ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

ADDIS ABABA INSTITUTE OF THECHNOLOGY

DEPARPMENT OF CIVIL ENGINEERING

DEVELOPING CORRELATION BETWEEN DYNAMIC CONE PENETRATION INDEX AND UNDRAINED SHEAR STRENGTH OF SOILS THAT ARE FOUND IN DEBRE MARKOS TOWN

BY:GEDEYON ANDUALEM

"A thesis submitted to the school of graduate studies of Addis Ababa University in partial fulfilment of the requirements for the degree of Master of Science in civil engineering (Geotechniques)"

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ABBREVIATIONS, ACRONYMS AND SYMBOLS

ASTM	American Society for Testing of Materials
A_o	Initial area
BS	British Standard
C	Clay Content
CBR	California Bearing Ratio
CH	Inorganic Clay of High Plasticity
CIRIA	Construction Industry Research Information Association
c_u	Undrained cohesive resistance
d	depth/diameter
DCP	Dynamic Cone Penetrometer
DCPI	Dynamic Cone Penetrometer Index
E_{eff}	Elasticity Modulus
ε	Axial Strain
FS	Free Swell
G_s	Specific Gravity
γ	Unit weight of soil
h	Depth of cone tip while recording
Ic	Consistency Index
kPa	kilo Pascal
LL	Liquid Limit
m	meter
MH	Inorganic Silt of High Plasticity
mm	Millimetre
NGL	Natural Ground Level
NMC	Natural Moisture Content

N_c	Bearing capacity factor for cohesion
N_q	Bearing capacity factor for surcharge
N_g	Bearing capacity factor for unit weight
ORN	Overseas Road Note
w	Water Content
PI	Plasticity Index
PL	Plastic Limit
ϕ	Angle of internal friction
p	Foundation Pressure
q	Effective vertical pressure
q_{ult}	Ultimate bearing capacity
R^2	Coefficient of determination
s_u	Undrained shear strength of soil
SPTN-value	Standard Penetration Test Number (blows/300mm penetration)
SPSS	Statistical Package for the Social Sciences
σ	Normal Stress on Shear Plane
TRRL	Transport Road Research Laboratory
UCS	Unconfined Compression Strength
UK	United Kingdom
USAID	United States Agency for International Development
USCS	Unified Soil Classification System

ABSTRACT

Determination of the in-situ engineering properties of foundation materials has always been a challenge for practicing engineers in developing countries. Especially in a country like Ethiopia, where construction of small scale buildings like residential, warehouses, health centres, which are thought to solve the problem of the society in general, is growing in a fast rate. Debre-Markos is one of the fastest developing towns in Ethiopia, which is exposed to rapid civil engineering works like high-rise buildings, and large residential buildings, banks, etc. Since the municipality (sub-city) does not force the designers to conduct soil investigation for buildings less than G+4, designers depend on simple visual inspection and then use the minimum recommended presumptive bearing capacity by EBCS.Even some designers or companies do not conduct soil investigation for building greater than G+4. Some of these predictions may lead to unexpected failures of structure or uneconomical design. To avoid such problems, this research introduces the use of Dynamic Cone Penetration (DCP) which is a simple test device that is inexpensive, portable, and easy to operate and understand.

In this thesis, field tests were conducted by locally manufactured dynamic cone penetration equipment from available materials. Laboratory tests needed to classify the soil and study the parameters that affect the dynamic cone penetration index and undrained shear strength of soil were also conducted. The data has been categorised in to two categories, correlation between undrained shear strength and dynamic cone penetration index has been developed. In addition correlation between parameters that affect the DCPI and undrained shear strength of soil has also been developed. It has been found out that parameters like unconfined compression strength& liquidity index have influence on the Dynamic Cone Penetration Index (DCPI). These research work revealed that Undrained shear strength can be estimated by UCS= -209.5*ln (DCPI) +800.5 for red clay soils of Debre Markos and UCS= -7.1661*(DCPI) +416.82 for black clay (expansive) soils of Debre Markos in general. These correlations were used to develop bearing capacity equation based on the bearing capacity theory. The equation found were q_{ult}= -538.625ln (DCPI) +12058.086+γh for red clay soils of Debre Markos, q_{ult}= -18.423*(DCPI) +1071.64+γh for black clayey (expansive) soils of Debre Markos. The results are expected to have wide application in the construction sector.

1. INTRODUCTION

1.1 GENERAL

Determination of the in situ engineering properties of foundation materials has always been a challenge for practicing engineers in developing countries. Especially in a country like Ethiopia, where construction of small scale buildings like residential, warehouses, health centres, which are thought to solve the problem of the society in general, is growing in a fast rate. Since the municipality (sub-city) does not force the designer to conduct soil investigation for buildings less than G+4, designers depend on simple visual inspection and then use the minimum recommended presumptive bearing capacity by EBCS.Even some designer or company does not conduct soil investigation for building greater than G+4. Some of these predictions may lead to unexpected failures of structure or uneconomical design which affecting the quality of the construction output to great extent. This trend should have to be replaced by a more reliable mode of determining the bearing properties and performance of soils.

Debre Markos is one of the fastest developing towns in Ethiopia, however the designer does not conduct soil investigation during construction. Thus, it is important to make some soil investigation. There are several devices developed for the determination of the in situ engineering properties of foundation materials so far. Dynamic Cone Penetration (DCP) test is the most versatile amongst them. DCP is a simple test device that is inexpensive, portable, easy to operate, and easy to understand. It does not require extensive experience to interpret results but correlations to more widely known strength measurements have to be established for the adaptation of the application. Among which undrained shear strength values of soils is the one. Due to the relatively fast and easy use of DCP test the development of correlation between DCPI and UCS test will reduce cost and time required for design and construction.

1.2 OBJECTIVES

1.2.1 General Objective

The general objective of this thesis is to develop correlation between Dynamic Cone Penetration Index (DCPI) and undrained shear strength (s_u) soils found in Debre Markos town.

1.2.2 Specific Objective

The specific objectives of the research are:

- **a)** To determine the index properties of soils found in Debre Markos.
- **b)** To study factors and parameters of soil properties that affect DCP results.
- **c)** To develop bearing capacity equations based on bearing capacity theory and the correlation obtained from this study.

1.3 APPLICATION AND LIMITATION OF THE STUDY

1.3.1 Application of the Results

The results from this research can be used to estimate bearing capacity of shallow foundations in the area of study which has mostly clayey soils; both red and black cotton /expansive/soils. The results are especially useful where building or structure is of lightweight construction. It can also be used for construction control and field exploration where value does not justify the cost of a drilling rig or where access prohibits a drilling machine.

In addition to the estimation of bearing capacity, once manufactured the equipment can be used for measuring the strength of soil on site, to estimate the thickness and location of underlining layers; to identify weak spots and measuring uniformity of materials in compacted fills.

1.3.2 Limitation of the Study

As in most researches that attempt to correlate different engineering parameters, the size of statistical data is the main factor that limits the applicability of the results obtained. The other limitation would be the locations of sample collection. Since DCP result is highly material dependent, the applicability will also be limited to the areas of the study. Therefore, the results should only be applied to the study areas.

1.4 ORGANIZATION OF THE THESIS

The thesis is organized into seven Chapters. The first Chapter presents a general description and major engineering problems associated with bearing capacity determination, application and limitation of this research work. A review on the dynamic cone penetrometer and undrained shear strength including previous works on determination of the parameters are discussed in the second Chapter. The review also covers bearing capacity theory. Description of the study area is presented in the third Chapter. The fourth Chapter presents test methods, data collection and test results. The analyses of parameters under study are presented in the fifth Chapter. Discussions on analysis are presented in the sixth Chapter. Finally, conclusions and recommendation are given.

2. LITRATURE REVIEW

2.1 GENERAL

The purpose of this literature review is to give introduction about DCP's historical background and undrained shear strength of soil. The review also covers the following areas:

- summary of DCP equipment, procedures and specifications that exist on the device;
- discussion on soil parameters that affect DCP result;
- discussion on shear strength of soils and ways of predicting the parameter;
- discussion on different applications of DCP test for the evaluation of shear strength and other geotechnical parameters;
- Discussion on bearing capacity theory and shear resistance characteristics of penetrometers;
- Discussion on related studies.

2.2 DYNAMIC CONE PENTROMETER

2.2.1 History of DCP Equipment

Soil penetration testing devices like the DCP have a long history. Perhaps the earliest penetration testing devices were driven piles. On a project requiring piles, a builder would install "test" piles to determine their required length. These "test" piles would be driven until a certain rate of penetration was achieved. Once that rate was reached, it was assumed that future installation of the same length piles would be satisfactory [6].

The earliest record of a subsoil penetration testing device similar to the DCP is a "ram penetrometer," developed in Germany at the end of 17th century by Nicholas Goldman. The next major development again came from Germany, when Künzel in 1936 developed what was known as a "Prüfstab" which mean dip stick. This device was later used by Paproth in 1943, and eventually become standardized in 1964 as the "Light Penetrometer"[6].

Concurrent with the German standardization of the "Light Penetrometer", several other countries developed their own standard penetration devices. Development of the hand-held DCP is credited to scale of Australia in the mid-1950^{s}.

The developed DCP was based on an older Swiss origin, to evaluate the shear strength of the material in a pavement. This consisted of a 9-kg mass dropping 508-mm and knocking a cone with a 30° point into the material being tested. The next generation of DCP equipment was developed by Van Vuuren from South Africa. Basically it was similar to the DCP apparatus developed in Australia except that the weight of the drop hammer was changed to 10Kg and the drop height was changed to 383.5mm.

The potential of this device was noted and development of the device continued in South Africa. With time a number of variants were in use, all with different masses, fall-distances and even cone dimensions although the energy imparted (mass x fall) was generally similar. During the early 1970's the device was standardized in South Africa with the dimensions of 8-kg mass with falling height of 575-mm and 60° cone and this has become the standard for DCPs, although a 30^0 cone can be used when measuring the penetration index in stiffer soils[4].

2.2.2 Description of the device

The Dynamic Cone Penetrometer (DCP) used in this research is based on the widely accepted South African standard as modified by the Transvaal Road Department [24]. The device was manufactured locally for the purpose of conducting this research and used for collecting dynamic cone penetration data for sites inside Debre Markos town. The device consists of two 16-mm diameter shafts coupled near midpoint. The lower shaft contains an anvil and a pointed tip which is driven into the soil by dropping a hammer contained on the upper shaft onto the anvil. The underlying soil strength is determined by measuring the penetration of the lower shaft into the soil after each hammer drop. This value is record in millimetres per blow and is known as the dynamic cone penetration index (DCPI).Since the manufactured device does have extension rods; it will only be possible to measure up to 1.0m with this arrangement.

Complete drawings of the DCP are given in figure 2.1.The DCP is comprised of the following elements.

a) **Handle:** The handle is located at the top of the device. It is used to hold the DCP shafts plumb and to limit the upward movement of the hammer.

b) **Hammer:** The 8-kg hammer is manually raised to the bottom of the handle and then allowed to fall freely to transfer energy through the lower shafts to the cone tip. It is guided by the upper shaft.

c) **Drop Height (Upper Shaft):** The upper shaft is a 16-mm diameter steel, on which the hammer moves. The length of the shaft allows the hammer to drop a distance of 575-mm.

d) **Anvil:** The anvil serves as the lower stopping mechanism for the hammer. It also serves as a connector between the upper and the lower shaft. This allows for disassembly which reduces the size of the instrument for transport.

e) **Steel Rod (Lower Shaft):** The lower shaft could be 900-1200-mm long, if possible marked in 5-mm increment for recording the penetration after each hammer drop.

f) The cone measures 20 mm in diameter and has a 60^0 cone.

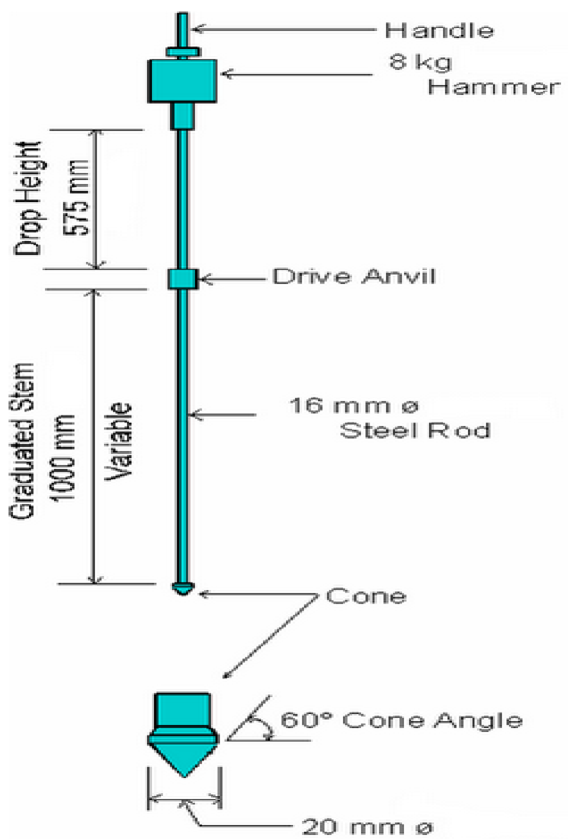

Figure 2-1 Dynamic Cone Penetration Equipment [24]

The combined mass of the upper shaft, anvil, lower shaft and cone is approximately 3.1-kg. The DCP (except the hammer) is usually constructed of stainless steel to prevent corrosion. But, if ordinary or mild steel is employed, the instrument has to be cleaned and dried after each use to prevent rusting. The cone tip should be replaced when the diameter of its widest section is deformed by more than 10% (2-mm).

2.2.3 Test Procedure

Operation of the DCP requires three persons; one person to hold the apparatus vertical and one person operating the hammer and the other to record the depth of penetration. The following steps are followed:

1. The operator holds the device vertical by the handle on the top shaft and "sealing" the cone tip by dropping the hammer until the widest part of the cone is just below the testing surface. A second person records the height at the bottom of the anvil in reference to the ground, this is recorded as initial penetration as "blow zero".

2. The operator lifts the hammer from the anvil to the handle, and then releases the hammer. The second person records the new height at the bottom of the anvil.

3. Step 2 is repeated until the desired depth of testing is reached or the full length of the lower rod is buried. The rod is 1m high and since there is unavailability of extension rod the test is done by excavating the soil every 1m.the soil is less confined near the surface and during excavation the upper soil is disturbed so that the DCP is able to penetrate further per drop thus making the initial drops unreliable hence the first two reading are taken as seating blows. The test is done up to 4m based on the soil condition. At that time, a specially adapted jack is used to extract the device. If the tip is disposable (i.e., not fastened to the lower shaft and left in the soil after test is complete), hitting the hammer lightly on the handle is acceptable.

2.2.4 Factors Affecting DCP Results

2.2.4.1 Material Effects

Several investigators have studied the influence of several factors on DCPI.Kleyn and Savage (1982) cited by Amini[26] indicated that moisture content, gradation, density, and plasticity were important material properties influencing the DCPI.Hansen (1996) cited by Amini [14] performed a study on the effects of several variables on DCPI.He conclude that for fine grained soils, moisture content, soil classification, dry density and confining pressure influence the DCPI.However for coarse-grained soils, coefficient of uniformity and confining pressures were important variables.

2.2.4.2 Vertical Confinement Effect

Livneh, et al. (1995) performed a comprehensive study about the vertical confinement effect on dynamic cone penetration strength value in pavement and subgrade evaluation. The results have shown that there is no vertical confinement effect by rigid pavement structure or by cohesive layers on DCP values of lower cohesive subgrade layer for homogenous soil layer. Any difference between confined and unconfined values in the case of granular materials is due to the friction developed in the DCP rod by tilted penetration or by a collapse of the granular material on the rod surface during penetration.

2.2.4.3 Side Friction Effect

Since the DCP device is not completely vertical while penetrating through the soil, the penetration resistance would be apparently higher due to side friction. This apparent higher resistance may also be caused when penetrating in a collapsible granular material. This effect is usually small in cohesive soils compared to collapsible granular material [19].

2.2.5 Benefits and Limitations

The DCP offers many benefits compared to other similar hand-held testing devices. Its benefits make the device not only inexpensive, portable and easy to operate and understand but also the most versatile among other similar equipment. Some of these benefits are listed below:

a) **Easy to Use:** It does not take extensive experience to interpret results. An operator can be trained in a matter of minutes. Its light weight makes it preferable for field exploration for light weight structures.
b) **Large Penetration Depth:** Data can be collected to a depth of 6m using extension rods compared to a maximum of 0.3m for other hand-held testing devices like the vane shear test.

c) **Fast:** A large amount of data can be taken quickly, and the DCPI values are easily converted into other indices which are used to determine the bearing properties and performance of the underlying soil.

d) **Low Cost:** Currently, the device can be manufactured locally from available material or even could be rented cheaply.

e) **Versatility**: The device has found many applications in the construction field for construction control, supervision and design parameter determination. Some of the items are:

The dynamic cone penetrometer has its own limitations; some of these are caused by the operators of the equipment. One should not be surprised to find out that the results of two DCP tests done on the same site only a few meters apart are not the same. These errors include tilting of the equipment, falling height of the hammer, etc.

Other than manpower errors there are also other limitations:

- ❖ Adhesion between the rod and the soil for highly plastic soil and collapsible granular soils.
- ❖ It is difficult to penetrate hard and granular materials.
- ❖ As in most dynamic tests, the DCP does not give reliable result in saturated fine graded soils. This is because the dynamic load from the equipment is carried by a developed pore water pressure rather than the soil grains in these type soils.

The maximum depth suggested for this test is about 6m using extension rod. If tests have to be conducted beyond 6m depth, one has to use lubrication between the hole and the rod throughout the test.

2.3 UNDRIANED SHEAR STRENGTH OF SOIL

2.3.1 General

Shearing strength of soil is the most important of its engineering properties. This is because all stability analysis in the field of geotechnical engineering, either they relate to foundation, slope of cuts or earth dam, involve a basic knowledge of this engineering property of the soil. Shear strength may be defined as the resistance to shearing stresses and a consequent tendency for shear deformation.

Basically, a soil derives its shearing strength from the following

- ✓ Resistance due to the interlocking of particles.
- ✓ Frictional resistance between the individual soil grain, which may be sliding friction, rolling friction or both
- ✓ Cohesion between soil particles.

Granular soils of sands may derive their strength from the first two sources, while cohesive soils may derive their shear strength from the second and third source. Highly plastic clays, however, may exhibit the third source alone for their shearing strength. Most natural soil deposits are partially cohesive and partially granular and as such, may fall in to the second of the three categories just mentioned, from the point of view of shearing strength.

The shear strength of a soil cannot be tabulated in codes of practise since a soil can significantly exhibits different shear strength under different field and engineering conditions.

The shear strength is measured in terms of two soil parameters; interparticle attraction or cohesion and resistance to interparticle slip called the angle of internal friction .Grain crushing resistance to rolling, and other factors are implicitly include in these two parameters. In equation form the shear strength in terms of total stress is

$$s = c + \sigma \tan\phi \quad \text{……………………………………………………………………… (2.1)}$$

Where s is the shear stress at failure along any plane

σ is the normal stress on that plane and

c and ϕ are the shear strength parameters; cohesion and angle of shearing resistance.

A complication arises when the normal stresses within a soil are carried partly by the soil skeleton itself and partly by water within the soil voids. Considering only the stresses within the soil skeleton, equation 2.1 is modified to equation 2.2.

$$s = c' + (\sigma - u)\tan\phi' \quad \text{...(2.2)}$$

Or

$$s = c' + \sigma'\tan\phi \quad \text{...(2.3)}$$

Where u is the pore water pressure

$\sigma' = (\sigma' - u)$, the effective normal stress (on the soil skeleton) and

u is pore water pressure developed

c' and ϕ' are the shear strength parameters related to effective stresses.

