comparison was evaluated by the statistical parameters of mean and standard deviation. These values were calculated from the ratio of predicated vs measured C_u values. In Fig 5.7 the comparison was made using proposed eq. 5.3, which is based on uncorrected SPT values. The values of Mean and SD are 1.01 and 0.60 respectively. From this result indicated it's the predicted value from field N-value and measured values get close to the fit line with minor deviation.

In Fig 5.8 the comparison was made using proposed eq. 5.4, which is based on corrected SPT values. The Mean and SD values of this plot are 1.03 and 0.40 respectively. it was showed that ratio of predicted value from corrected N-value to measured C_u values get close to the fit line with minor deviation.

From the above graphs, the comparison was made using corrected SPT N-values have less deviate from the best fit line than comparison was made using uncorrected SPT N-values. Thus it is showed the corrected N-values are predicated the shear strength than field N-values. Generally result indicated that the proposed equation is reasonably predicted the undrained shear strength.

5.6.2 Comparison with Existing Correlation

As mentioned earlier several researchers have been suggested an approximation for the undrained shear strength which can be obtained from a standard penetration test. In this study correlations were established between the two parameters obtained from a research oriented site investigation. Summarized empirical correlation of C_u versus N-values suggested by different researchers are presented in Table 5.4.

Table 5.4 Existing empirical equations estimating C_u of fine grain soils

Terzaghi & Peck (1967)	6.25N
Hara et al. (1974)	$29N^{0.72}$
Ajayi& Ballogun (1988)	1.39N+74.2
Sivikaya &togrol (2002)	4.32N $6.18N_{60}$
Stroud (1974)	4.2N
Hettiarachichi & Brown (2009)	$4.1N_{60}$
Sirvikaya (2009)	$C_u = 2.41N - 0.82Wn + 0.14LL + 1.44PI$ $C_u = 3.24N_{60} – 0.53Wn – 0.43LL + 2.14PI$
Frazad & Behzad (2011)	$C_u = 1.5N – 0.1Wn – 0.9LL + 2.4PI +21.1$ $C_u = 2N_{60} – 0.4Wn – 1.1LL + 2.4PI +33.3$

As shown in the table 5.4 the existing correlations uses the uncorrected SPT blow count and corrected N-values (N_{60}).