The choice between total and effective stress analysis depends on the application. In case of foundation design, because it imposes both shear stresses and compressive stresses (confining pressures) on the underlying soil; the shear stresses must be carried by the soil skeleton but the compressive stresses are initially carried largely by the resulting increase in pore water pressures. This leaves the effective stresses little changed, which implies that the foundation loading is not accompanied by any increase in shear strength. As the excess pore pressures dissipate, the soil consolidates, and effective stresses increase, leading to an increase in shear strength. Thus, for foundations, it is the short term condition, the immediate response of the soil, which is most critical. This is the justification for the use of quick undrained shear strength tests rather than effective stress analysis for foundation design. Effective stress analysis must be used where long-term stability is important. Based on the USCS the soil in the research area is categorized as MH (highly plastic silt soil) and CH (highly plastic clay soil).So most of the discussion in this thesis consider fine grained soil. The undrained shear strength of soil can be determined from the laboratory test and beside that there are other methods of determining the shear strength of fine-grained soil.

2.3.2 Method of Predicting Undrained Shear Strength of soil

There are many ways to predict the undrained shear strength of clay soils where the normal laboratory becomes difficult to perform or when cross checking is required.

i. From simple classification test

One way of predicting undrained shear strength is by using simple laboratory tests like Atterberg limits. It is known that the liquid and plastic limits are moisture contents at which soil has specific values of undrained shear strength. It therefore follows that, for a remoulded soil, the shear strength depends on the value of the natural moisture content in relation to the liquid and plastic limit values. This can be conveniently expressed by using the concept of liquidity index.

Curves relating remoulded undrained shear strength to liquidity index have been established by Skempton and Northey (1952) as cited by [14] and these are given in Figure 2-2.

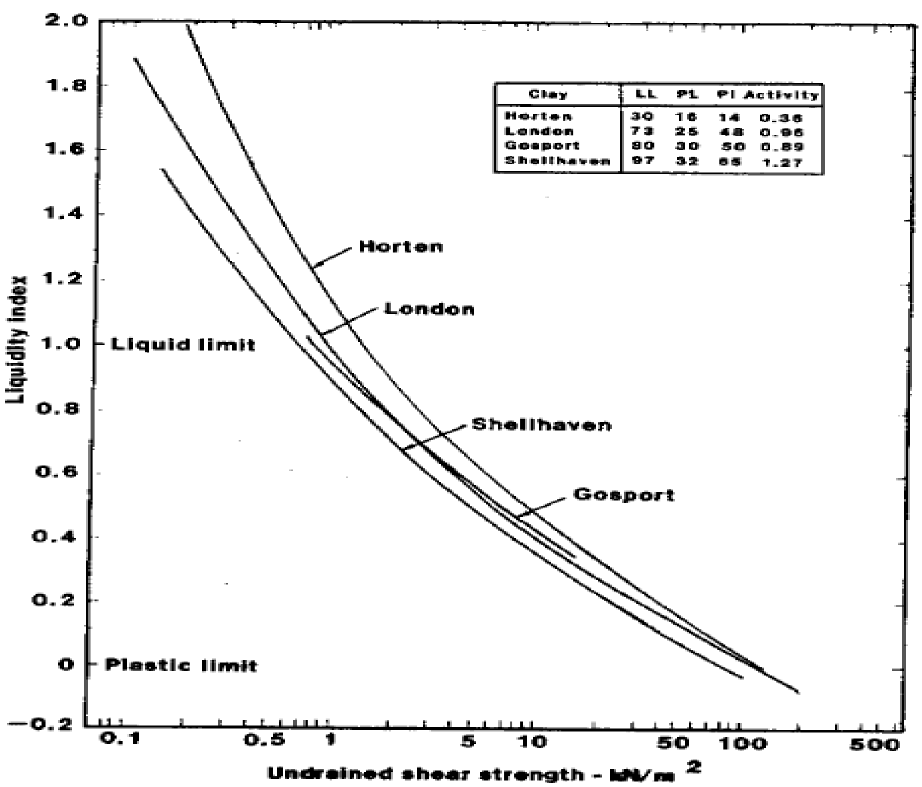

Figure 2-2 Correlation between shear strength and liquidity index established by Skempton and Northey as cited by [14].

The shear strength of undisturbed clays depends on the consolidation history of the clay as well as the fabric characteristics. The ratio of natural shear strength to remoulded shear strength is known as the sensitivity. It is most marked in soft, lightly consolidated clays which have an open structure and high moisture content. Sensitivity may be related to liquidity index, and this has indeed been found so by a number of researchers. The work of Skempton and Northey (1952) as cited by [14] relates mainly to clays of relatively moderate sensitivity with natural moisture contents below the liquid limit. Their findings are given in Figure 2-3.

It has been shown that both remoulded shear strength and sensitivity can be correlated with liquidity index. It follows that a correlation must exist between undisturbed shear strength and liquidity index. Such a relationship provides a useful predictive tool for assessing the shear strength of undisturbed soils (Figures 2-4).

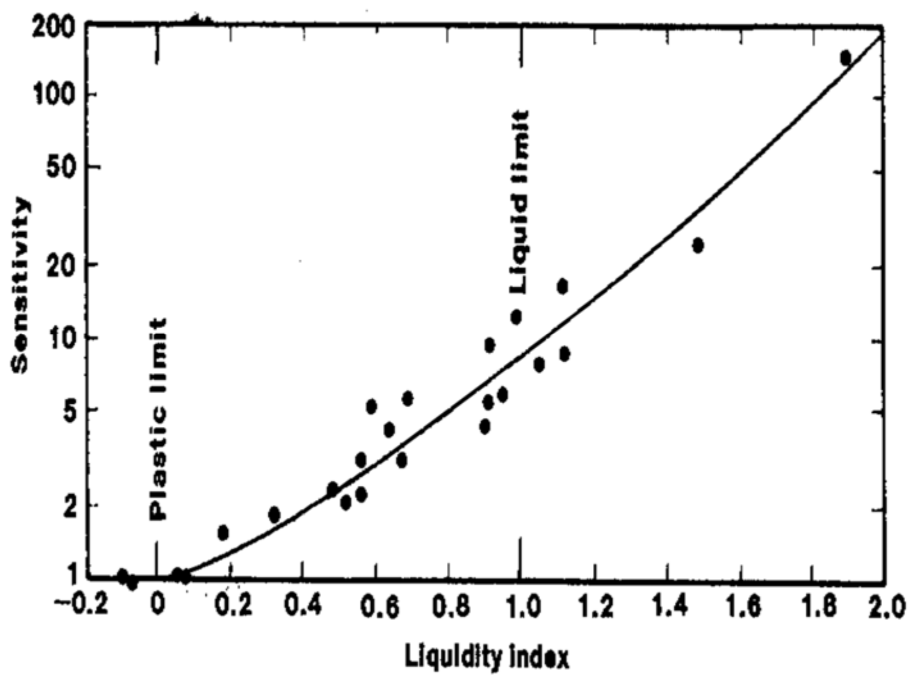

Figure 2-3 Correlation between sensitivity and liquidity index after Skempton and Northey as cited by [14]

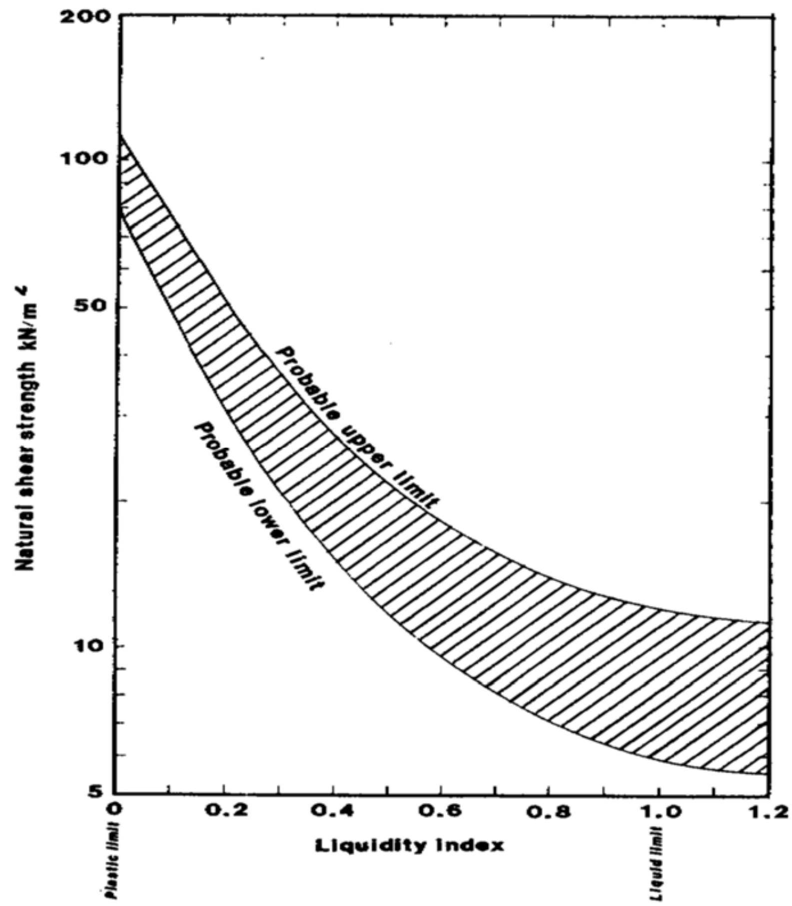

Figure 2-4 Correlation between natural shear strength and liquidity index [14]

It is also found that for most normally consolidated clays, undrained shear strength is proportional to effective overburden pressure. This is to be expected that, in terms of effective stress, shear strength is basically a frictional phenomenon and depends on confining pressure. If the constant of proportionality between shear strength and effective overburden pressure is known, then shear strength can be inferred from effective overburden pressure; that is, from depth. This problem has been investigated by a number of researchers, with a view to establishing a correlation between the shear strength/overburden pressure ratio and some soil classification parameter, typically the plasticity index. Such a correlation would be of great practical value, since it would enable the undrained shear strength to be estimated from a simple classification test.

Historically, much use has been made for normally consolidated clays of the relationship of Skempton as cited by [14]:

$$\sigma'_v = \frac{s_u}{0.11 + 0.0037PI} \quad \text{……………………………………(2.4)}$$

Where PI is the plasticity index

s_u is shear strength and

σ'_v is over burden pressure

ii. From SPT N-Value

For overconsolidated clays, Stroud as cited by Clayton [19] has reported good correlations between N and c_u. The strength of these correlation results from the standardization of the SPT in UK and the fact that the undrained shear strength was determined in a single way, using triaixal compression test on 102-mm diameter specimens.

$$c_u = f_1 N_{60} \quad \text{……………………………………………(2.5)}$$

Where c_u is undrained shear strength; N_{60} is the blow count normalised to an effective overburden pressure of 100kPa and corrected to 60% of free fall energy; f_1 is a coefficient whose values depend strictly upon the plasticity of the clay (Figure 2.5). With known plasticity index, f_1 could be read from the other axis since f_1 is equal to c_u/N.

Undrained shear strength obtained in this way will give good estimates of the mean undrained strength taking in to account fissuring. They are equivalent to values determined from 100mm diameter specimens. If the deposit is not fissured, then Equation 2.5 will under-estimate the undrained shear strength [19].

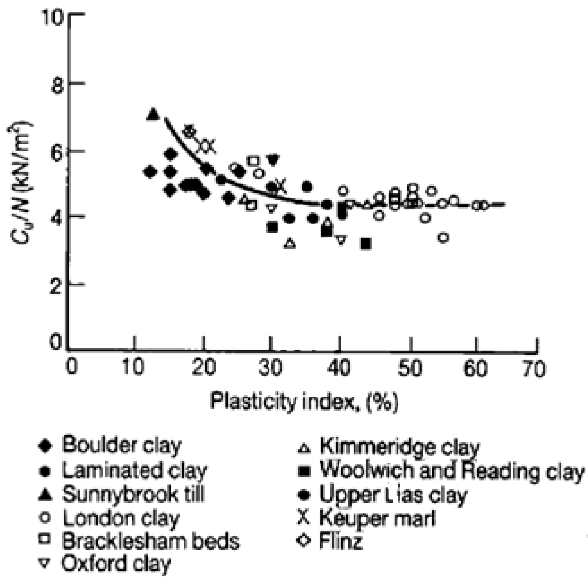

Figure 2-5 Correlation between N value and undrained shear strength for insensitive clays Stroud as cited by [8]

Many other attempts have been made to correlate the unconfined compressive strength or the undrained shear strength of clays with the results of standard penetration tests, with varying degrees of success. De Mello as cited by Carter [14] and Clayton [19] show values with c_u/N ratios apparently varying between 0.4 and 20.

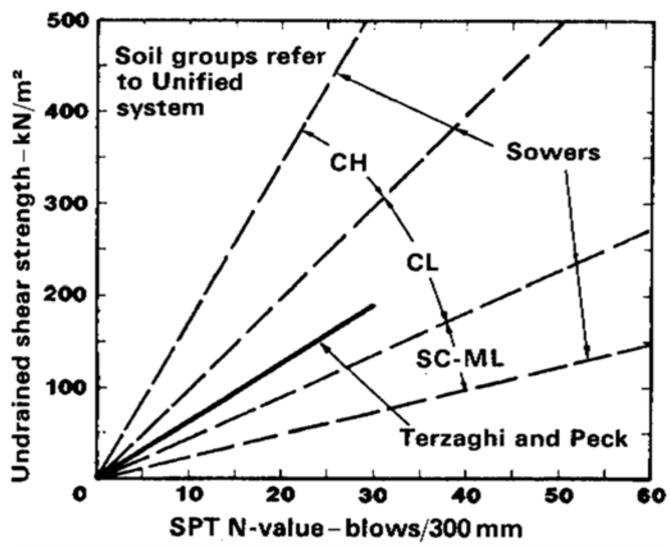

Figure 2-6 approximate correlations between undrained shear strength and SPT N-values De Mello (1971).

iii. From Simple Hand Tests

The other way is to mould a piece of clay between the fingers and applying the observations indicated in Table 2-1.

Table 2-1 Estimating the Shear Strength and SPT N-Value form consistency [9]

Consistency			N'_{70}	q_u, kPa	Remarks
Very soft	NC	Young clay	0–2	<25	Squishes between fingers when squeezed
Soft			3-5	25– 50	Very easily deformed by squeezing
Medium			6-9	50– 100	? ?
Stiff	Increasing OCR	Aged/ cemented	10–16	100– 200	Hard to deform by hand squeezing
Very stiff			17–30	200– 400	Very hard to deform by hand squeezing
Hard			>30	>400	Nearly impossible to deform by hand

2.4. DETERMINATION OF GEOTECHNICAL PARAMETERS USING DCP

2.4.1 Prediction of Undrained Shear Strength

Temnit [35] indicated that DCPI values can be correlated to the unconfined compressive strength (UCS) of Addis Ababa Red clay soil. She took undisturbed sample and performed the unconfined compressive strength test in the laboratory and made the DCP tests on the field. She had concluded that the unconfined compression strength (UCS) is highly influenced by DCPI, bulk unit weight and natural moisture content (NMC) and liquidity Index (LI).

This observation was valid only for red clay soil of Addis Ababa and have the following form:

UCS(kPa) = -115.59Ln(DCPI)mm/blow +456.41 (Liquidity Index,LI) +645.70………(2.6)

with coefficient of determination R^2=0.713, Adjusted R^2=0.676 [N=30].

2.4.2 Prediction of California bearing ratio(CBR)

Yitagesu [36] indicated that the dynamic cone penetration index (DCPI) have good correlation between unsoaked California bearing ratio (CBR) values. The test for the study were collected by conducting field and laboratory test in Jimma-Bonga road project from

Km100+200 to Km 105+050 (from Jimma town) along the road. He had concluded that good correlation does exist between the dynamic cone penetration indexes (DCPI) and unsoaked California bearing ratio (CBR) values. However, care should have to be made while using the formula as the CBR values obtained from the correlation indicates the in situ CBR value at the time of testing rather than the CBR values at the worst condition (soaked CBR).

The correlation developed in the study was developed for locally used subgrade soils. The applicability of the DCP-CBR relation proposed on the study for other pavement layer; sub base and base course should be investigated and the correlation have the following form:

$\log(CBR) = 2.954 - 1.496\log(DCPI)$ ………(2.7) With R^2=0.943.

2.4.3 Prediction of SPT N-value

Currently, the most widely used correlation between DCP and SPT N-value is performed by Transport Road Research Laboratory (TRRL), Overseas Road Note (ORN) 9, Design of small bridges [17] (See Table 2-2).

Table 2-2 Typical correlation between DCP and SPT values [17]

DCP value mm/blow	SPT N value blows/300mm
5	50
6	44
7	38
8	33
9	28
10	24
12	22
14	18
16	16
18	15
20	14

2.5 BEARING CAPACITY THEORY

An important problem of foundation engineering is the computation of the maximum load (the bearing capacity) and Coulomb's method is used for the analysis of soil pressures in which the soil is on the verge of failure. This type of analysis can be given a firm theoretical basis by the theory of plasticity [25].

For purely cohesive soil ($\phi = 0$), the spiral becomes a circular arc, and Prandtl's analysis give the following equation for the ultimate bearing capacity,

$$q_{ult} = (\pi + 2)C = 5.14c \ldots\ldots (2.8)$$

In this theory, the material is considered to be weightless ($\gamma = 0$), and frictionless ($\phi = 0$), so that its only relevant property that is considered is the cohesive strength c. After this finding, many others incorporated the influence of the depth of the foundation and other parameters in to the equation. The above formula has been extended by Keverling Buisman, Caquot, Terzaghi and Brinch Hansen with various terms, including one for the unit weight of the soil. The complete formula is written in the form [25]

$$q_{ult} = cN_c + qN_q + 0.5B\gamma N_\gamma \ldots\ldots (2.9)$$

Where

$$N_q = \frac{1 + \sin\phi}{1 - \sin\phi} \exp(\pi \tan\phi -) \ldots\ldots (2.10)$$

$$N_c = (N_q - 1)\cot\phi \ldots\ldots (2.11)$$

B is the total width of the loaded strip. For the coefficient N_γ, various suggestions have been made on the basis of theoretical analysis or experimental evidence or depending on the safety needed, for instance [25]

$$N_\gamma = 2(N_q - 1)\tan\phi \ldots\ldots (2.12)$$

or

$$N_\gamma = 1.5(N_q - 1)\tan\phi \ldots\ldots (2.13)$$

Even though the values of N_c, N_q and N_γ are given, as a function of the friction angle ϕ, since we are considering the limiting case $\phi = 0$, the value of $N_c = 2 + \pi = 5.142$, $N_q = 1$ and $N_\gamma = 0$ [25]. The following ultimate bearing capacity equation is used for the current thesis:

$$q_{ult} = 5.142c + \gamma D \quad (2.14)$$

Where, q_{ult} = the ultimate bearing capacity,

c = undrained shear strength (c_u)

D = depth of footing

γ = unit weight of the soil

3. DESCRIPTION OF THE STUDY AREA

3.1 GENERAL

Debre Markos with twelve Kebeles and one Woreda is one of the oldest historical medium towns of Ethiopia. It is found 300 kilometers Northwest of Addis Ababa and 265 kilometres Southeast of the Amhara National Regional State capital city-Bahir Dar. The geographical coordinates of the town are 10^{o} 21^{1} latitude north and $37^{o}43^{1}$ longitude east. Situated at 2420 meters above sea level, the weather condition, most of the time is, 'Woinadega'. The town enjoys a tropical climate with a mean annual rainfall of 1308 mm, temperature 16^{o}c, while the maximum and minimum recorded temperature being 24^{o}c and 4^{o}c respectively. [34]

3.2. HYDRLOGY

3.2.1. Climate

The mean annual rainfall of Debre Markos town is 1308 mm. The main rainy season is from June to October and most of the annual rainfall is precipitating in these four months' time. The second annual rainfall precipitates from March to May in three months' time. The remaining months are relatively dry (Figure 3.2) [34].