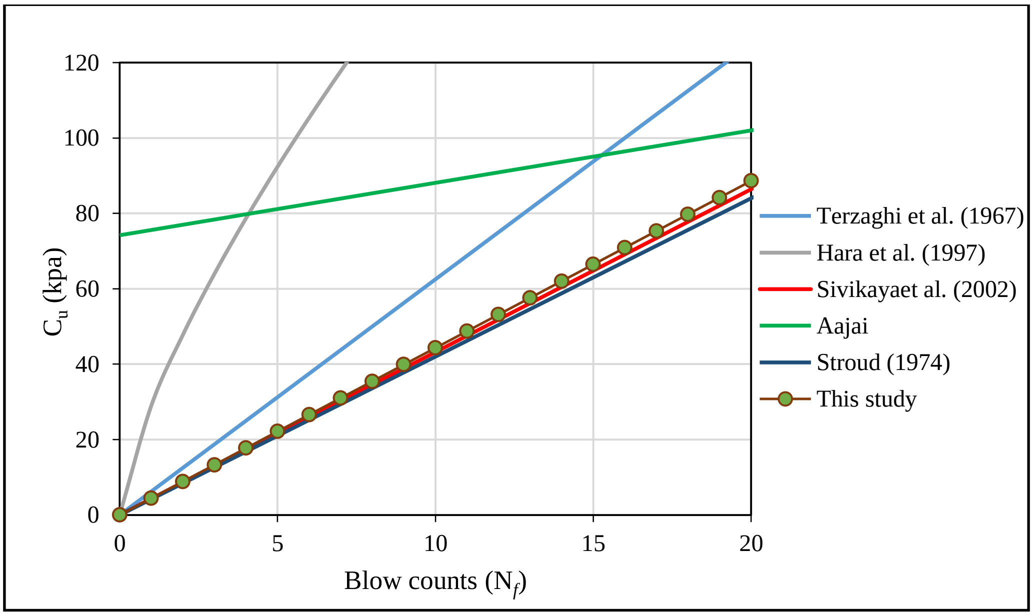
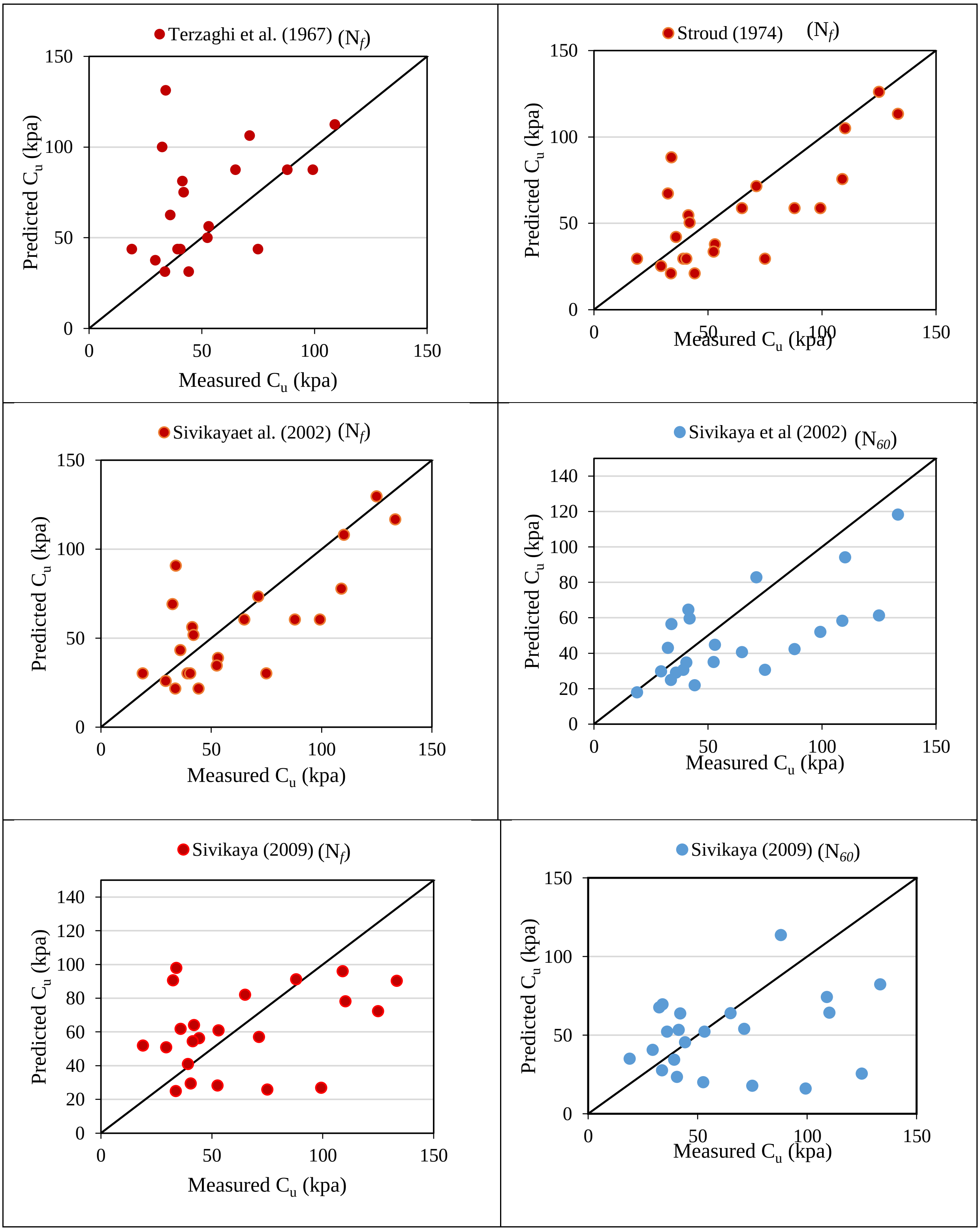


Fig. 5.8 Existing and proposed C_u equation

Some of the existing empirical equations obtained from the Terzaghi (1967), Stroud (1974), Hara (1974), Sivikaya &Togrol (2002), and Ajayi& Ballogun (1988) were undertaken to obtain an overview of the current information within the knowledge base regarding the correlation of SPT N-values with undrained shear strength. From the above figure 5.9 based on measured N-values (N_f) the proposed equation is best fit with Sirvikaya et al., (2002), and Stroud equation's.