The maximum monthly temperature is recorded from February to May which is more than 25^{o}C and the lowest maximum monthly average temperature is recorded June to October which is 17 ^{o}C to 21 ^{o}C (Figure 3.3) [34].

3.2.2. Geomorphology

Debre Markos town is located on a plateau of north-western highlands of Ethiopia. The plateau where the town is built is surrounded by Abay River's deep gorge on the eastern and southern sides and western lowlands on the western side. The general elevation of the region is over 2,000 m.a.s.l. and Choke volcanic mountain that rises to over 4,000 m.a.s.l. is situated on the plateau.

The surroundings of Debre Markos town are undulating hills and valleys but between the hills there are wide marshy plane areas. Some of these planes are over 10 km. across. These plains form swamps and water logged areas during the rainy months [34].

Fig 3.1 Location of the research area on the map of Ethiopia

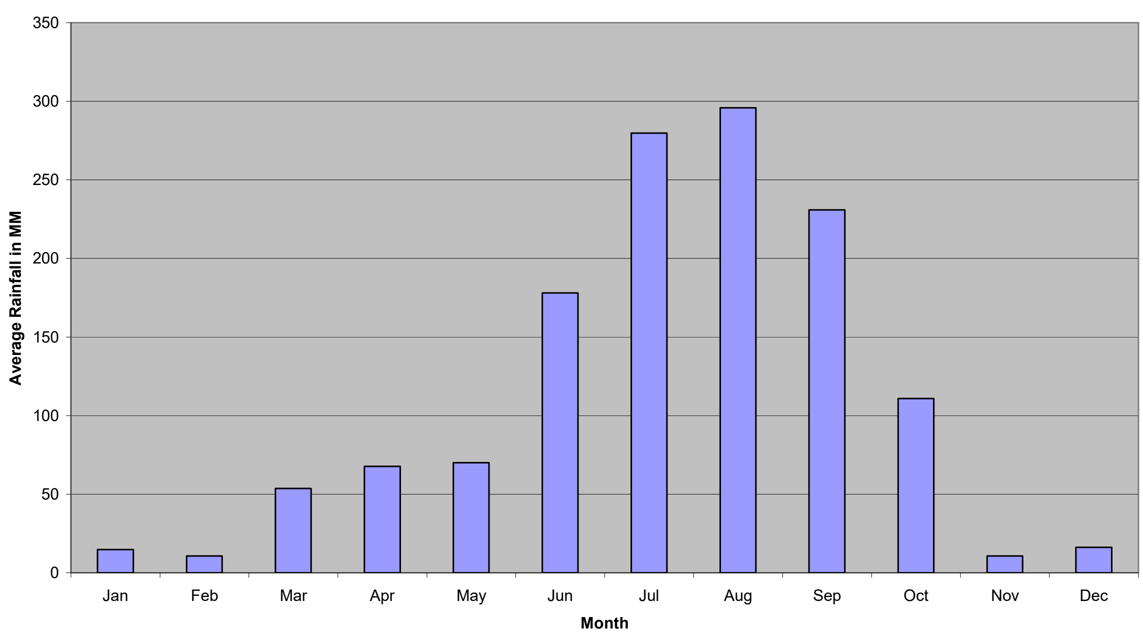

Figure 3.2 Average annual rain fall of Debre Markos 1998-2007 E.C (Source Ethiopian Metrological Agency)

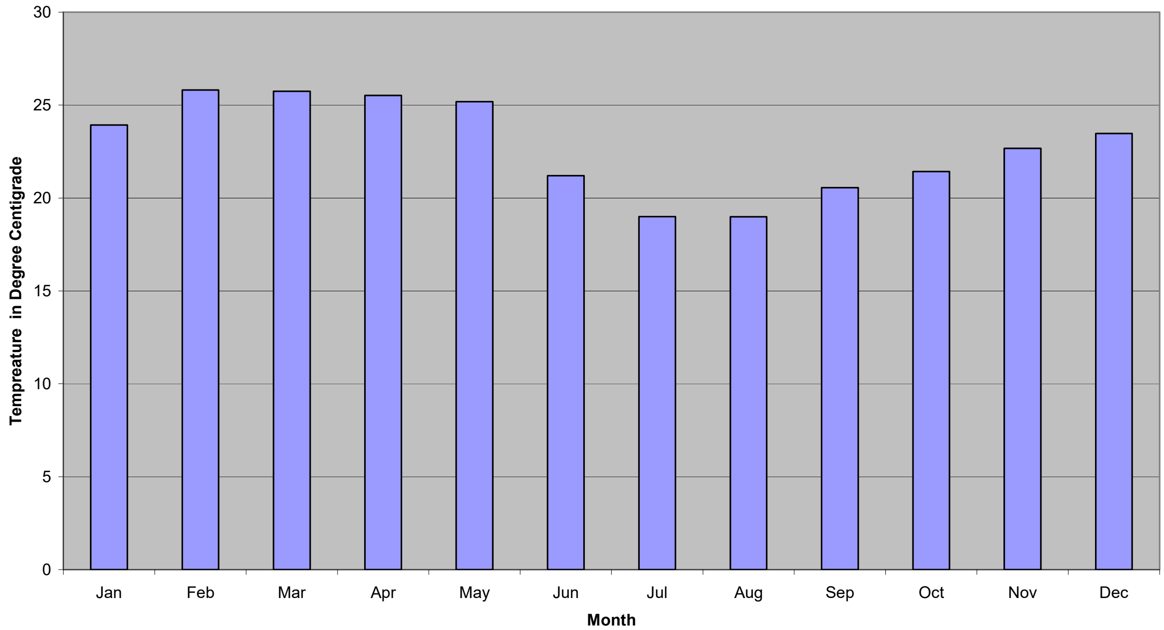

Figure 3.3 Average annual temperature of Debre Markos town 1999-2007 E.C (Source Ethiopian Metrological Agency)

4. METHODS, DATA COLLECTION AND RESULT

4.1 GENERAL

The subsurface strata encountered in the test pits, test methods used and other supplementary information is provided in this section. Both disturbed and undisturbed soil samples were taken to the laboratory as per the procedure stated in Table 4-1. Other tests were also done according to the methods and standards stated in section 4.2.

4.2 TEST METHODS

4.2.1 Field Tests and Sampling Methods

Table 4-1 Field test and sampling methods used [22 and 24]

Test Description	Test Procedure
Dynamic Cone Penetration test	Minnesota Department of Transportation, "User Guide to the Dynamic Cone penetrometer," office of Material and Road Research, 1998[24]/ASTM D 6951-03.
Soil sampling	Representative disturbed soil samples were collected using Sampling tube from the different layers of test pits, individually, for classification tests (refer to ASTM D 4220). Undisturbed samples were collected for unconfined compressive strength, bulk density and in situ moisture content tests (refer to ASTM D 1587).

4.2.2 Laboratory test methods

Table 4-2 Laboratory test description and sampling methods used [1, 13 and 22]

Test Descriptions	Test Procedure
Grain size analysis	ASTM D 422
Moisture content of soil	ASTM D 2216
Unit weight of soil	ASTM D 1188 and D2216
Specific Gravity of soil	ASTM D 854
Atterberg Limits	ASTM D 4318/ AASHTO 89-90/BS for soil of low plasticity.
Unconfined compressive strength of cohesive soil	ASTM D 2166
Free swell test	According to Gibbs and Holtz, 1956 [1]
Classification System	ASTM D-2487

4.3 DATA COLLECTION AND TEST RESULTS

Before selecting sampling area Visual investigation, information from local people and construction firm were collected and 17 test pits are excavated and 33 disturbed and undisturbed samples are taken. During laboratory test, from the three test pits (Filklik, Dinbza high school 1.5m and Silass 2m) is difficult to take undisturbed sample in the laboratory for the unconfined compressive strength test so they are not taken for the correlation purpose. Depth of excavation was dependent on the existing condition of the ground. In some places, boulder was encountered and in others the ground water table was very near to the surface. On average, samples were generally taken from depth of 1m to 3m.

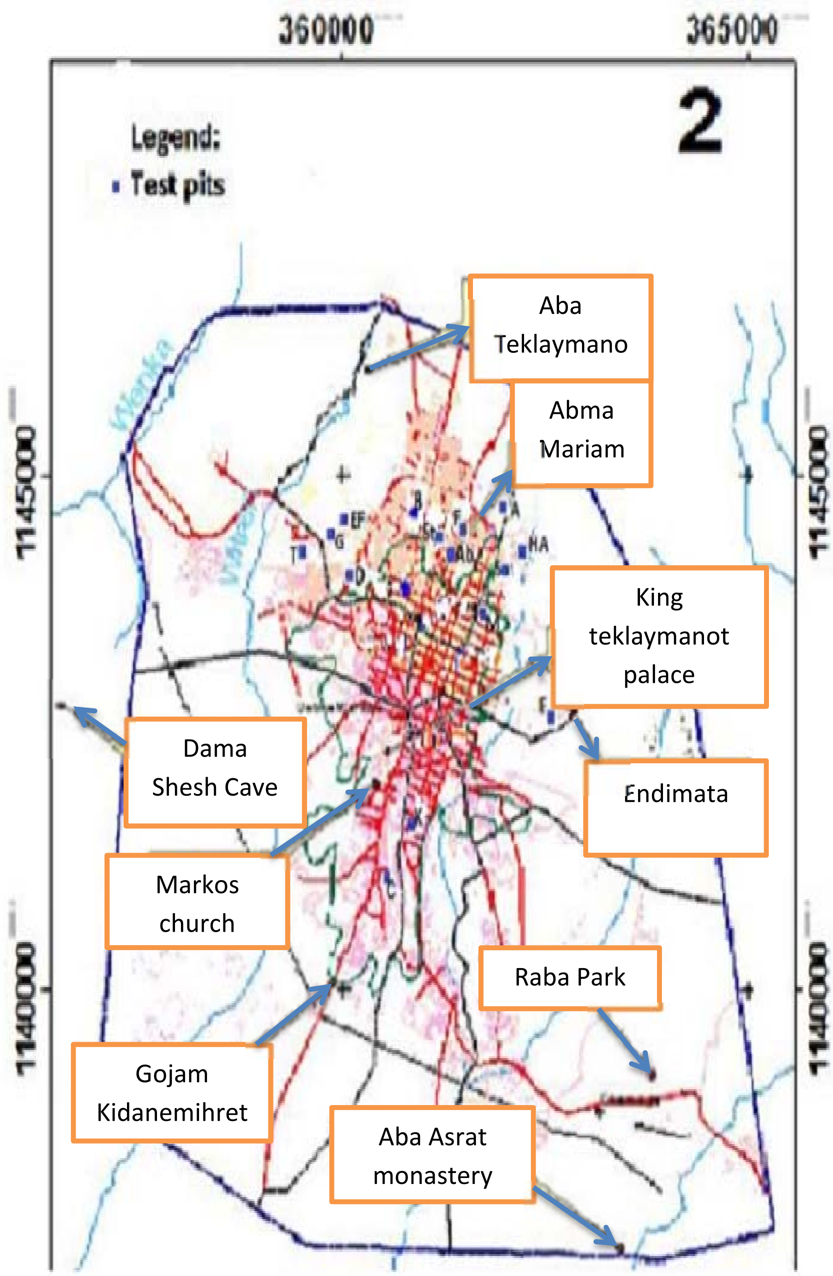

Figure 4-1 Locations of data collection in Debre Markos [34]

Based on the free swell values and soil classifications of the research area two categories are made for the analysis and correlation development purpose

Category-1) Red clay soils of Debre Markos and

Category-2) Black Clayey (expansive) soils of Debre Markos

As part of the DCP procedure, the first few drops were used as seating blows, since the soil is less confined near the surface and the DCP is able to penetrate further per drop thus making the initial drops unreliable. It should be noted that DCP should be taken out of the newly formed test pit. The number of lifts and drops of the hammer before each penetration reading is taken depends upon the strength of the soil at that test location [32].The Penetration data obtained in the field was conducted using the procedure stated in Table 4.1. The data was analysed using simple software called UK DCP [Version 3.1].Using this software one can input depth of penetration and number of blows to get penetration index and layers of in-situ characteristics in an enhanced way. The details of field penetration data are presented in Appendix A.The summery of all the field and laboratory data used are presented in Table 4-3.

4.4. SUMMARY OF TEST RESULTS

4.4.1 General

Based on USCS twenty (20) soil samples are classified as inorganic SILT with high plasticity (MH) and eight (8) samples are classified as inorganic CLAY with high plasticity (CH) (Refer to Figurer 4-2).

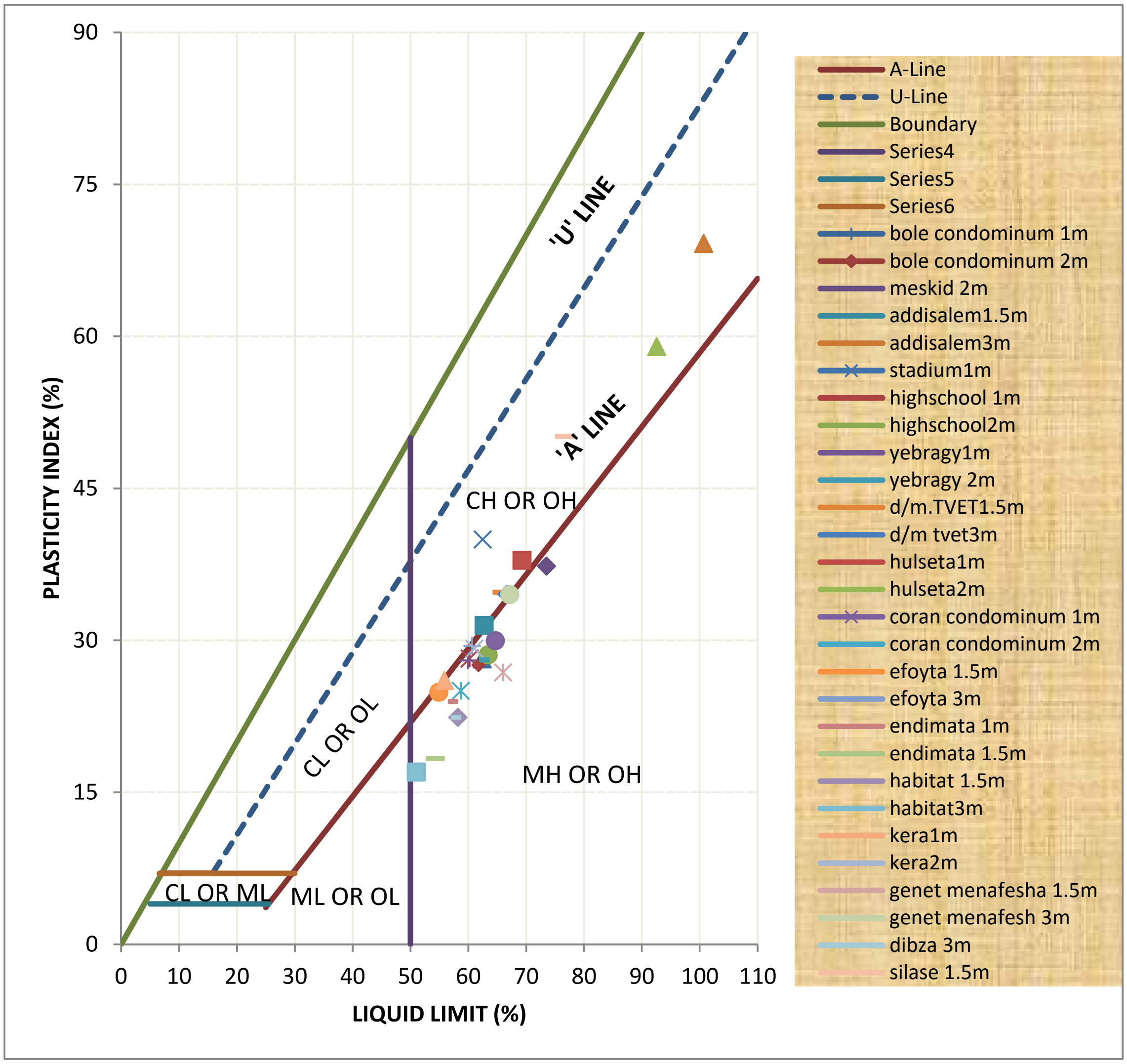

Figure 4-2 Soil classification based on USCS

Grain size analysis and other index property tests were also conducted on twenty eight (28) representative samples (i.e. twenty (20) red clay soils and eight (8) black cotton soils) in the current research. The plot of soil gradations conducted in the current research are presented in Figure 4-3 and Figure 4-4.

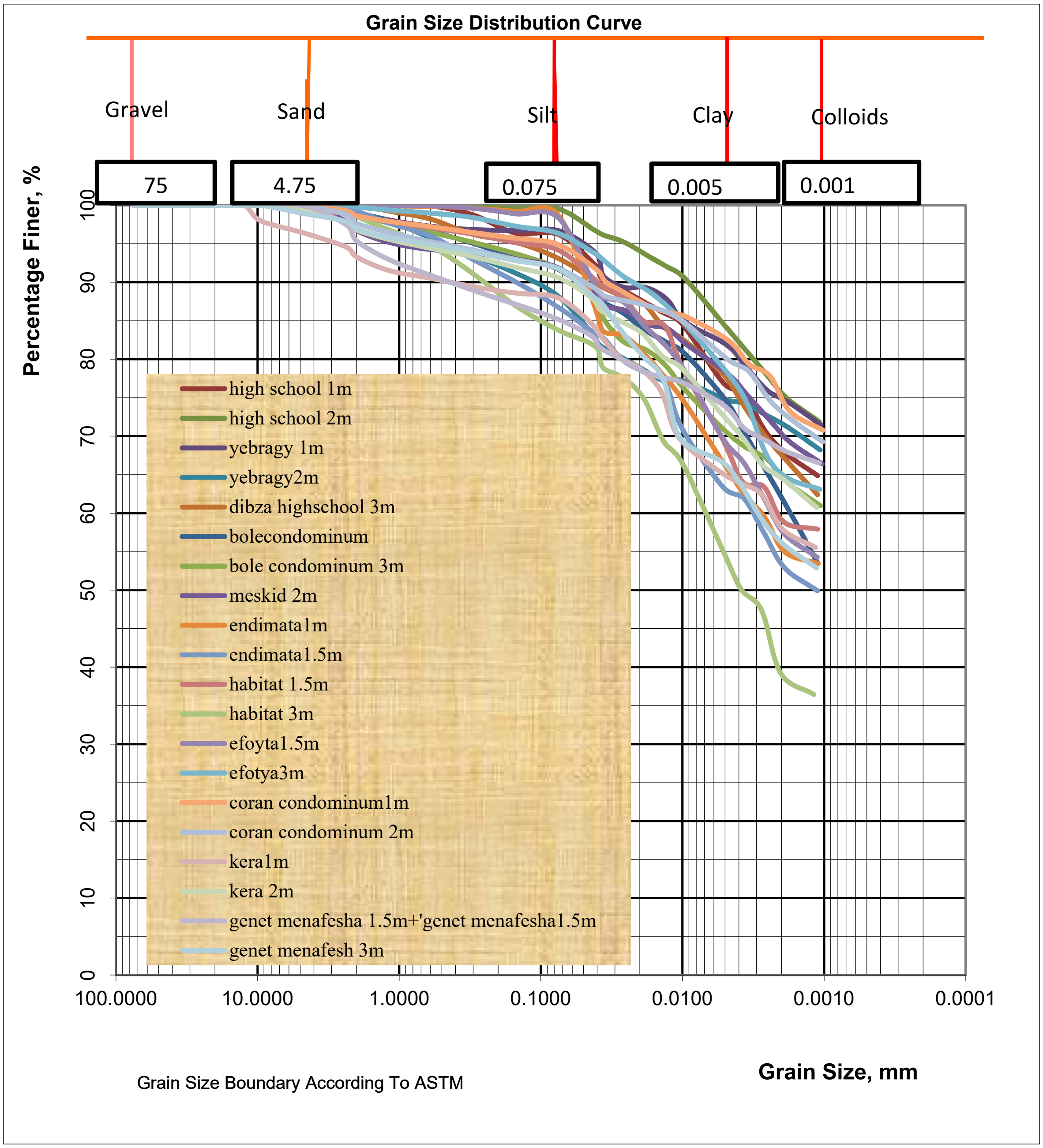

Figure 4-3 Grain size analysis for Red clay (Non-expansive) soil

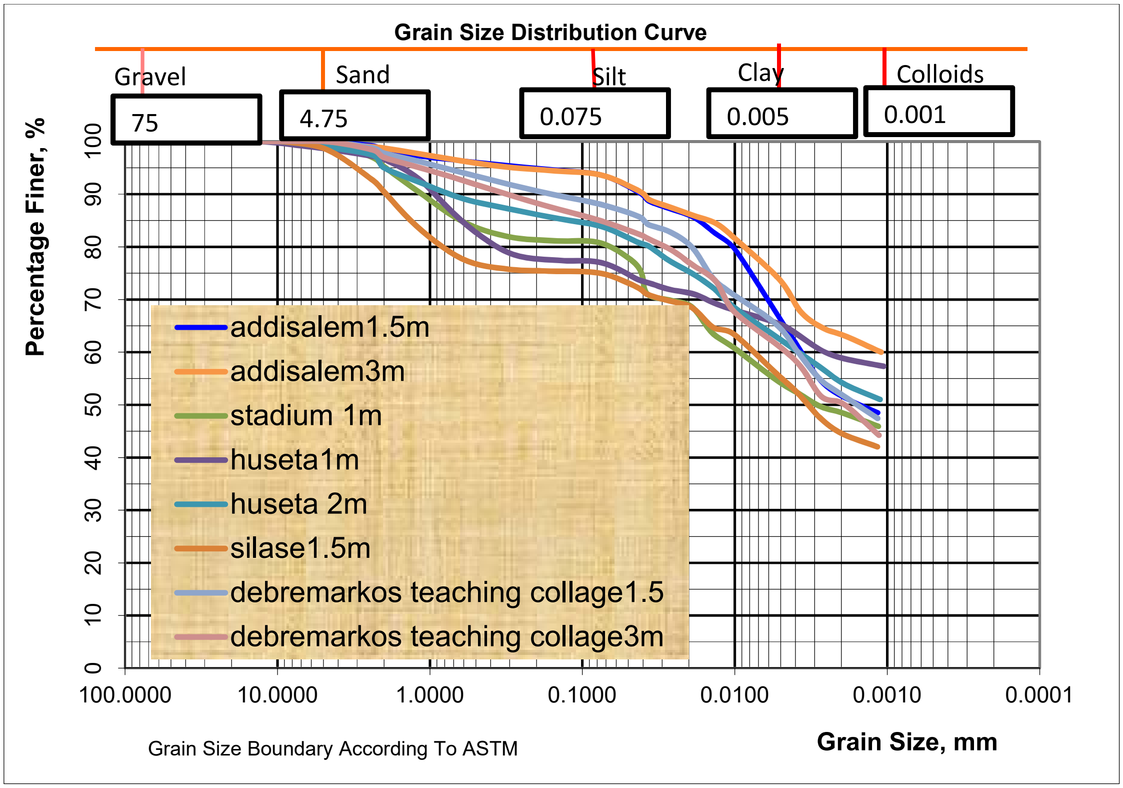

Figure 4-4 Grain size analysis curve for black cotton (expansive) soil

4.4.2 SUMMERY OF TEST RESULTS

Table 4-3 Summary of test results

Site designation	Depth (m)	NMC (%)	Gs	C (%)	LL (%)	PL (%)	PI (%)	Liquidity index =[wn-wp]/PI	FS (%)	γ_{bulk} (KN/m^3)	UCS (KN/m^2)	DCPI (mm/blow)	USCS
01) Bole condominum_[b1]	1	35.8	2.82	74.71	62.37	34.19	28.18	0.06	45	17.73	147.5	30.33	MH
02) Bole condominum_[b2]	2	35.5	2.81	70.69	61.76	34.38	27.38	0.04	40	17.36	158.1	20.6	MH
03)Meskid)_[M2]	2	32.6	2.87	78.1	73.50	36.17	39.34	-0.09	35.00	17.33	448.50	8.08	MH
04)Addisalem_[A1]*	1.5	35.3	2.77	63.98	62.72	31.24	31.48	0.13	58	17.27	176.4	38	CH
05) Addisalem_[A2]*	3	49.1	2.81	73.07	100.68	31.5	69.18	0.25	110	16.75	50.5	45	CH
06) Stadium_[S2]*	1	29.2	2.77	53.74	62.47	22.5	39.97	0.17	83	15.48	76.5	47.9	CH
07)Highschool_[H1]	1	35	2.76	75.5	60.16	31.88	28.28	0.11	40	17.41	79.18	28.75	MH
08) Highschool_[H2]	2	38.5	2.77	81.84	63.42	34.85	28.75	0.13	45	16.11	62.4	32	MH
09) Yebragey_[Y1]	1	34.4	2.85	78.3	59.91	31.88	28.02	0.09	45	17.30	92.08	25.4	MH
10) Yebragey_[Y2]	2	34.3	2.82	84.28	62.07	33.99	28.8	0.01	48	17.41	164.8	15.36	MH
11)TVET collage_[T1]*	1.5	21.4	2.84	62.76	65.81	31.07	34.74	-0.28	60	17.12	232.9	20.8	CH
12)TVET collage_[T2]*	3	30.5	2.78	61.23	66.63	32.07	34.57	-0.05	68	18.02	192.92	35	CH
13) Huseta_[H1]*	1	36.9	2.81	62.5	69.27	31.36	37.91	0.15	50	18.25	126.6	40.38	CH
14) Huseta_[H2]*	2	49.6	2.84	62.6	92.54	33.54	59	0.27	105	16.40	44.4	49.5	CH
15) Coran condominium)_[C1]	1	36.5	2.84	82.4	64.67	34.7	29.97	0.06	37	16.90	146.4	32.8	MH
16) Coran condominum_[C2]	2	35.7	2.84	80.55	58.69	33.67	25.02	0.08	30	16.25	103.7	22.6	MH
17) Efoyta_[E1]	1.5	34.9	2.79	69.6	61.88	33.33	28.58	0.05	35	17.42	147	27.14	MH
18) Efoyta_[E2]	3	32.6	2.82	78.35	60.66	31.2	29.46	0.05	40	17.71	162.77	16	MH
19) Endimata_[EN1]	1	36.3	2.89	66.44	56.62	32.66	23.96	0.15	30	16.14	52.2	35.64	MH
20) Endimata_[EN2]	1.5	40.8	2.89	63.38	54.27	35.94	18.33	0.27	25	16.90	44.9	51.2	MH
21) Habitat_[HA1]	1.5	35.2	2.76	69.57	58.18	35.77	22.41	-0.03	35	17.02	222.9	11.71	MH
22) Habitat_[HA2]	3	30.2	2.85	54.18	50.99	33.98	17.01	-0.22	34	17.22	305.5	8.35	MH
23) Kera_[K1]	1	32.3	2.77	60.44	55.86	29.81	26.06	0.10	40	17.15	90.56	27.08	MH
24) Kera_[K2]	2	35.4	2.79	70.35	61.03	31.85	29.17	0.12	45	17.94	73.61	30.71	MH
25)Genet menafesha_[G1]	1.5	37.1	2.81	52.4	65	37.41	27	-0.01	33	15.94	184	14.41	MH
26)Genet menafesha_[G2]	3	36	2.84	66.1	69	35.38	34	0.02	50	15.39	161.8	17.73	MH
27)dinbza highschol_[D1]	3	26.6	2.76	76.6	53.04	32.18	20.86	-0.27	40	16.71	552.4	6.25	MH
28)Silase_[S1]*	1.5	25.2	2.74	51.89	76.65	26.51	50.15	-0.03	80	19.95	187.2	37	CH

* Black cotton (expansive) clayey soils

5. ANALYSIS

5.1 GENERAL

Regression analysis is concerned with the procedure how the values of Y depend on the corresponding values of X. Y, whose value is to be predicted, is known as dependent variable and X, which is used in predicting the value of dependent variable, is called independent variable. A regression model that contains more than one independent variable is called multiple regression models. Alternatively, regression model containing one independent variable is termed as simple regression model.

Fitting a regression model requires several assumptions. Estimation of the model parameters requires the assumption that, the residuals (actual values less estimated values) corresponding to different observation are uncorrelated random variables with zero mean and constant variance. Test of hypothesis and interval estimation requires that the error be normally distributed. In addition, one assumes that the order of model is correct*;* that is, if one fits a simple linear regression model, one is assuming that the phenomenon actually behaves in a linear or first order manner. During regression analysis, a regression model with higher value coefficient of determination (R^2), which quantifies the proportion of the variance of one variable by the other, is accepted [27].

In this thesis two sets of investigations have been conducted. The first set considers UCS as the dependent variable where as DCPI and LI are independent variables. The second set considers DCPI as the dependent variable and the independent parameters employed for the investigation of UCS are used. To carry out statistical analysis, Microsoft® excel was used for single regression with both linear and non-linear functions whereas SPSS was use for multiple regression. Different models are used and those models with a higher value of coefficient of determination are accepted. Variable numbers of samples are used in correlating the different parameters. So, coefficients of determinations encountered cannot be simply described in narrative terms due to the fact that correlations between different parameters varied from correlation to correlation. The statistical significance of correlation is a function of the number of datasets being analysed. As a result, when a parameter's correlation is described as "good", "fair" or "poor" in later discussions, the description is given for the relation being discussed.

The parameters considered as principal component of analysis is shown in Table 4-3 included unconfined compressive strength, dynamic cone penetration index, dry unit weight, natural water content and liquidity index. Other parameters are selected by considering their effect on unconfined compressive strength and DCP values. For example the unconfined compressive strength of cohesive soils are affected the liquidity index of the soil.

5.2. SINGLE REGRATION

5.2.1 Scatter Plot for Category-1 (Red clay soils of Debre Markos)

In developing correlations, the first step is creating a scatter plot of the data, to visually assess the strength and form of the relationship. In the figures below (Figure 5-1 to 5-3) the scatter plot of UCS with DCPI and LI are presented.

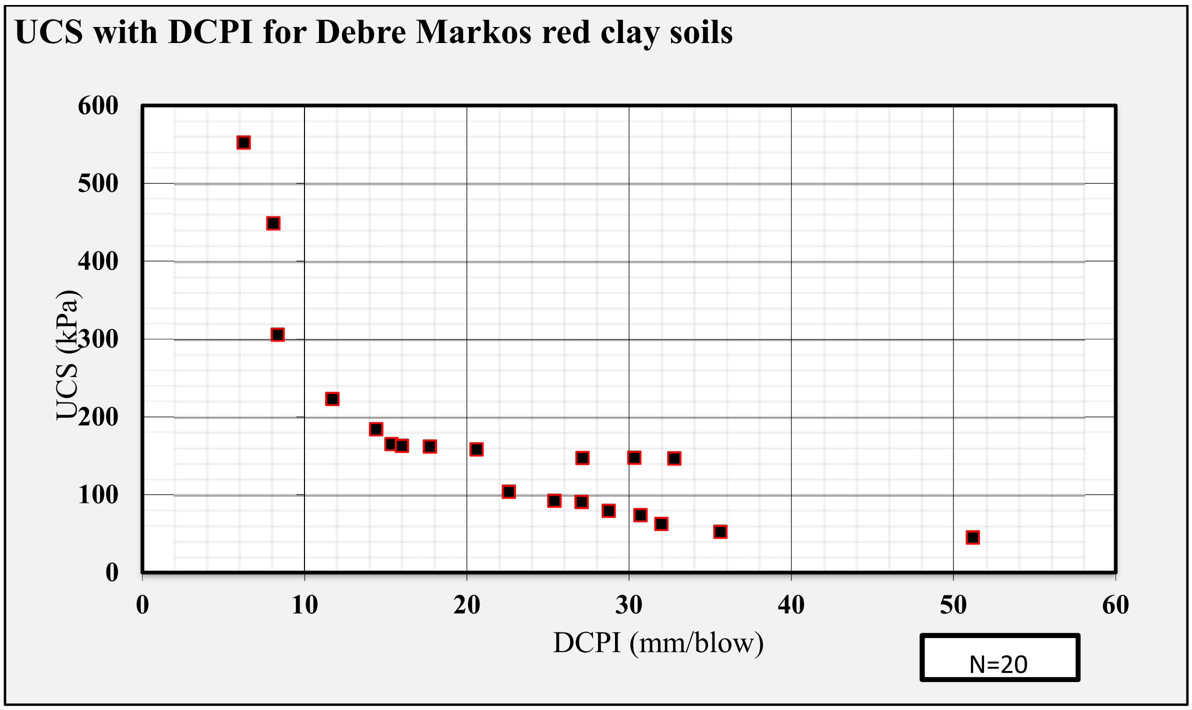

Figure 5-2 Scatter plot of UCS with DCPI for red clayey soils of Debre Markos

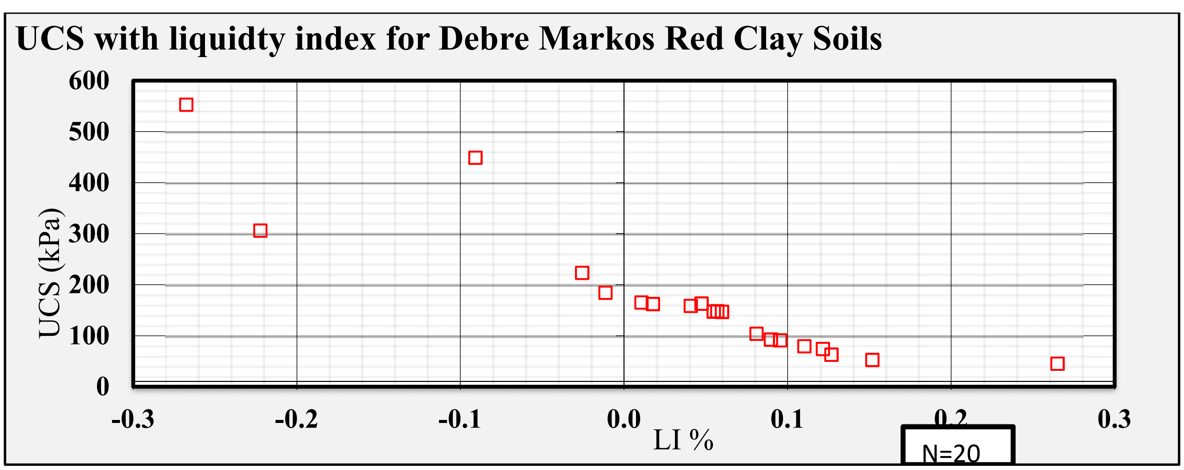

Figure 5-2 Scatter plot of UCS with LI for red clayey soils of Debre Markos

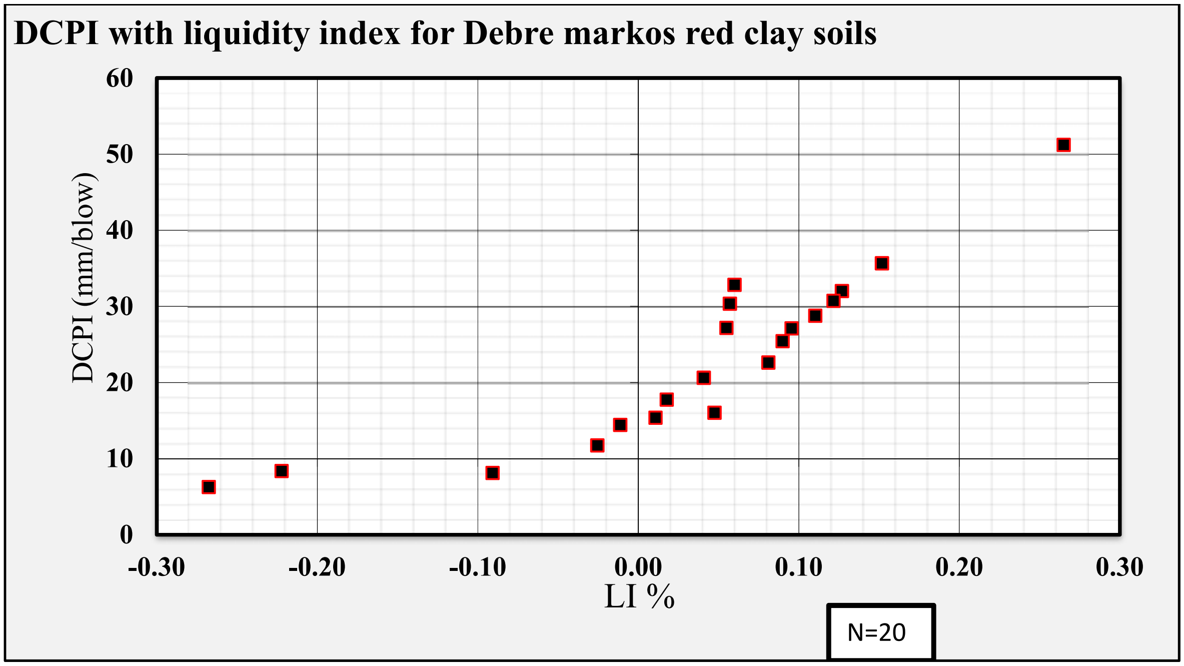

Figure 5-3 Scatter plot of DCPI with LI for red clayey soils of Debre Markos

5.2.2 Scatter Plot for Category-2 (Black clay soils of Debre Markos)

Similar to 5.2.1 the scatter plots of UCS with DCPI and LI are presented for black clay soils of Debre Markos (Figure 5-4 – 5-6).