To evaluate the abilities of present proposed correlations, predicted C_u values were plotted versus measured C_u values and also compared with previous studies in fig 5.10. Correlations were developed by Sirvikaya (2009), Hettiarachchi et al., (2009), Terzaghi (1967), Stroud (1974), Aajai et al., (1988) and Frazad et al., (2011) were selected for comparison. Statistical evaluations of correlations were used from statistical data of Mean and Standard deviation (ratio of measured C_u to predicted C_u), respectively.



Terzaghi et al. (1967) (N_f)
Predicted C_u (kpa)
Measured C_u (kpa)
Stroud (1974) (N_f)
Predicted C_u (kpa)
Measured C_u (kpa)
Sivikayaet al. (2002) (N_f)
Predicted C_u (kpa)
Measured C_u (kpa)
Sivikaya et al (2002) (N_{60})
Predicted C_u (kpa)
Measured C_u (kpa)
Sivikaya (2009) (N_f)
Predicted C_u (kpa)
Measured C_u (kpa)
Sivikaya (2009) (N_{60})
Predicted C_u (kpa)
Measured C_u (kpa)

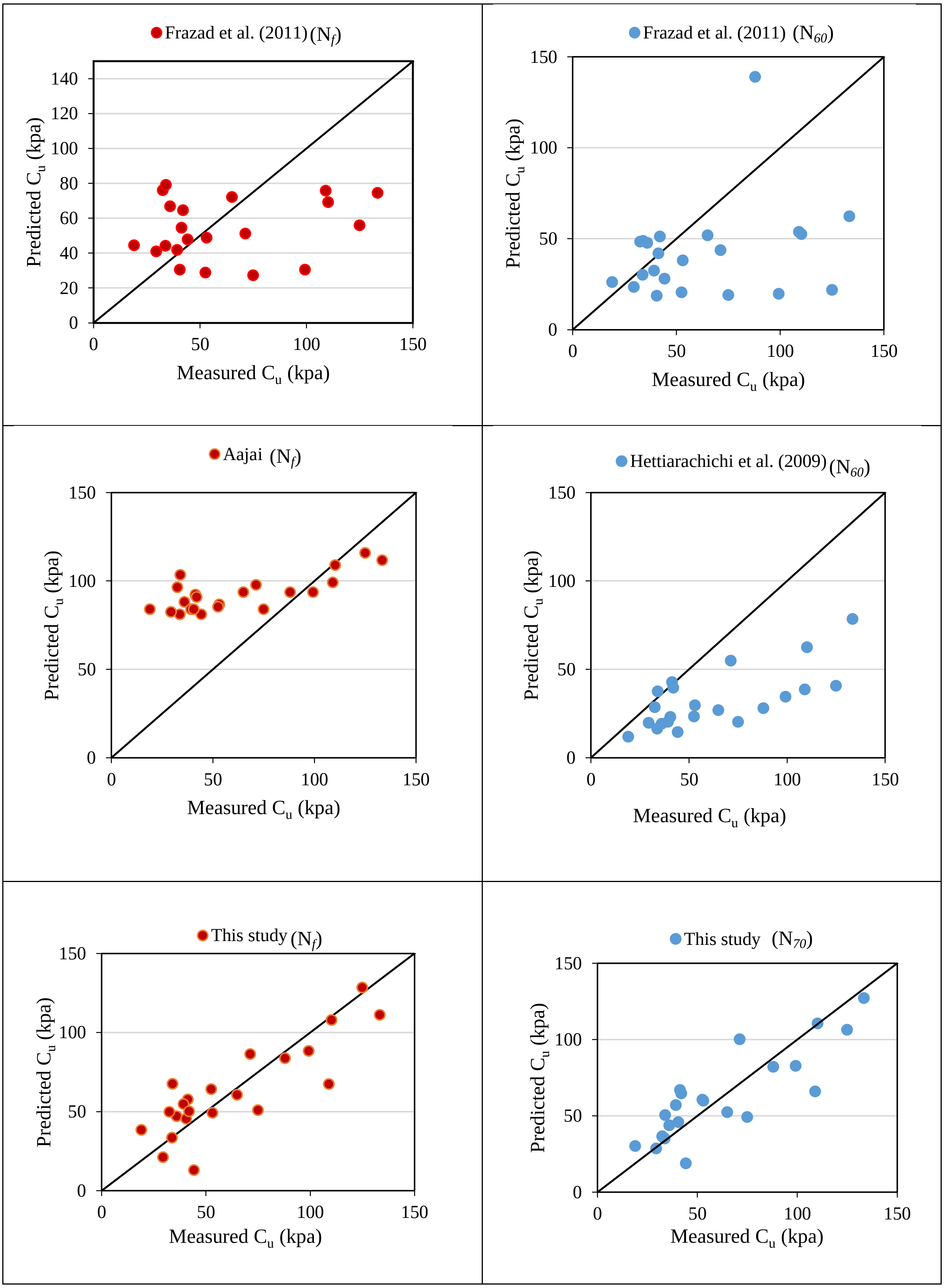


Fig. 5.9 Measured versus predicted C_u for present and previous proposed correlations

Generally the above plots shows that correlation was made between predicated C_u versus measured C_u values. The comparison was made based on corrected and uncorrected N-values of previous and the present proposed empirical equation. As mentioned earlier the previous empirical equations were developed based on SPT N-values to predict the shear strength of fine grained soils. The empirical equations that proposed by researchers were different. This can be even if the research was conducted in fine grained soil but it has different region with different SPT equipment and laboratory testing.