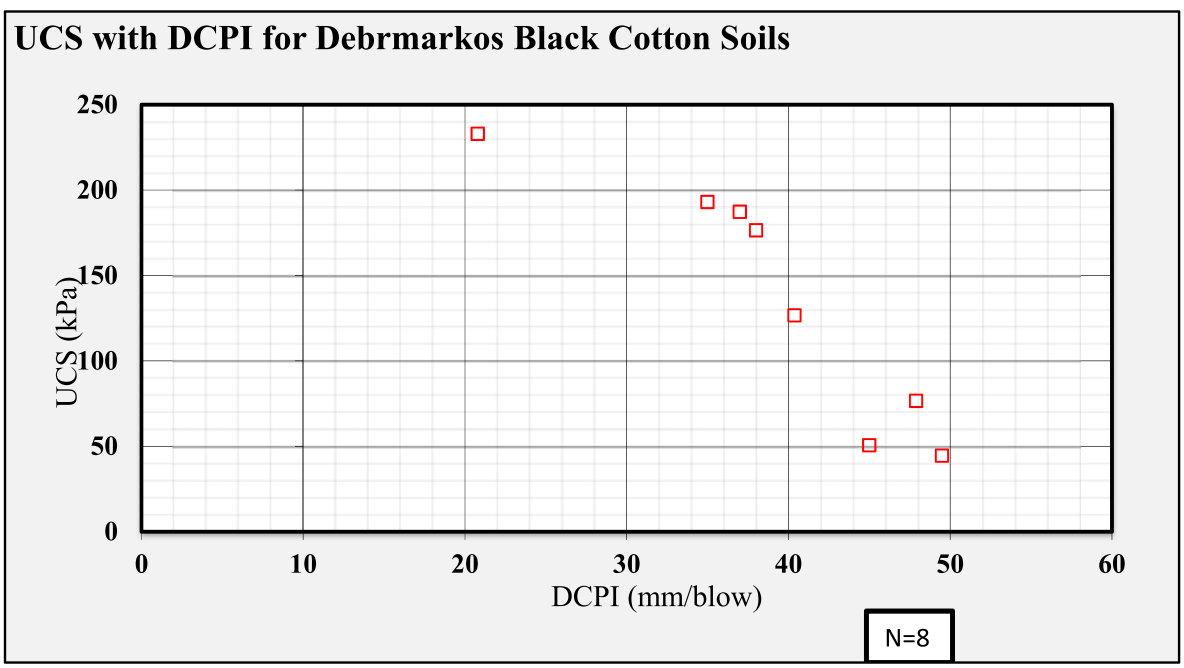

Figure 5-4 Scatter plot of UCS with DCPI for black clayey soils of Debre Markos

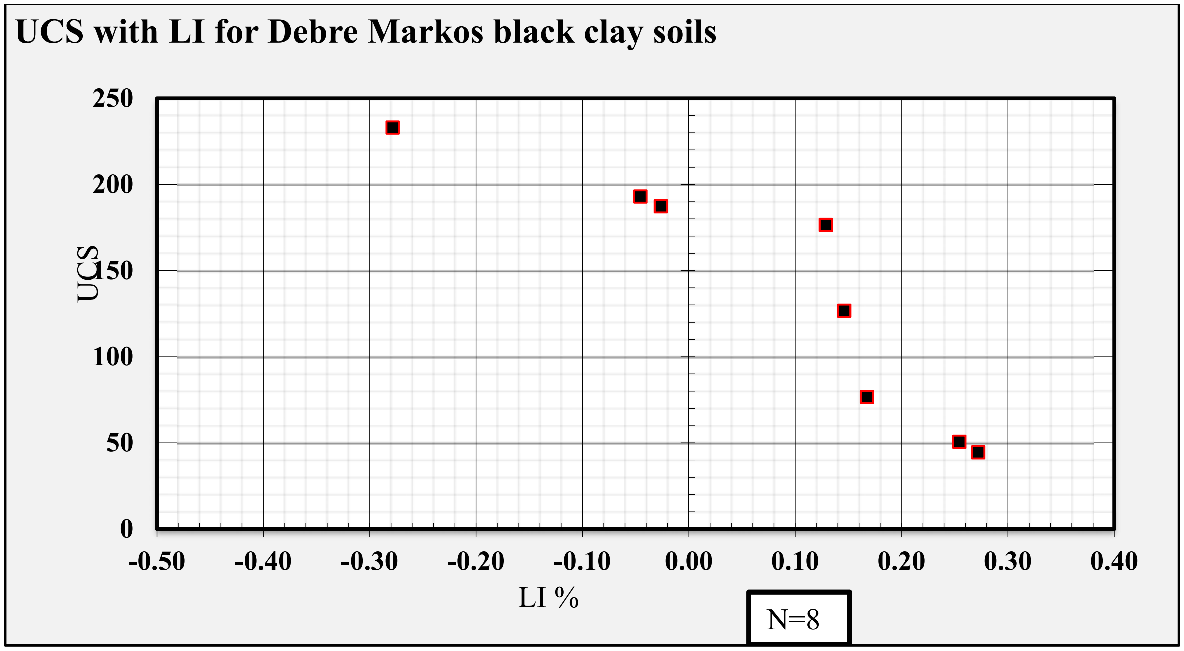

Figure 5-5 Scatter plot of UCS with LI for black clayey soils of Debre Markos

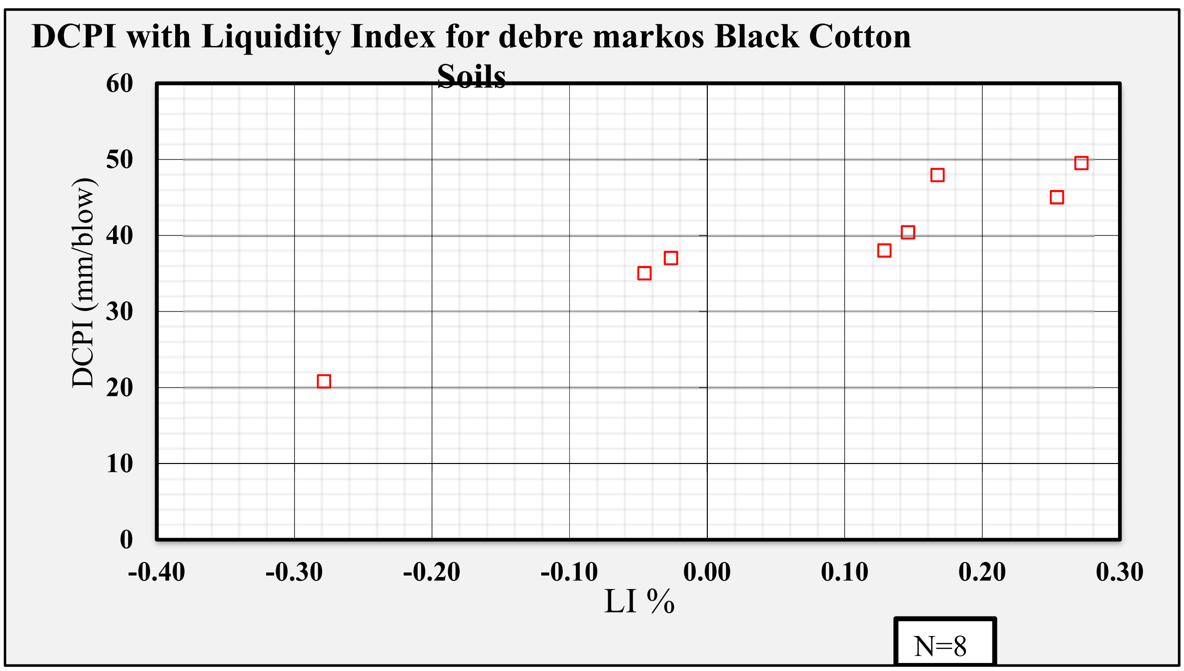

Figure 5-6 Scatter plot of DCPI with LI for clayey soils of Debre Markos

5.2.3 Summary of Correlations for Category-1 (Red Clay Soils of Debre Markos)

After carefully studying the data trend, correlations were developed for this category. The summary of the correlations is presented in Tables 5-1.

Table 5-1 Summary of Correlations for Category-1(Red Clay Soils of Debre Markos)

Equation	R^2	Sample Size
UCS= -209.51ln(DCPI)+800.5	0.8019	20
UCS =-970.23(LI)+204.69	0.8144	20
DCPI=81.415(LI)+20.197	0.7776	20

5.2.4 Summary of Correlations for Category-2 (Black Clay Soils of Debre Markos)

After carefully studying the data trend, correlations were developed for this category. However the number of sample size is to meagre hence it is important to use the equation with caution. The summary of the correlations is presented in Tables 5-2

Table 5-2 Summary of Correlations for Category-2 (Black Clay Soils of Debre Markos)

Equation	R^2	Sample Size
UCS= -7.1661(DCPI)+416.82	0.8215	8
UCS =-352.8(LI)+163.25	0.8152	8
DCPI =46.915(LI)+35.565	0.9011	8

5.3. MULTIPLE REGRESSION

Multiple regression attempts to model the relationship between two or more explanatory variables and a response variable by fitting an equation to observed data. Every value of the independent variable x is associated with a value of the dependent variable y.To examine the combined effect of some index property on UCS and also DCPI, a multiple regression analysis is conducted. The basic form of the equation is as follows:

$$y = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + \ldots + \beta_k x_k + \varepsilon \quad \text{...} (5.1)$$

The single regression discussed previously (i.e., the regression between UCS, DCPI and LI) had shown that the undrained shear strength is significantly affected by parameters like dynamic cone penetration index (DCPI) and Liquidity index.

The adjusted coefficient of determination mentioned in the following sections of the multiple regressions describes the amount of variance in Y which could be explained by the regression equation.

5.3.1 Multiple Regression for Category-1 (red clay soils of Debre Markos)

The developed equation for multiple regression of UCS (kPa) with DCPI (mm/blow) and LI for red clay soils, with N=20 and adjusted R^2=81.6% is

UCS = -96.408ln (DCPI) -552.123*(LI) +480.035 …………………… (5.2)

5.3.2 Multiple Regression for Category-2 (Black clay Soils of Debre Markos)

The developed equation for multiple regression of UCS (kPa) with DCPI (mm/blow) and LI for black cotton (expansive) soil, with N=8 and adjusted R^2=77.4 is

UCS = -4.054*(DCPI) -161.541*(LI)+307.147 ………………………(5.3)

Since the number of soil sample used for the regression is too small it is suggested to use the above formula with caution.

6. DISCUSSIONS

6.1 GENERAL

Based on USCS twenty (20) soil samples are classified as inorganic SILT with high plasticity (MH).However as one can see from Table 4.3 the clay fraction of the soil ranges from 51% up to 83% which indicate the principal component of the soil is clay soil and the less prominent soil is silt soil so that the soil should be classified as either silty clay soil or clay soil because USCS is constructed by concerning the temperate zone soil.The other eight (8) samples are classified as inorganic CLAY with high plasticity (CH).

The index properties of Debre Markos red clay soils from current research and previous researches are presented in Table 6-1 for comparison purpose.

Table 6-1 Index properties of Debre Markos soil from previous and current researches

Researches	Site	Clay Content (%)	LL (%)	PI (%)	Gs
Adem Ebrahim[34]	Debre Markos soil	50-72	45-68	14-40	2.69-2.84
Current Research	Debre Markos soil	52.4-84.28	50.99-73.50	17.01-39	2.76-2.89

From Table 6-1 one can see that the test results of the current research is nearly Similar to the previously conducted research.

6.2. REGRESSION

6.2.1. CATEGORY-1 (RED Clayey SOILS OF DEBRE MARKOS)

❖ **Simple regression**

- After carefully studying the data trend, this category show that unconfined compression strength is significantly influenced by dynamic cone penetration index and liquidity index by achieving coefficient of determination of 80.19% and 81.44% respectively. The summery of the correlations is presented in Tables 5-2.
- The dynamic cone penetration index is also significantly influenced by liquidity index in addition to undrained shear strength by achieving coefficient of determination of 77.76% and 80.19% respectively. The summery of the correlations is presented in Tables 5-2.

❖ **Multiple regression**

- The multiple regression of UCS with DCPI and LI indicate that UCS has good correlation with the parameters by achieving adjusted coefficient of determination of 81.6% with 20 samples (refer to Equation 5-2).

6.2.2. CATEGORY-2 (BLACK CLAYEY SOILS OF DEBRE MARKOS)

❖ **Simple regression**

- After carefully studying the data trend ,this category show that unconfined compression strength is significantly influenced by dynamic cone penetration index and Liquidity index by achieving coefficient of determination of 82.15% and 81.52% respectively. The summary of the correlations is presented in Tables 5-3.
- The dynamic cone penetration index is also significantly influenced by liquidity index in addition to undrained shear strength by achieving coefficient of determination of 77.76% and 80.19% respectively. The summery of the correlations is presented in Tables 5-3.

- **Multiple regression**

  - The multiple regression of UCS with DCPI and LI indicated that UCS has good correlation with the parameters by achieving adjusted coefficient of determination of 77.4% with 8 samples (refer to Equation 5.3).

6.3. DEVELOPMENT OF EQUATION BASED ON BEARING CAPACITY THEOREY

Some good correlations between undrained shear strength and the DCP penetration index for the two categories are obtained (refer to Table 5-2 and Table 5-3 respectively). It is tempting to develop a bearing capacity equation from the correlations developed.

Converting unconfined compression strength into cohesion (i.e., c=UCS/2) is applicable for saturated soils. Since the soil in the current research is unsaturated, the reader should understand that this equation can only give an approximate estimate of the cohesion for this type of soils.

We have observed a correlation between UCS (kPa) and DCPI (mm/blow) gave:

UCS = -209.5*ln(DCPI)+800.5, R^2=80.19% and N=20, for red clay soils of Debre Markos....(6.1)

UCS = -7.1661*(DCPI)+416.82, R^2=82.15% and N=8, for black cotton soils of Debre Markos..(6.2)

Inserting equations 6.1 and 6.2 into cohesion, the corresponding relation will be:

c = -104.75*ln(DCPI)+400.25, for red clay soils of Debre Markos.……………………..………...(6.3)

c = -3.583*(DCPI)+208.41 for black cotton soils of Debre Marko…………………….……......(6.4)

After inserting equations 6.3 and 6.4 into equation 2.8, the corresponding relation of bearing capacity equation for initial loading condition will be:

q_{ult} =-538.625*ln(DCPI)+2058.086+γ*D for red clayey soils of Debre Markos…..…………(6.5)

q_{ult} = -18.423*(DCPI)+1071.64+γ*D, for black clay soils of Debre Markos…………………(6.6)

Since the number of sample size used for the regression analysis of black clay soil is too small the validity of equation 6.5 and 6.6 should be checked by conducting the test for Cu.

The objective of this thesis is to develop a simple method to predict the shear strength of clay soils without getting into tiresome and costly laboratory tests. It has been shown that introduction of parameters like LI in to equation between UCS and DCPI will improve the prediction of shear strength. If the above mentioned parameters are available, it is advisable to use the multiple regression developed equation for clay soils.

6.4. VALIDATION OF EQUATION

To verify the equation developed, five additional samples were taken from research paper that are done on correlation of red clay soils of Addis Ababa[35].The selection of sample is done based on the liquidity index range indirectly.

Sample number	Depth(m)	NMC (%)	γ_{bulk} (KN/m^3)	DCPI (mm/blow)	Lliqudity Index (LI)	UCS from current research derived equation (KN/m2)	UCS from Laboratory test (KN/m2)	Varation from current research (%)
Ferensai Abo Church	**1.50**	**17.66**	**19.01**	**9.16**	**-0.17**	**360.37**	**330.00**	**8.43**
Teferi Mekonen Voc.Scool	**1.00**	**13.82**	**19.01**	**9.45**	**-0.23**	**390.49**	**340.00**	**12.93**
Ferensai Abo Church	**1.50**	**14.86**	**18.26**	**10.85**	**-0.19**	**355.09**	**288.00**	**18.89**
Teferi Mekonen Voc.Scool	**1.00**	**18.16**	**16.33**	**12.59**	**-0.11**	**296.58**	**252.00**	**15.03**
Winget-Asko Road Project	**1.50**	**35.02**	**18.32**	**17.95**	**0.03**	**185.08**	**155.00**	**16.25**

Table 6-2 Verification of UCS data using the developed equation

7. CONCLUSION AND RECOMMENDATION

7.1. CONCLUSION

The objective of introducing DCP as a simple test device that it is inexpensive, portable, and easy to operate by manufacturing it from locally available materials and applying of the equipment for determination of undrained shear strength (s_u) of clayey soils is dealt in this thesis.

In addition to estimation of UCS from DCPI and bearing capacity equation development; parametric studies are also carried out on parameters affecting DCPI. The study is conducted by categorizing the locations of sample collection in to two categories and the following conclusions can be made:

- ***Catigory-1 (Red Clay Soils of Debre Markos)*** revealed that UCS is significantly influenced by DCPI and Liquidity index. UCS, for this category, can be estimated from DCPI by

  UCS=-209.5*ln (DCPI) +800.5, R^2=80.19% with the corresponding bearing capacity equation of q_{ult}=-538.625*ln (DCPI) +12058.086+γ*D.

- ***Catigory-2(Black Clay Soils of Debre Markos)*** revealed that UCS is significantly influenced by DCPI and liquidity index.UCS, for this category, can be estimated from DCPI by

  UCS=-7.1661*(DCPI) +416.82, R^2=82.15% with the corresponding bearing capacity equation of q_{ult}=-18.423*(DCPI) +1071.64+γ*D.

  - Since the sample size used for the regression of black clay soils of Debremrkos is too small, caution should be done to use the above equation.

- From the Comparison made between the newly developed equation and the laboratory test results of sample made outside the study area one can see that the newly developed equation approximates the unconfined compressive strength of Red clay soil in a good way with in a +19%.This shows the use of this developed equation would save cost and time.

- From both researches the one which is done on red clay soils of Addis Ababa [35] and current research has shown that the unconfined compression strength (UCS) is highly influenced by DCPI and Liquidity Index(LI).

7.2. RECOMMENDATION

- The size of sample did not cover the whole area of study hence detail investigation should be done by increasing the size of samples.
- The sample size used for the regression of black clay soils of Debre Markos is too small, hence caution should be done during the use of derived equation.
- Different correlation like correlation between DCP and CBR should be done in the study area since the study area is exposed to rapid civil engineering work.

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APPENDIXES

APPENDIX – A: Field Tests Result

A – 1) DCPI Result for Sites Inside Debre Markos

Table A - 1.1 Penetration Data Report for Efoyta[E1]

Penetration data report for Efoyta site 01[1m-2m]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
			1000	
-	-	-	**87**	-
1	1	1	140	53
2	1	2	190	50
3	1	3	245	55
4	1	4	290	45
5	1	5	332	42
6	1	6	370	38
7	1	7	406	36
8	1	8	441	35
9	1	9	475	34
10	1	10	508	33
11	1	11	543	35
12	1	12	572	29
13	1	13	605	33
14	1	14	645	40
15	1	15	682	37
16	1	16	705	23
17	1	17	745	40
18	1	18	770	25
19	1	19	800	30
20	1	20	826	26
21	1	21	854	28
22	1	22	882	28
23	1	23	910	28
24	1	24	935	25
25	1	25	965	30
26	1	26	972	7

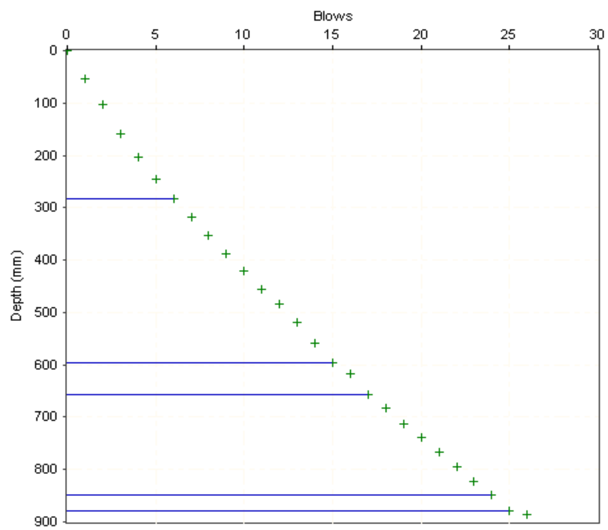

Figure A-1.1 the Dynamic Cone penetration used for Efoyta [E1] [1m-2m]

Table A - 1.2 Penetration Data Report for Efoyta [E2]

Penetration data report for Efoyta site 02[2.6m-3.5m]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
			2600	
-	-	-	**105**	-
1	1	1	200	95
2	1	2	283	83
3	1	3	310	27
4	1	4	326	16
5	1	5	342	16
6	1	6	355	13
7	1	7	369	14
8	1	8	383	14
9	1	9	398	15
10	1	10	410	12
11	1	11	422	12
12	1	12	434	12
13	1	13	444	10
14	1	14	456	12
15	1	15	467	11
16	1	16	480	13
17	1	17	490	10
18	1	18	500	10
19	1	19	510	10
20	1	20	523	13
21	1	21	532	9
22	1	22	541	9
23	1	23	553	12
24	1	24	562	9
25	1	25	572	10
26	1	26	584	12
27	1	27	593	9
28	1	28	603	10
29	1	29	612	9
30	1	30	620	8
31	1	31	630	10
32	1	32	639	9
33	1	33	648	9
34	1	34	657	9
35	1	35	665	8
36	1	36	672	7
37	1	37	679	7
38	1	38	687	8
39	1	39	696	9
40	1	40	701	5
41	1	41	710	9
42	1	42	718	8
43	1	43	726	8
44	1	44	735	9
45	1	45	740	5
46	1	46	749	9
47	1	47	755	6
48	1	48	763	8
49	1	49	772	9
50	1	50	778	6
51	1	51	784	6
52	1	52	792	8
53	1	53	803	11
54	1	54	809	6
55	1	55	818	9
56	1	56	824	6
57	1	57	831	7
58	1	58	839	8
59	1	59	846	7
60	1	60	854	8
61	1	61	862	8

62	1	62	870	8
63	1	63	876	6
64	1	64	884	8
65	1	65	892	8
66	1	66	900	8
67	1	67	909	9
68	1	68	915	6
69	1	69	922	7
70	1	70	930	8
71	1	71	938	8
72	1	72	944	6
73	1	73	950	6
74	1	74	958	8
75	1	75	962	4

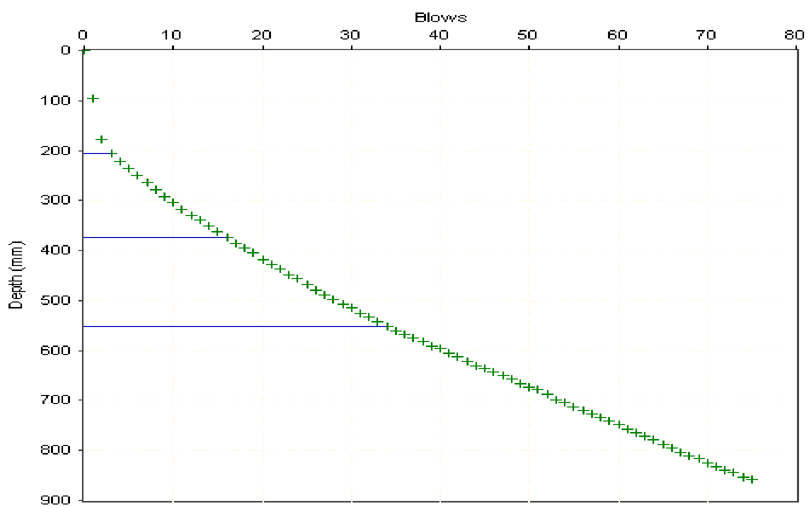

Figure A-1.2 the Dynamic Cone penetration used for Efoyta [E2] [2.6m-3.5m]

Table A - 1.3 Penetration Data Report for Addis Alem [A1]
Penetration data report for Addis Alem site 01[1m-2m]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
-	-	-	**80**	-
1	1	1	126	46
2	1	2	162	36
3	1	3	195	33
4	1	4	226	31
5	1	5	263	37
6	1	6	305	42
7	1	7	352	47
8	1	8	393	41
9	1	9	430	37
10	1	10	470	40
11	1	11	515	45
12	1	12	554	39
13	1	13	591	37
14	1	14	631	40
15	1	15	665	34
16	1	16	700	35
17	1	17	733	33
18	1	18	770	37
19	1	19	813	43
20	1	20	856	43
21	1	21	892	36
22	1	22	924	32
23	1	23	954	30

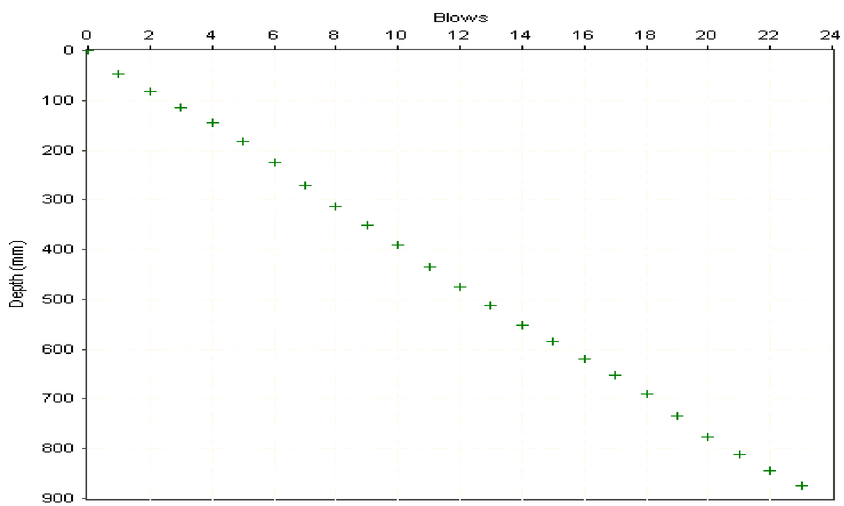

Figure A-1.3 the Dynamic Cone penetration used for Addis Alem [A1] [1m-2m]

Table A - 1.4 Penetration Data Report for Addis Alem [A2]Penetration data report for Addis Alem site 02[2.7m-3.6m]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
			2700	
-	-	-	**60**	-
1	1	1	180	120
2	1	2	223	43
3	1	3	270	47
4	1	4	290	20
5	1	5	310	20
6	1	6	331	21
7	1	7	360	29
8	1	8	387	27
9	1	9	408	21
10	1	10	430	22
11	1	11	450	20
12	1	12	470	20
13	1	13	493	23
14	1	14	520	27
15	1	15	540	20
16	1	16	563	23
17	1	17	583	20
18	1	18	603	20
19	1	19	620	17
20	1	20	634	14
21	1	21	650	16
22	1	22	670	20
23	1	23	687	17
24	1	24	703	16
25	1	25	717	14
26	1	26	730	13
27	1	27	740	10
28	1	28	750	10
29	1	29	760	10

30	1	30	770	10
31	1	31	783	13
32	1	32	800	17
33	1	33	813	13
34	1	34	830	17
35	1	35	850	20
36	1	36	863	13
37	1	37	878	15
38	1	38	893	15
39	1	39	913	20
40	1	40	940	27
41	1	41	943	3

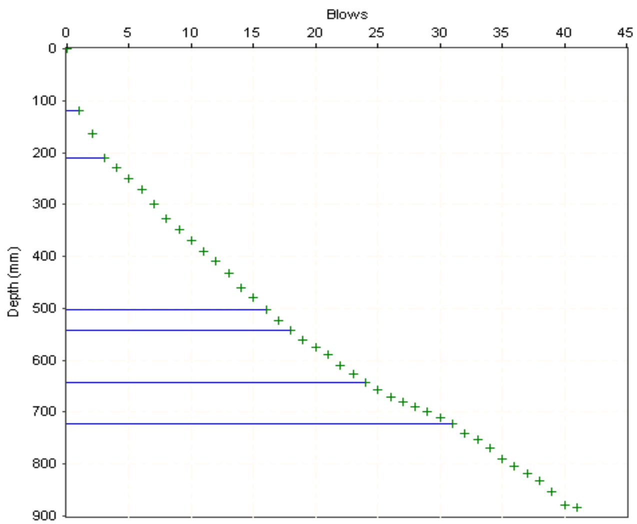

Figure A-1.4 the Dynamic Cone penetration used for Addis Alem [A2] [2.7m-3.6m]

Table A - 1.5 Penetration Data Report for Bole condominium [B1] Penetration data report for Bole condominium site 01[Ground-1m]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
			0	
-	-	-	**150**	-
1	1	1	165	15
2	1	2	177	12
3	1	3	185	8
4	1	4	192	7
5	1	5	200	8
6	1	6	205	5
7	1	7	210	5
8	1	8	215	5
9	1	9	218	3
10	1	10	222	4
11	1	11	225	3
12	1	12	229	4
13	1	13	232	3
14	1	14	235	3
15	1	15	238	3
16	1	16	240	2
17	1	17	242	2
18	1	18	245	3
19	1	19	249	4
20	1	20	251	2
21	1	21	255	4
22	1	22	259	4
23	2	24	262	1.5
24	1	25	265	3
25	1	26	268	3
26	1	27	271	3
27	1	28	276	5
28	1	29	280	4
29	1	30	282	2

30	1	31	287	5
31	1	32	292	5
32	1	33	296	4
33	2	35	298	1
34	1	36	302	4
35	1	37	304	2
36	1	38	310	6
37	1	39	313	3
38	2	41	320	3.5
39	1	42	324	4
40	1	43	326	2
41	1	44	330	4
42	1	45	335	5
43	1	46	339	4
44	1	47	343	4
45	1	48	350	7
46	1	49	351	1
47	1	50	354	3
48	1	51	358	4
49	1	52	364	6
50	1	53	366	2
51	1	54	370	4
52	1	55	375	5
53	1	56	380	5
54	1	57	385	5
55	1	58	387	2
56	1	59	393	6
57	1	60	400	7
58	1	61	409	9
59	1	62	418	9
60	1	63	425	7
61	1	64	436	11
62	1	65	449	13
63	1	66	462	13
64	1	67	480	18
65	1	68	497	17
66	1	69	517	20
67	1	70	536	19
68	1	71	559	23
69	1	72	587	28
70	1	73	618	31
71	1	74	658	40
72	1	75	703	45
73	1	76	755	52
74	1	77	795	40
75	1	78	825	30
76	1	79	863	38
77	1	80	900	37
78	1	81	933	33
79	1	82	965	32

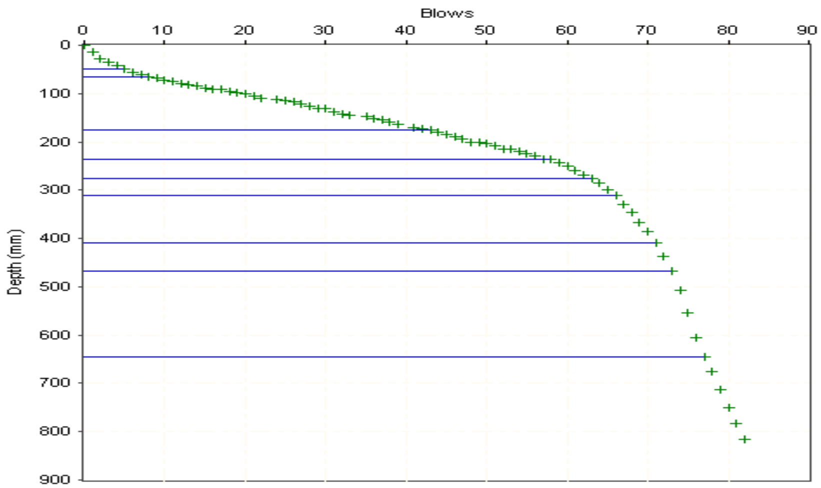

Figure A-1.5 the Dynamic Cone penetration used for Bole condominum [B1] [ground-1m]

Table A - 1.6 Penetration Data Report for Bole condominium [B1]Penetration data report for Bole condominium site 02[1m-2m]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
			1000	
-	-	-	**155**	-
1	1	1	196	41
2	1	2	233	37
3	1	3	271	38
4	1	4	304	33
5	1	5	345	41
6	1	6	377	32
7	1	7	409	32
8	1	8	438	29
9	1	9	470	32
10	1	10	500	30
11	1	11	526	26
12	1	12	555	29
13	1	13	580	25
14	1	14	609	29
15	1	15	633	24
16	1	16	653	20
17	1	17	684	31
18	1	18	710	26
19	1	19	736	26
20	1	20	766	30
21	1	21	794	28
22	1	22	818	24
23	1	23	843	25
24	1	24	864	21
25	1	25	884	20
26	1	26	908	24
27	1	27	928	20
28	1	28	946	18

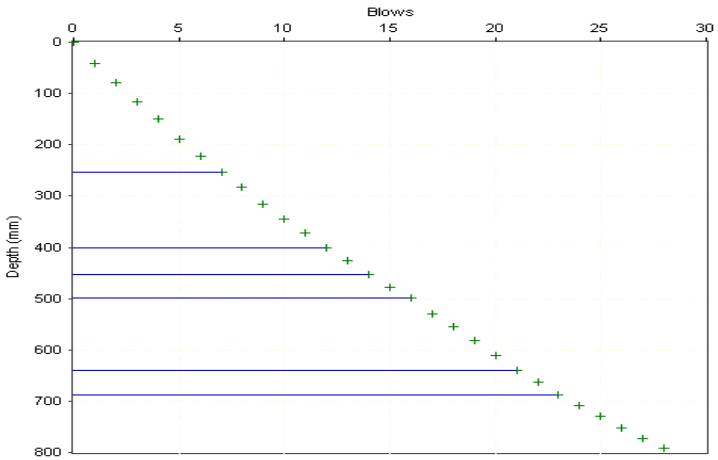

Figure A-1.6 the Dynamic Cone penetration used for Bole condominium [B2] [1m-2m]

Table A - 1.7 Penetration Data Report for Habitat [HA1]
Penetration data report for Habitat site 01[1m-2m]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
			1000	
-	-	-	**75**	-
1	1	1	103	28
2	1	2	129	26
3	1	3	158	29
4	1	4	184	26
5	1	5	211	27
6	1	6	235	24
7	1	7	256	21
8	1	8	283	27
9	1	9	311	28
10	1	10	335	24
11	1	11	360	25
12	1	12	386	26
13	1	13	401	15
14	1	14	424	23
15	1	15	448	24
16	1	16	473	25
17	1	17	499	26
18	1	18	525	26
19	1	19	551	26
20	1	20	576	25
21	1	21	600	24
22	1	22	623	23
23	1	23	645	22
24	1	24	671	26
25	1	25	696	25
26	1	26	722	26
27	1	27	747	25
28	1	28	775	28
29	1	29	800	25
30	1	30	821	21
31	1	31	847	26
32	1	32	869	22
33	1	33	891	22
34	1	34	918	27
35	1	35	944	26
36	1	36	962	18
37	1	37	965	3

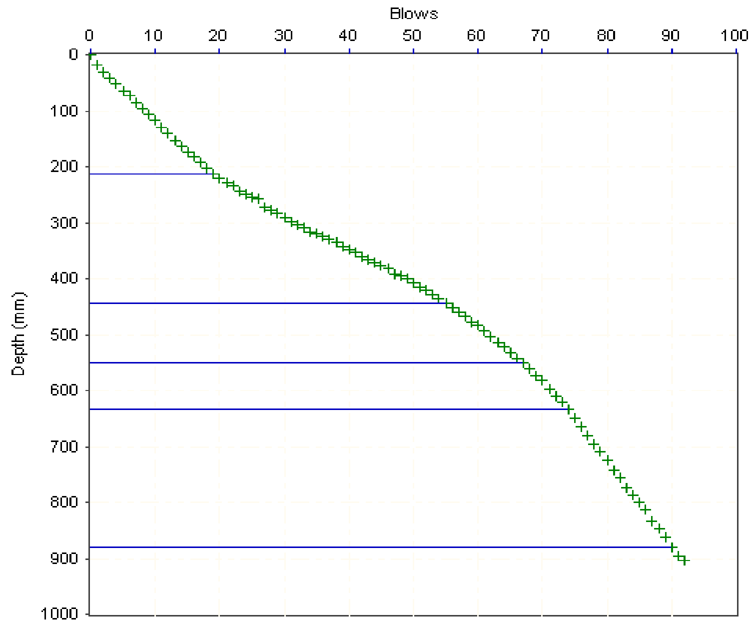

Figure A-1.7 the Dynamic Cone penetration used for Habitat [HA1] [1m-2m]

Table A - 1.8 Penetration Data Report for Habitat [HA2]

Penetration data report for Habitat site 02[2.7m-3.6m]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
-	-	-	**65**	-
1	1	1	91	26
2	1	2	112	21
3	1	3	126	14
4	1	4	141	15
5	1	5	151	10
6	1	6	161	10
7	1	7	171	10
8	1	8	181	10
9	1	9	191	10
10	1	10	201	10
11	1	11	210	9
12	1	12	219	9
13	1	13	226	7
14	1	14	235	9
15	1	15	242	7
16	1	16	252	10
17	1	17	260	8
18	1	18	267	7
19	1	19	275	8
20	1	20	285	10
21	1	21	295	10
22	1	22	302	7
23	1	23	310	8
24	1	24	319	9
25	1	25	327	8
26	1	26	335	8
27	1	27	344	9
28	1	28	353	9
29	1	29	361	8
30	1	30	370	9
31	1	31	380	10
32	1	32	390	10
33	1	33	400	10
34	1	34	410	10
35	1	35	420	10
36	1	36	430	10
37	1	37	440	10
38	1	38	452	12
39	1	39	463	11
40	1	40	475	12
41	1	41	490	15
42	1	42	504	14
43	1	43	516	12
44	1	44	529	13
45	1	45	542	13
46	1	46	555	13
47	1	47	569	14
48	1	48	581	12
49	1	49	593	12
50	1	50	605	12
51	1	51	616	11
52	1	52	628	12
53	1	53	643	15
54	1	54	655	12
55	1	55	667	12
56	1	56	678	11
57	1	57	690	12
58	1	58	702	12
59	1	59	715	13
60	1	60	726	11
61	1	61	740	14
62	1	62	752	12
63	1	63	763	11

64	1	64	776	13
65	1	65	793	17
66	1	66	805	12
67	1	67	818	13
68	1	68	830	12
69	1	69	845	15
70	1	70	860	15
71	1	71	870	10
72	1	72	884	14
73	1	73	895	11
74	1	74	906	11
75	1	75	917	11
76	1	76	927	10
77	1	77	940	13
78	1	78	951	11
79	1	79	965	14

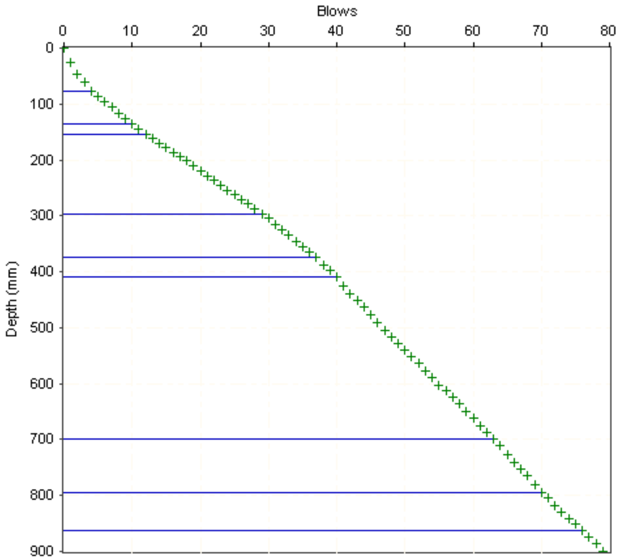

Figure A-1.8 the Dynamic Cone penetration used for Habitat [HA2] [2.7m-3.6m]

Table A - 1.9 Penetration Data Report for Genet menafesha [G1]
Penetration data report for Genet menafesha site 01[0.85m-1.75m]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
			850	
-	-	-	**75**	-
1	1	1	107	32
2	1	2	133	26
3	1	3	158	25
4	1	4	184	26
5	1	5	211	27
6	1	6	235	24
7	1	7	258	23
8	1	8	281	23
9	1	9	316	35
10	1	10	352	36
11	1	11	382	30
12	1	12	410	28
13	1	13	433	23
14	1	14	456	23
15	1	15	477	21
16	1	16	502	25
17	1	17	531	29
18	1	18	559	28
19	1	19	587	28
20	1	20	618	31
21	1	21	647	29
22	1	22	671	24
23	1	23	693	22
24	1	24	715	22
25	1	25	732	17
26	1	26	750	18
27	1	27	766	16
28	1	28	787	21
29	1	29	800	13
30	1	30	817	17
31	1	31	835	18
32	1	32	850	15
33	1	33	862	12
34	1	34	875	13
35	1	35	890	15
36	1	36	897	7
37	1	37	911	14
38	1	38	926	15
39	1	39	940	14
40	1	40	955	15
41	1	41	960	5