The correlation were developed by Stroud (1974), Sivikaya &Togrol (2002) and Terzhagi (1968), relatively good estimate with the present proposed equation whereas the equation developed by Hettiarachchi et al., (2009), and Aajai et al., (1988) had a wide difference. this difference come maybe the research is conducted in different region this lead the engineering properties of the fine grained soil and the equipment that used the test like SPT equipment (hammer system, split spoon sampler drill rod), drilling rigs, drilling system etc... The other possible cause of variation is most of the correlation SPT N- value standardized to N_{60} but the present proposed equation were developed based on $N_{70.}$

5.7 Comparison with SPT Method and Bearing Capacity

The comparison was done by calculating the bearing capacity of fine grained soils using SPT method and Meyerhof bearing capacity equation. The parameters that used for this comparison were corrected SPT N-value and shears strength parameters (c & Ø).

5.7.1 Bearing capacity based on SPT

After adjusting the N-counts based on the above formula a design N-value is chosen from consecutive depths where the test is performed. The design N-value is taken as the average of N-values which are found in between ½ B above and 2B below the proposed footing depth where B is the width of the foundation.

The bearing capacity for an isolated footing can be calculated from the SPT N- values using Meyerhof's equation as follows (Bowles, 1997);

$$q_{all} = \frac{N'}{F_2} * \left(1 + \frac{F_3}{B}\right)^2 * K_d,\ B > K_d \text{..} \textbf{Eq. 5.1}$$

Where

q_{all}	=	Allowable bearing pressure for settlement limited to 25 mm.
K_d	=	1+0.33D/B < 1.33
F_1	=	0.04
F_2	=	0.06
F_3	=	0.3
F_4	=	1.2
B	=	Width of foundation
D	=	Depth of foundation
(Factor of safety) FS	=	3

Table 5.5 presented the allowable bearing capacity of fine grained soil using SPT method and the assumed foundation depth is 3m and widths are varied from 2.0m to 6.0m.