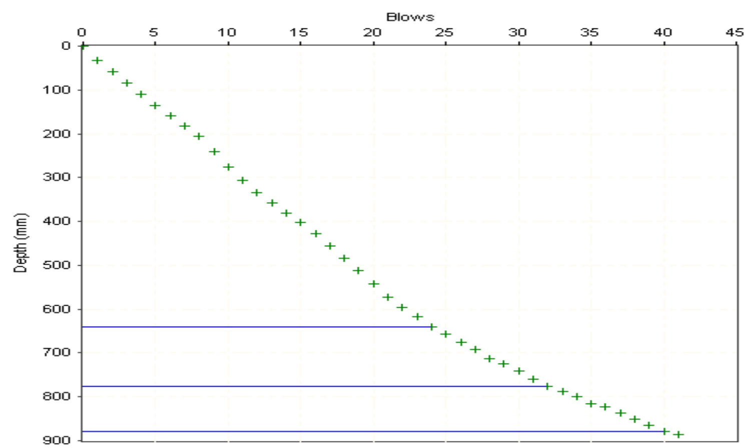

Figure A-1.9 the Dynamic Cone penetration used for Genet menafesha [G1] [0.85m-1.75m]

Table A - 1.10 Penetration Data Report for Genet menafesha [G2]
Penetration data report for Genet menafesha site 02[2m-3m]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
			2000	
-	-	-	**65**	-
1	1	1	86	21
2	1	2	104	18
3	1	3	126	22
4	1	4	146	20
5	1	5	162	16
6	1	6	180	18
7	1	7	196	16
8	1	8	220	24
9	1	9	240	20
10	1	10	263	23
11	1	11	284	21
12	1	12	304	20
13	1	13	320	16
14	1	14	335	15
15	1	15	348	13
16	1	16	363	15
17	1	17	377	14
18	1	18	393	16
19	1	19	407	14
20	1	20	421	14
21	1	21	435	14
22	1	22	449	14
23	1	23	463	14
24	1	24	477	14
25	1	25	495	18
26	1	26	502	7
27	1	27	531	29
28	1	28	552	21
29	1	29	573	21
30	1	30	592	19
31	1	31	609	17
32	1	32	626	17
33	1	33	645	19
34	1	34	661	16
35	1	35	680	19
36	1	36	698	18
37	1	37	715	17
38	1	38	730	15
39	1	39	747	17
40	1	40	765	18
41	1	41	783	18
42	1	42	799	16
43	1	43	816	17
44	1	44	835	19
45	1	45	853	18
46	1	46	873	20
47	1	47	885	12
48	1	48	903	18
49	1	49	920	17
50	1	50	945	25
51	1	51	960	15

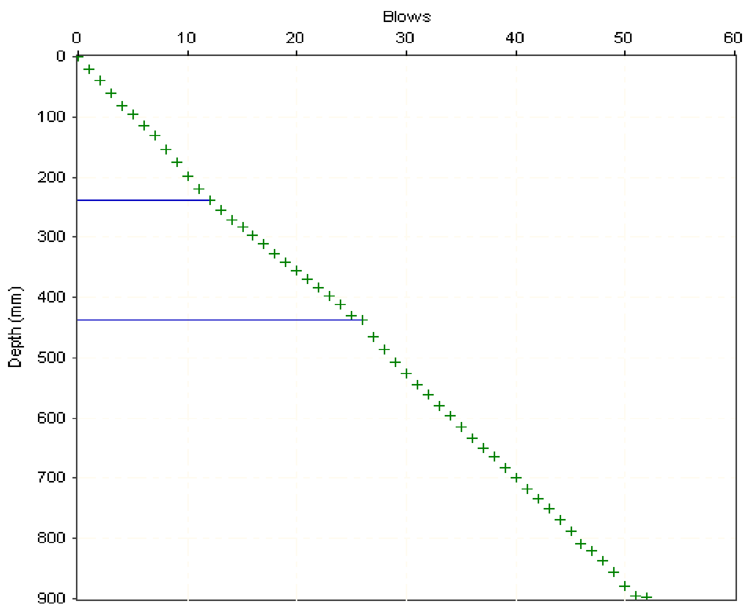

Figure A-1.10 the Dynamic Cone penetration used for Genet menafesha [G2] [2m-3m]

APPENDIX – B: Laboratory Test Results

B – 1) Moisture Content Determination

Table B - 1.1 Moisture Content for Efoyta [E1] at 1.5m

mass of container		15.2	15.8	15.7
mass of container + wet soil		52.1	47	57.5
mass of container dry soil		42.5	38.9	46.7
Moisture content		35.2	35.1	34.8
Aver.		35.0		

Table B - 1.2 Moisture Content for Efoyta [E2] at 3m

mass of container		15.4	15.7	15.5	15.5
mass of container + wet soil		48.6	40.6	50.4	50.4
mass of container dry soil		40.5	34.6	41.6	41.6
Moisture content		32.3	31.7	33.7	33.7
Aver.		32.6			

Table B - 1.3 Moisture Content for Addis Alem [A1] at 1.5m

mass of container		15.3	15.5	15.8
mass of container + wet soil		43.1	57	45.2
mass of container dry soil		36.2	46.1	37.2
Moisture content		33.0	35.6	37.4
Aver.		35.3		

Table B - 1.4 Moisture Content for Addis Alem [A2] at 3m

mass of container	15.5	15.7	15.6
mass of container + wet soil	59.3	50.2	41.5
mass of container dry soil	44.9	38.8	33
Moisture content	49.0	49.4	48.9
Aver.	49.1		

Table B - 1.5 Moisture Content for bole condominium [B1] at 1m

mass of container	15.3	15.6	15.1
mass of container + wet soil	40.1	44.3	38.3
mass of conatiner dry soil	33.6	36.8	32.1
Moisture content	35.5	35.4	36.5
Aver	35.8		

Table B - 1.6Moisture Content for bole condominium [B2] at 2m

Table B - 1.7 Moisture Content for Habitat [HA1] at 1.5m

mass of container	15.7	15.7	15.2
mass of container + wet soil	42.1	46.7	45.3
mass of conatiner dry soil	35	38.6	37.6
Moisture content	36.8	35.4	34.4
Aver.	35.5		

Table B - 1.8 Moisture Content for Habitat [HA2] at 3m

mass of container	11.7	15.6	15.5
mass of container + wet soil	40.1	42.7	45.9
mass of container dry soil	33.5	36.4	38.9
Moisture content	30.3	30.3	29.9
Aver.	30.2		

Table B - 1.9 Moisture Content for Genet Menafesha [G1] at 1.5m

mass of container	11.7	20.7	15.6
mass of container + wet soil	32.5	53.2	40.1
mass of conatiner dry soil	27	44.3	33.4
Moisture content	35.9	37.7	37.6
Aver.	37.1		

Table B - 1.10Moisture Content for Genet Menafesha [G2] at 3m

mass of container	15.5	15.8	15.5
mass of container + wet soil	32	41.7	43.8
mass of conatiner dry soil	27.5	34.9	36.5
Moisture content	37.5	35.6	34.8
Aver	36.0		

B – 2) Specific Gravity Determination

Table B-2.1(a) Specific Gravity for Efoyata[E1] at 1.5m

Determination No.	1	2
Pycnometer No.	1	2
Weight of pycnometer + soil + water, W_{pws} (g)	160.6	164.8
Temperature, T_x(°c)	23.5	21.3
Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g)	144.58	148.73
Weight of dry soil , w_s (gm)	25	25
Conversion factor , K	0.9996	0.9993
Specific gravity of soil at 20°c.	2.78	2.80
Average specific gravity of soil.	2.79	

Table B-2.1 (b) Specific Gravity for Efoyta [E2] at 3m

Determination No.	1	2
Pycnometer No.	1	2
Weight of pycnometer + soil + water, W_{pws} (g)	160.8	164.8
Temperature, T_x(°c)	24.4	24.8
Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g)	144.58	148.73
Weight of dry soil , w_s (gm)	25	25
Conversion factor , K	0.9996	0.9993
Specific gravity of soil at 20°c.	2.85	2.80
Average specific gravity of soil .		2.82

Table B-2.2(a) Specific Gravity for Addis Alem [A1] at 1.5m

Determination No.	1	2
Pycnometer No.	1	2
Weight of pycnometer + soil + water, W_{pws} (g)	160.5	164.8
Temperature, T_x(°c)	23.6	23.4
Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g)	144.58	148.73
Weight of dry soil , w_s (gm)	25	25
Conversion factor , K	0.9996	0.9993
Specific gravity of soil at 20°c.	2.75	2.80
Average specific gravity of soil .		2.77

Table B-2.2 (b) Specific Gravity for Addis Alem [A2] at 3m

Pycnometer No.	1	2
Weight of pycnometer + soil + water, W_{pws} (g)	160.4	165.1
Temperature, T_x(°c)	26.3	24
Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g)	144.58	148.73
Weight of dry soil , w_s (gm)	25	25
Conversion factor , K	0.9996	0.9993
Specific gravity of soil at 20°c.	2.72	2.89
Average specific gravity of soil.		2.81

Table B-2.3(a) Specific Gravity for Bole condominium [B1] at 1m

Determination No.	1	2
Pycnometer No.	1	2
Weight of pycnometer + soil + water, W_{pws} (g)	160.7	164.9
Temperature, T_x(°c)	21.4	21
Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g)	144.58	148.73
Weight of dry soil , w_s (gm)	25	25
Conversion factor , K	0.9996	0.9993
Specific gravity of soil at 20°c.	2.81	2.83
Average specific gravity of soil .		2.82

Table B-2.3 (b) Specific Gravity for Bole condominium [B2] at 2m

Determination No.	1	2
Pycnometer No.	1	2
Weight of pycnometer + soil + water, W_{pws} (g)	160.4	165.1
Temperature, T_x(°c)	26.3	24
Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g)	144.58	148.73
Weight of dry soil , w_s (gm)	25	25
Conversion factor , K	0.9996	0.9993
Specific gravity of soil at 20°c.	2.72	2.89
Average specific gravity of soil .		2.81

Table B-2.4 (a) Specific Gravity for Habitat [HA1] at 1.5m

Determination No.	1	2
Pycnometer No.	1	2
Weight of pycnometer + soil + water, W_{pws} (g)	160.5	164.7
Temperature, T_x(°c)	25	24.2
Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g)	144.58	148.73
Weight of dry soil , w_s (gm)	25	25
Conversion factor , K	0.9996	0.9993
Specific gravity of soil at 20°c.	2.75	2.77
Average specific gravity of soil .		2.76

Table B-2.4 (b) Specific Gravity for Habitat [HA2] at 3m

Determination No.	1	2
Pycnometer No.	1	2
Weight of pycnometer + soil + water, W_{pws} (g)	160.9	164.9
Temperature, T_x(°c)	24.2	24.2
Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g)	144.58	148.73
Weight of dry soil , w_s (gm)	25	25
Conversion factor , K	0.9996	0.9993
Specific gravity of soil at 20°c.	2.88	2.83
Average specific gravity of soil .		2.85

Table B-2.5 (a) Specific Gravity for Genet menafesha [G1] at 1.5m

Determination No.	1	2
Pycnometer No.	1	2
Weight of pycnometer + soil + water, W_{pws}(g)	160.7	160.6
Temperature, T_x(°c)	20.2	20.2
Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g)	144.58	144.53
Weight of dry soil , w_s (gm)	25	25
Conversion factor , K	0.9996	0.9993
Specific gravity of soil at 20°c.	2.81	2.80
Average specific gravity of soil .		2.81

Table B-2.5 (a) Specific Gravity for Genet menafesha [G2] at 3m

Determination No.	1	2
Pycnometer No.	1	2
Weight of pycnometer + soil + water, W_{pws}(g)	160.9	164.8
Temperature, T_x(°c)	21.7	21
Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g)	144.58	148.73
Weight of dry soil , w_s (gm)	25	25
Conversion factor , K	0.9996	0.9993
Specific gravity of soil at 20°c.	2.88	2.80
Average specific gravity of soil .		2.84

B – 3) Atterberg Limits Determination

Table B – 3.1 (a) Atterberg Limits for Efoyta [E1] at1.5m

	Liquid Limit			Plastic Limit	
Trial No					
Container No	1	2	3.00	1.00	2.00
Mass of container, g	15.60	15.50	15.90	15.30	15.40
Mass of container + Wet soil, g	34.80	34.40	34.50	22.50	23.00
Mass of container + Dry soil, g	27.50	27.10	27.20	20.70	21.10
Mass of water, g	7.30	7.30	7.30	1.80	1.90
Mass of dry soil, g	11.90	11.60	11.30	5.40	5.70
Water content, %	61.34	62.93	64.60	33.33	33.33
No of blows	26	23	19	------	------
Log(No of blows)	1.415	1.361	1.278		

Liquid Limit, % = 61.88   Plastic Limit, % 33.33   PI, %= 28.55

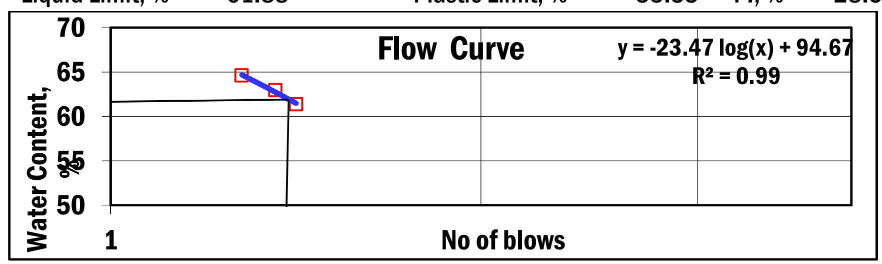

Figure B -3.1 (a) Atterberg Limits for Efoyta [E1] at 1.5m

Table B – 3.1 (b) Atterberg Limits for Efoyta [E2] at3m

Liquid limit=60.66 plastic limit=31.20 PI=29.46

	Liquid Limit				Plastic Limit	
Container No	1	2	3.00	4.00	1.00	2.00
Mass of container, g	15.60	15.60	15.40	15.70	15.70	15.70
Mass of container + Wet soil, g	29.50	28.70	27.50	29.30	24.10	23.70
Mass of container + Dry soil, g	24.20	23.60	23.00	24.20	22.10	21.80
Mass of water, g	5.30	5.10	4.50	5.10	2.00	1.90
Mass of dry soil, g	8.60	8.00	7.60	8.50	6.40	6.10
Water content, %	61.63	63.75	59.21	60.00	31.25	31.15
No of blows	20	15	33	28	------	------
Log(No of blows)	1.301	1.176	1.518	1.447		

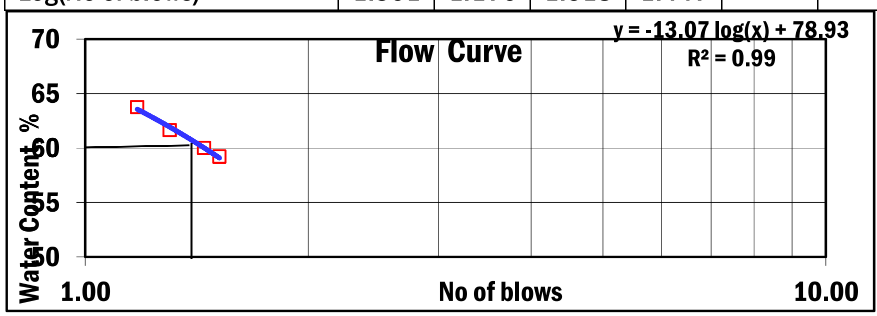

Figure B -3.1 (b) Atterberg Limits for Efoyta [E2] at 3m

Table B – 3.2 (a) Atterberg Limits for Addis Alem [A1] at 1.5m

Liquid limit=62.72 plastic limit=31.24 PI=31.48

	Liquid Limit				Plastic Limit
Container No	2	3.00	4.00	1.00	2.00
Mass of container, g	15.50	15.40	15.70	15.30	15.50
Mass of container + Wet soil, g	36.20	33.90	30.80	22.50	23.00
Mass of container + Dry soil, g	28.30	26.70	24.70	20.80	21.20
Mass of water, g	7.90	7.20	6.10	1.70	1.80
Mass of dry soil, g	12.80	11.30	9.00	5.50	5.7
Water content, %	61.72	63.72	67.78	30.91	31.58
No of blows	27	23	18	------	------
Log(No of blows)	1.431	1.361	1.255		

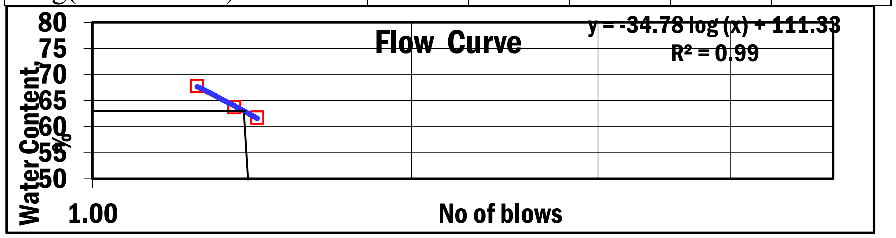

Figure B -3.2 (a) Atterberg Limits for Addis Alem [A1] at 1.5m

Table B – 3.2 (b) Atterberg Limits for Addis Alem[A2]at 3m

Liquid limit=100.68 plastic limit=31.5 PI=69.18

	Liquid Limit			Plastic Limit	
Trial No					
Container No	1	2	3.00	1.00	2.00
Mass of container, g	15.10	15.70	15.10	15.60	15.70
Mass of container + Wet soil, g	30.90	31.40	33.10	21.20	23.40
Mass of container + Dry soil, g	22.90	23.50	24.10	19.90	21.50
Mass of water, g	8.00	7.90	9.00	1.30	1.90
Mass of dry soil, g	7.80	7.80	9.00	4.30	5.80
Water content, %	102.56	101.28	100.00	30.23	32.76
No of blows	16	24	27	------	------
Log(No of blows)	1.20	1.38	1.431		

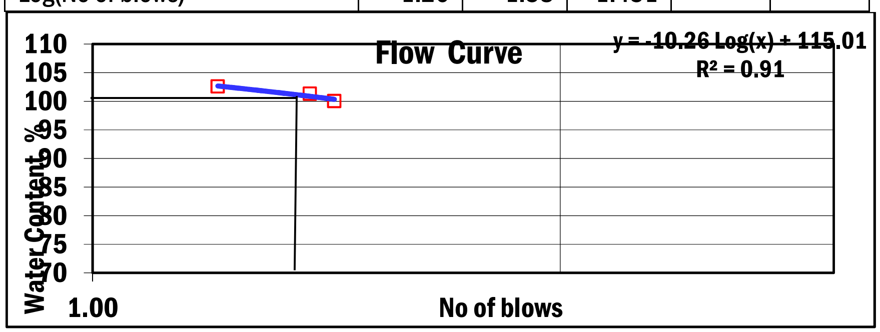

Figure B -3.2 (b) Atterberg Limits for Addis Alem [A2] at 3m

Table B – 3.3 (a) Atterberg Limits for Bole condominium [B1] at 1m

Liquid limit=62.37 plastic limit=34.19 PI=28.18

	Liquid Limit			Plastic Limit	
Container No	1	2.00	3.00	2.00	1.00
Mass of container, g	15.60	15.50	15.60	15.40	15.70
Mass of container + Wet soil, g	36.50	38.40	33.90	23.60	22.80
Mass of container + Dry soil, g	28.50	29.20	27.20	21.50	21.00
Mass of water, g	8.00	9.20	6.70	2.10	1.80
Mass of dry soil, g	12.90	13.70	11.60	6.10	5.30
Water content, %	62.02	67.15	57.76	34.43	33.96
No of blows	24	17	39	------	
Log (No of blows)	1.38	1.23	1.59		