Table 5.5 Allowable Bearing Capacity Based on SPT

			Allowable Bearing Capacity, kPa								
			Assumed Depth of Foundation (3m)								
			Width of Foundation (B), m								
Soil type	BH-No.	N_{70}	2	2.5	3	3.5	4	4.5	5	5.5	6
Dark clay	BH-2	4	117	111	103	97	93	90	87	85	84
	BH-3	4	117	111	107	101	96	93	90	87	86
	BH-4	4	117	111	107	105	99	95	92	90	88
	BH-7	6	176	167	161	157	154	151	149	145	141
	BH-13	5	147	139	134	131	128	126	125	120	117
	BH-14	4	117	111	107	105	102	101	100	96	94
	BH-21	4.5	132	125	121	118	115	113	112	108	105
	BH-22	9.8	287	272	263	256	251	247	244	236	230
Red clay	BH-1	6	176	159	148	140	133	130	127	125	122
	BH-5	10	293	278	268	261	256	245	237	230	224
	BH-9	15	440	417	402	392	384	378	374	361	351
	BH-12	13	381	361	349	340	333	328	324	313	305
	BH-16	4.6	135	128	123	120	118	116	115	111	108
	BH-17	6	176	167	161	157	154	151	149	145	141
	BH-20	4	117	111	107	105	102	101	100	96	94
Red silt	BH-6	5	147	139	134	131	128	126	121	118	115
	BH-8	11	322	306	295	287	282	277	274	265	258
	BH-10	9	264	250	241	235	231	227	224	217	211
	BH-11	10	293	278	268	261	256	252	249	241	234
	BH-15	7	205	195	188	183	179	177	174	169	164
	BH-18	9.4	276	261	252	246	241	237	234	226	220
	BH-19	4	117	111	107	105	102	101	100	96	94

From the above analysis, the allowable bearing pressure values for dark clay soils vary from 287 to 84kPa, red clay soils varies from 440 to 94kpa and red silt soils varies from 322 to 94kpa.

5.7.2 Bearing Capacity based on UCS

The net ultimate bearing pressure for vertical loads on cohesive soils is normally computed as a simplification of either the Meyerhof or Hansen bearing capacity equations (Bowles, 1997). For cohesive soils, changes in ground water levels do not affect theoretical ultimate bearing capacity. For the most critical stability state ($\varphi = 0^o$), which is created when the foundation load is applied so rapidly, the immediate bearing capacity is independent of the location of the water table.

The ultimate bearing capacity of the footings can be calculated using: In order to get the shear strength properties of the silty clay to clayey silt soil, Unconfined Compressive Strength test was conducted on the sample retrieved. The Meyerhof's bearing capacity equation is used to calculate the bearing capacity values. Allowable Bearing Capacity for Footing Using Meyerhof's equation:

$$\mathbf{q_{ult} = CN_c s_c d_c + qN_q s_q d_q + 0.5\gamma B N\gamma s\gamma d\gamma} \quad \text{.....................................Eq. 5.2}$$

Where,

q_{ult} = Ultimate Bearing Capacity

q_{all} = Allowable Bearing Capacity

C = Cohesion (kPa)

γ = Unit Weight (kN/m3)

B = Width of Footing

D = Depth of Footing

sc, sq, sγ = Shape Factors

dc,dq,dγ = Depth Factors

Nc,Nq,Nγ = Bearing Capacity Factors

Substituting the relevant values in Meyerhof's equation we have obtained the ultimate and allowable bearing capacity values for the foundation soil.

It was used the c values of the UCS tests. When c = Cu, we have $\varphi = 0^0$, Nc = 5.14. Since $N\gamma = 0$ when $\varphi = 0^o$, the equation is then simplified as follows:

$$q_{ult} = CNcscdc + qNqsqdq \quad \text{Eq. 5.3}$$

Table 5.6 Allowable Bearing Capacity Based on UCS

Soil type	BH-No.	Allowable Bearing Capacity, kPa — Assumed Depth of Foundation (3m) — Width of Foundation (B), m								
		2	2.5	3	3.5	4	4.5	5	5.5	6
Dark clay	BH-2	291	279	271	265	261	257	255	252	251
	BH-3	176	169	165	162	159	157	156	155	154
	BH-4	103	100	97	96	94	93	93	92	91
	BH-7	163	157	153	150	148	146	144	143	142
	BH-13	238	229	222	217	214	211	209	207	206
	BH-14	203	195	189	185	182	180	178	177	176
	BH-21	191	183	178	175	172	170	168	167	166
	BH-22	196	188	183	179	176	174	172	171	170
Red clay	BH-1	458	438	424	415	408	402	397	394	391
	BH-5	570	545	528	517	508	501	495	490	487
	BH-9	469	449	435	425	418	412	408	404	401
	BH-12	568	543	527	515	506	499	494	489	486
	BH-16	145	140	136	133	131	130	129	128	127
	BH-17	184	177	172	169	166	164	163	161	160
	BH-20	316	302	293	286	281	277	274	272	269
Red silt	BH-6	157	151	147	144	142	141	139	138	137
	BH-8	381	365	354	346	341	336	332	329	327
	BH-10	425	407	394	386	379	374	370	367	364
	BH-11	161	155	150	147	145	143	142	141	140
	BH-15	191	184	179	175	172	170	168	167	166
	BH-18	314	301	292	286	281	278	275	272	270
	BH-19	234	225	218	214	210	207	205	203	202

The allowable bearing pressure values listed in table 5.6 computed based on c & φ. The values the allowable bearing pressure values for dark clay soils vary from 291 to 91kPa, red clay soils varies from 570 to 127kpa and red silt soils varies from 425 to 137kpa.

Comparison was made the value given in Table 5.5 and 5.6. In this case, it was tried investigating the capability of SPT method determining the bearing capacity.

Table 5.7 Bearing capacity comparison between SPT and UCS

Soil Type	Allowable bearing capacity (kpa)	
	Based on SPT	**Based on UCS**
Dark clay	287 to 84	291 to 91
Red clay	440 to 94	570 to 127
Red silt	322 to 94	425 to 137

Table 5.7 present the comparison between SPT and UCS. Generally the allowable bearing pressures calculated using the C_u values obtained from the UCS tests are higher than the ones obtained using the SPT method. It was not expect that the field and laboratory values give the same result in the determination of bearing capacity. SPT get close result with the bearing capacity equation with minor variation.

5.8 Final Remark

As it is known the aim of this analysis is assessing the capability SPT determining bearing capacity of fine grained soils. This identifying the relationship between mentioned parameters. The correlations were developed using simple and multi linear regressing analysis. In single regression analysis was considered two parameters; SPT N-value and C_u. Empirical equations were also proposed to estimate the undrained shear strength parameter of fine grain soil using corrected SPT number and the accuracy of the equation is proved by moderately relationship.

In multi linear regressing analysis, the correlation was considered PI, PL, LL and W_n in addition to N-value.in predicting undrained shear strength the second correlation was better than the first analysis. Thus its implies SPT significantly estimating the undrained shear strength especially in addition with plastic index, liquid limit , plastic limit and moisture content. Lastly this analysis and interpretation is evaluated by comparing measured and predicated value of undrained shear strength and existing correlation. Statistical evaluations of correlations were used from statistical data of Mean and Standard deviation (ratio of measured C_u to predicted C_u, respectively. the recent correlations were developed Sivikaya &Togrol (2002), relatively good estimate with the proposed correlation. Therefore standard penetration test is capable to predict undrained shear strength.

CHAPTER 6 CONCLUSION AND RECOMMENDATIONS

6.1 Conclusion

In-situ SPT testing of fine grained soils is a common practice in the world. It is mainly conducted to determine the bearing capacity of fine grain soils when taking samples is not reliable for laboratory testing. It is also involved to determine the soil's penetration resistance.

Likewise in Ethiopia, SPT has been widely practiced for foundation investigation and mostly empirical methods are employed to analyze the values for the estimation of bearing capacity. However, questions are being raised as the SPT measurement are affected by different kinds of hammers, sampling tubes, drill rig types, drill rod lengths and types, hammer blow rates and different energy delivery systems with different degrees of efficiency. The reliability of the method and the accuracy of the result depend largely on the test equipment, and test procedures well as and the test procedure. The present study, hence, has been done to assess the reliability of standard penetration test for determination of bearing capacity of fine grain soil.

This research was carried out based on the geotechnical data of fine grained soil that are collected from different locations of Addis Ababa. The studied soil have been classified based on their color into three category; dark clay, red clay and red silt. In order to achieve the objectives of this research a systematic methodology was followed. Twenty two borehole drillings were engaged in this study using different (donut hammer and automatic hammer. A total of 100 SPTs were conducted at 15 cm interval on average and 22 undisturbed soil samples were taken for the determination of unconfined compressive strength. Disturbed samples were also taken to describe the soil properties. Simultaneously information about SPT equipment has been collected for dealing a possible source of errors. The possible sources of errors during performing SPT were from test procedure; such as testing intervals and blow count were recorded. Generally, the rig type, hammer efficiency, the omission of liners in the spoon, split spoon sampler design variation and sharpness and the presence of gravel and pebbles were also documented.

In order to reduce the significant variability associated with the SPT tools, it was used that N-values to be standardized to N_{70}. This standardization was to be achieved by correcting the measured field N-values by applying different correction factors including hammer efficiency, split spoon sampler, rod length, borehole diameter, and anvil and blow rate corrections.

Based on the test results and analysis, interpretations were made to meet the general objectives of the present study. Accordingly, simple relationships were proposed to estimate undrained shear strength of fine grain soils. New empirical equations were also proposed to determine undrained shear strength of fine grain soil using linear and multi linear regression analysis. Furthermore, other parameters (PL, LL, PI and W_n) are correlated with SPT in the determination of undrained shear strength and likely increase the correlation coefficient of estimation. These relationships cannot be considered as a perfect substitute of laboratory values and the implementation of the findings requires experience and high certitude of engineering judgments as well as continuous controlling with obtained laboratory experiments.

Finally, comparison was made between predicted and measured value of undrained shear strength of fine grain soil. The predicted C_u result reasonably fair to good comparison between the predicted and measured undrained shear strength values. Compared to previous correlations, correlations presented by Sirvikaya (2002) and Stroud (1979) had the best conformity with present proposed correlations but those correlation presented by Aajai et al., (1988) and Hettiarachchi et al., (2009), had relatively large differences. These differences could have been caused by variation of the physical properties soils, local ground condition, SPT equipment and test procedure variation.

The following results are concluded based on the results, discussion and analysis presented in this research:

- Donut hammer system and its lifting mechanism is significantly affected the result of measured N-value relative to an automatic hammer.
- Automatic hammer is providing relatively a consistent result of N-values than donut hammer especially in energy transfer mechanism.
- The N-value is mainly influenced by hammer efficiency and split spoon design. Deformed cutting shoe was the major source of errors.

- The N-values is mainly influenced by the use of unlubricated cathead and rope in donut hammer.
- The depth of soil below ground surface significantly affects the SPT number.
- The Atterberg limits (LL, PL, PI) and moisture content (W_n) has not affect the SPT number.
- The undrained shear strength of fine grain soil is strongly affects the SPT number.
- Empirical equations to predict C_u of fine grain soil using SPT have been presented. The accuracy of these equations proved to be moderately relationships.
- The standard penetration test is considered reliable in predicting of undrained shear strength to determine bearing capacity with good maintenance of the equipment and quality control in the performance of the test.

6.2 Recommendations and Future Work

Based on the conclusion made above, the appropriate recommendations are given here under:

- SPT Hammer efficiency should be checked and recalibrated periodically using energy measurement tools.
- SPT equipment's used should be evaluated because the energy delivered by the hammer may be quite variable.

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- The SPT N-value of fine grain soils especially clay soils may vary significantly with seasonal fluctuations in the water table. These correlations are not as meaningful for sensitive and medium to soft clays where effects of disturbance during sampler penetration may cause a lowering in the SPT N-value.
- Users should carefully inspect the split spoon sampler especially the cutting shoe.
- For different regions, it is recommended the specific relationship regarding that particular area to be used when predicting values for Cu.
- SPT measurement should be conducted by a well professional drilling crew and a good engineer on- for a quality and reliable results.

- Understanding the SPT N-value, the engineer will be able use the SPT to come up with a quick estimation of the bearing capacity.

- The SPT should be used with discrete judgment when it is used to estimate the bearing capacity of cohesive soils; since silt and clay may be stiffened or softened depending on an increase or decrease of their moisture contents.

- The SPT number may be misleading if large-size gravel is wedged into the split spoon sampler resulting in apparently high N-values, so it should be checked and make a note if any coarse particle observed in the fine grain soil.

- It is hoped that this paper will help the testing much more reliable and can only be accomplished with careful attention to equipment and procedures when performing the test.

Finally, the results and findings of the present study may be considered as indicative only as these findings are based on limited parameters and on small number of in-situ and laboratory tests. Further detail investigation should be carried out. More elaborate sampling and field testing would be mandatory before implementation of results from the present study. However, the present study provides a general methodology to conduct similar studies on reliability of SPT method.

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