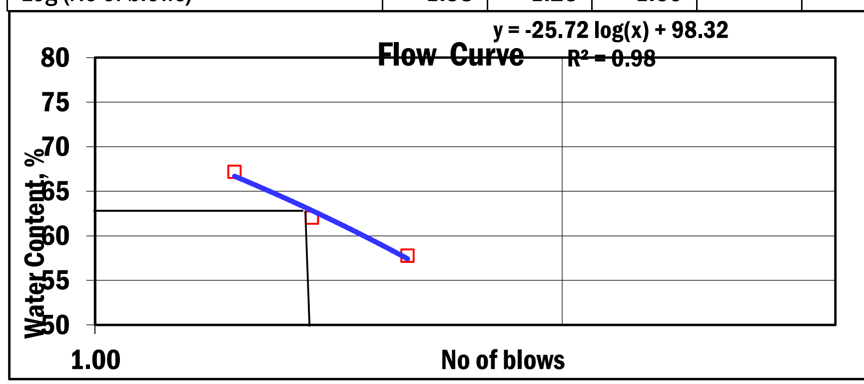

Figure B -3.3 (a) Atterberg Limits for Bole condominium [B1] at 1.5m

Table B – 3.3 (b) Atterberg Limits for Bole condominium [B2] at 2m

Liquid limit=61.76 plastic limit=34.38 PI=27.38

	Liquid Limit			Plastic Limit	
Trial No					
Container No	2	3.00	4.00	2.00	1.00
Mass of container, g	15.70	15.70	15.60	15.40	15.60
Mass of container + Wet soil, g	37.60	34.60	34.90	23.30	24.10
Mass of container + Dry soil, g	29.40	27.30	27.10	21.40	21.80
Mass of water, g	8.20	7.30	7.80	1.90	2.30
Mass of dry soil, g	13.70	11.60	11.50	6.00	6.20
Water content, %	59.85	62.93	67.83	31.67	37.10
No of blows	29	23	15	------	------
Log (No of blows)	1.46	1.361	1.17		

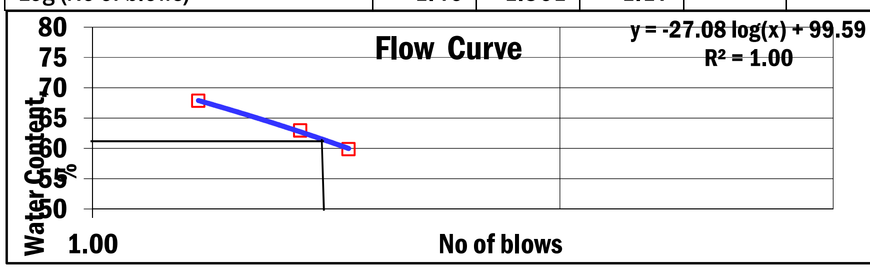

Figure B -3.3 (b) Atterberg Limits for Bole condominium [B2] at 2m

Table B – 3.4 (a) Atterberg Limits for Habitat [HA1] at 1.5m

Liquid limit=58.18 plastic limit=35.77 PI=22.41

	Liquid Limit				Plastic Limit	
Container No	a	b	c	d	1.00	2.00
Mass of container, g	15.30	15.60	15.60	15.60	15.60	15.50
Mass of container + Wet soil, g	36.00	35.90	31.60	36.40	22.60	24.40
Mass of container + Dry soil, g	28.20	28.40	25.70	29.00	20.80	22.00
Mass of water, g	7.80	7.50	5.90	7.40	1.80	2.40
Mass of dry soil, g	12.90	12.80	10.10	13.40	5.20	6.50
Water content, %	60.47	58.59	58.42	55.22	34.62	36.92
No of blows	16	24	26	39	------	------
Log (No of blows)	1.20	1.38	1.414	1.591		

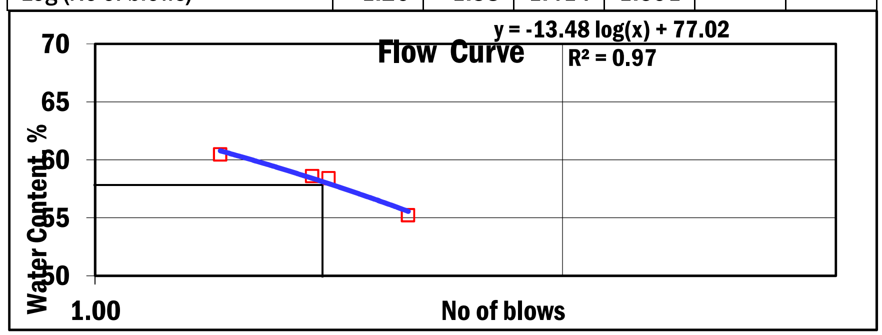

Figure B -3.4 (a) Atterberg Limits for Habitat [HA1] at 1.5m

Table B – 3.4 (b) Atterberg Limits for Habitat [HA2] at 3m

Liquid limit=50.99 plastic limit=33.98 PI=17.01

	Liquid limit			Plastic Limit	
Container No	2	3.00	4.00	1.00	2.00
Mass of container, g	15.70	15.70	15.70	15.70	15.50
Mass of container + Wet soil, g	40.30	41.80	38.10	22.40	22.60
Mass of container + Dry soil, g	31.90	33.10	30.70	20.70	20.80
Mass of water, g	8.40	8.70	7.40	1.70	1.80
Mass of dry soil, g	16.20	17.40	15.00	5.00	5.30
Water content, %	51.85	50.00	49.33	34.00	33.96
No of blows	22	28	39	------	------
Log (No of blows)	1.34	1.44	1.59		

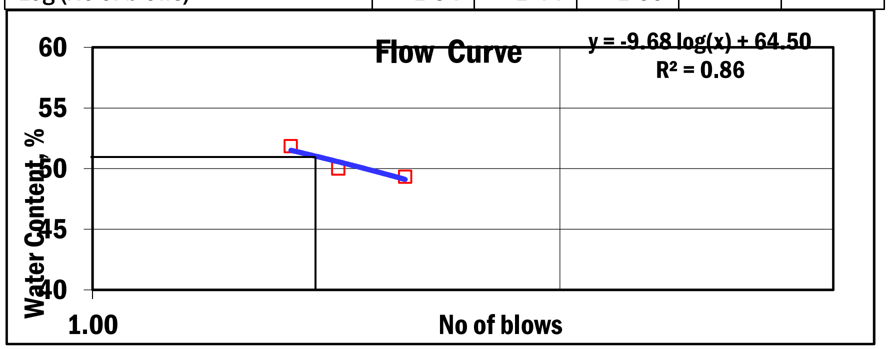

Figure B -3.4 (b) Atterberg Limits for Habitat [HA2] at 3m

Table B – 3.5 (a) Atterberg Limits for Genet menafesha [G1] at 1.5m

Liquid limit=65.2 plastic limit=37.42 PI=27.78

	Liquid Limit				Plastic Limit	
Trial No						
Container No	a	b	c	d	1.00	2.00
Mass of container, g	14.10	15.40	13.90	10.80	15.40	15.90
Mass of container + Wet soil, g	34.80	39.30	39.50	37.90	22.60	23.40
Mass of container + Dry soil, g	26.70	30.20	29.20	27.10	20.80	21.20
Mass of water, g	8.10	9.10	10.30	10.80	1.80	2.20
Mass of dry soil, g	12.60	14.80	15.30	16.30	5.40	5.30
Water content, %	64.29	61.49	67.32	66.26	33.33	41.51
No of blows	26	39	19	24	------	------
Log (No of blows)	1.41	1.591	1.278	1.38		

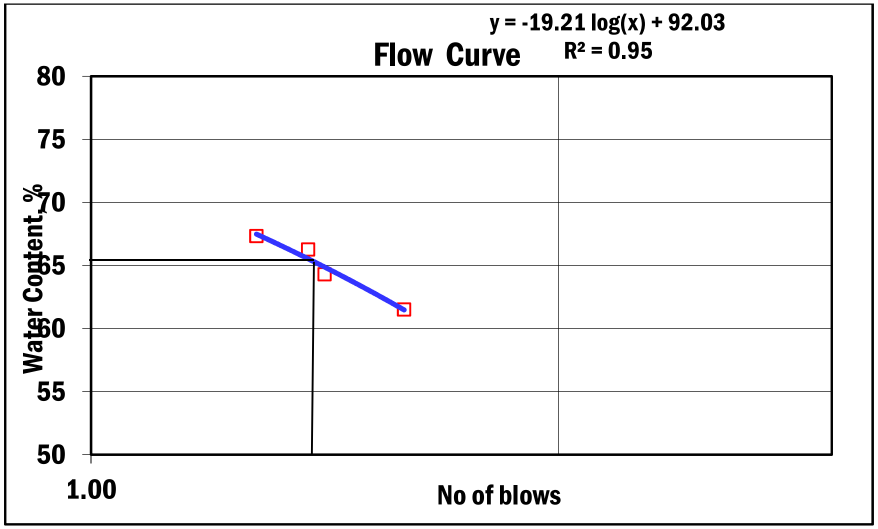

Figure B -3.5 (a) Atterberg Limits for genet menafesha [G1] at 1.5m

Table B – 3.5 (b) Atterberg Limits for Genet menafesha [G2] at 3m

Liquid limit=69.62 plastic limit=35.39 PI=34.23

	Liquid Limit			Plastic Limit	
Trial No					
Container No	1	2	3.00	1.00	2.00
Mass of container, g	15.40	15.50	15.50	15.30	15.40
Mass of container + Wet soil, g	29.40	34.50	34.10	22.00	21.70
Mass of container + Dry soil, g	23.80	26.80	26.40	20.20	20.10
Mass of water, g	5.60	7.70	7.70	1.80	1.60
Mass of dry soil, g	8.40	11.30	10.90	4.90	4.70
Water content, %	66.67	68.14	70.64	36.73	34.04
No of blows	35	28	23	------	------
Log (No of blows)	1.54	1.44	1.36		

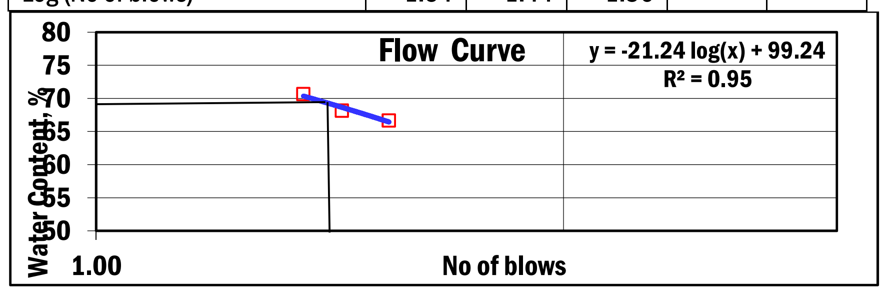

Figure B -3.5 (b) Atterberg Limits for genet menafesha [G2] at 3m

B – 4) Free swell Determination

Table B - 4.1(a) Free Swell for Efoyta [E1] at 1.5m

Initial Volume (cc)	Final Volume		Average Final Volume (cc)	Free Swell (%)
	Sample No.1 (cc)	Sample No.2 (cc)		
10.0	14.0	13.0	13.5	35

Table B - 4.1(a) Free Swell for Efoyta [E2] at 3m

Initial Volume (cc)	Final Volume		Average Final Volume (cc)	Free Swell (%)
	Sample No.1 (cc)	Sample No.2 (cc)		
10.0	14.0	14.0	14.0	40

Table B - 4.2(a) Free Swell for Addis Alem [A1] at 1.5m

Initial Volume (cc)	Final Volume		Average Final Volume (cc)	Free Swell (%)
	Sample No.1 (cc)	Sample No.2 (cc)		
10.0	15.5	16.0	15.8	58

Table B - 4.2(b) Free Swell for Addis Alem [A2] at 3m

Initial Volume (cc)	Final Volume		Average Final Volume (cc)	Free Swell (%)
	Sample No.1 (cc)	Sample No.2 (cc)		
10.0	21.0	21.0	21.0	110

Table B - 4.3(a) Free Swell for Bole Condominium [B1] at 1.5m

Initial Volume (cc)	Final Volume		Average Final Volume (cc)	Free Swell (%)
	Sample No.1 (cc)	Sample No.2 (cc)		
10.0	14.0	15.0	14.5	45

Table B - 4.3(b) Free Swell for Bole Condominium [B2] at 3m

Initial Volume (cc)	Final Volume		Average Final Volume (cc)	Free Swell (%)
	Sample No.1 (cc)	Sample No.2 (cc)		
10.0	14.0	14.0	14.0	40

Table B - 4.4(a) Free Swell for Habitat [HA1] at 1.5m

Initial Volume (cc)	Final Volume		Average Final Volume (cc)	Free Swell (%)
	Sample No.1 (cc)	Sample No.2 (cc)		
10.0	13.0	14.0	13.5	35

Table B - 4.4(b) Free Swell for Habitat [HA2] at 3m

Initial Volume (cc)	Final Volume		Average Final Volume (cc)	Free Swell (%)
	Sample No.1 (cc)	Sample No.2 (cc)		
10.0	13.8	13.0	13.4	34

Table B - 4.5(a) Free Swell for Genet Menafesha [G1] at 1.5m

Initial Volume (cc)	Final Volume		Average Final Volume (cc)	Free Swell (%)
	Sample No.1 (cc)	Sample No.2 (cc)		
10.0	13.0	13.5	13.3	33

Table B - 4.5(b) Free Swell for Genet Menafesha [G2] at 3m

Initial Volume (cc)	Final Volume		Average Final Volume (cc)	Free Swell (%)
	Sample No.1 (cc)	Sample No.2 (cc)		
10.0	15.0	14.5	14.8	48

B – 5) Unconfined Compression Strength Determination

Table B – 5.1(a) Unconfined Compression Strength for Efoyta [E2] at 3m

Test Pit No:	**Efoyta**	**Cross- Sectional Area , m²**	**0.001134**
Depth, m :	1.50	Ring Calibration Factor, kN/div	0.00142
Sampling	Undisturbed	Moisture content, %	34.86
Diameter of sample , mm	38	Wet unit weight, kN/m³	17.68
Length of sample , mm	82	Dry Unit Weight, kN/m³	13.11
		Rate of Strain, mm/min	1.70

Axial Deformation [mm]	Axial Strain [%]	Proving Ring Reading [div]	Axial Load [kN]	Corrected Area [m2]	Axial Stress [kPa]
0.00	0.00	0	0.0000	0.001134	0
0.20	0.25	14	0.0199	0.001137	17.49
0.40	0.50	20	0.0284	0.001140	24.92
0.60	0.75	26	0.0369	0.001143	32.31
0.80	1.00	31	0.0440	0.001146	38.43
1.00	1.25	36	0.0511	0.001148	44.51
1.20	1.50	42	0.0596	0.001151	51.80
1.40	1.75	48	0.0682	0.001154	59.05
1.60	2.00	55	0.0781	0.001157	67.49
1.80	2.25	60	0.0852	0.001160	73.43
2.00	2.50	66	0.0937	0.001163	80.57
2.20	2.75	72	0.1022	0.001166	87.67
2.40	3.00	78	0.1108	0.001169	94.73
2.60	3.25	84	0.1193	0.001172	101.76
2.80	3.50	92	0.1306	0.001175	111.16
3.00	3.75	96	0.1363	0.001178	115.69
3.20	4.00	102	0.1448	0.001181	122.60
3.40	4.25	107	0.1519	0.001184	128.28
3.60	4.50	113	0.1605	0.001188	135.12
3.80	4.75	117	0.1661	0.001191	139.53
4.00	5.00	120	0.1704	0.001194	142.74
4.20	5.25	123	0.1747	0.001197	145.92
4.40	5.50	124	0.1761	0.001200	146.72
4.60	5.75	123	0.1747	0.001203	145.15
4.80	6.00	120	0.1704	0.001207	141.23
5.00	6.25	116	0.1647	0.001210	136.16
5.20	6.50	111	0.1576	0.001213	129.95
5.40	6.75	105	0.1491	0.001216	122.59

Unconfined Compressive Strength , kPa = 147

Figure B-5.1 (a) Unconfined Compression Strength for Efoyta[E2] at 3m

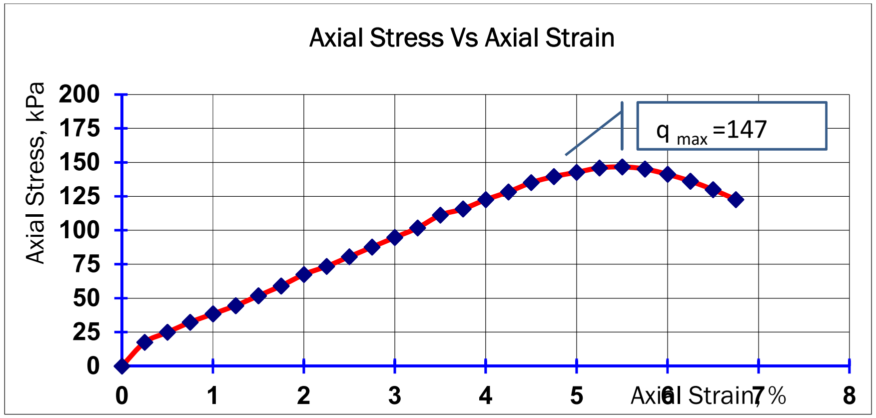

Table B – 5.2(a) Unconfined Compression Strength for Stadium [S1] at 1m

Test Pit :	**Stadium**	**Cross- Sectional Area , m²**	**0.001134**
Depth, m :	1	Ring Calibration Factor, kN/div	0.00142
Sampling	Undisturbed	Moisture content, %	29.20
Diameter of sample , mm	38	Wet unit weight, kN/m³	15.48
Length of sample , mm	76	Dry Unit Weight, kN/m³	11.98
		Rate of Strain, mm/min	1.70

Axial Deformation [mm]	Axial Strain [%]	Proving Ring Reading [div]	Axial Load [kN]	Corrected Area [m2]	Axial Stress [kPa]
0.00	0.00	0	0.0000	0.001134	0
0.20	0.26	3	0.0043	0.001137	3.75
0.40	0.53	4	0.0057	0.001140	4.98
0.60	0.79	5	0.0071	0.001143	6.21
0.80	1.05	6	0.0085	0.001146	7.43
1.00	1.32	8	0.0114	0.001149	9.88
1.20	1.58	10	0.0142	0.001152	12.32
1.40	1.84	12	0.0170	0.001155	14.75
1.60	2.11	15	0.0213	0.001159	18.39
1.80	2.37	17	0.0241	0.001162	20.78
2.00	2.63	19	0.0270	0.001165	23.16
2.20	2.89	20	0.0284	0.001168	24.32
2.40	3.16	22	0.0312	0.001171	26.68
2.60	3.42	23	0.0327	0.001174	27.81
2.80	3.68	24	0.0341	0.001177	28.94
3.00	3.95	25	0.0355	0.001181	30.07
3.20	4.21	26	0.0369	0.001184	31.18
3.40	4.47	27	0.0383	0.001187	32.29
3.60	4.74	28	0.0398	0.001191	33.40
3.80	5.00	29	0.0412	0.001194	34.49
4.00	5.26	30	0.0426	0.001197	35.59
4.20	5.53	31	0.0440	0.001200	36.67
4.40	5.79	32	0.0454	0.001204	37.75
4.60	6.05	33	0.0469	0.001207	38.82
4.80	6.32	34	0.0483	0.001211	39.88
5.00	6.58	36	0.0511	0.001214	42.11
5.20	6.84	37	0.0525	0.001217	43.16
5.40	7.11	38	0.0540	0.001221	44.20
5.60	7.37	39	0.0554	0.001224	45.23
5.80	7.63	40	0.0568	0.001228	46.26
6.00	7.89	41	0.0582	0.001231	47.28
6.20	8.16	42	0.0596	0.001235	48.30
6.40	8.42	43	0.0611	0.001238	49.31
6.60	8.68	44	0.0625	0.001242	50.31

6.80	8.95	45	0.0639	0.001246	51.30
7.00	9.21	46	0.0653	0.001249	52.29
7.20	9.47	48	0.0682	0.001253	54.41
7.40	9.74	49	0.0696	0.001256	55.38
7.60	10.00	50	0.0710	0.001260	56.34
7.80	10.26	52	0.0738	0.001264	58.43
8.00	10.53	54	0.0767	0.001268	60.50
8.20	10.79	55	0.0781	0.001271	61.43
8.40	11.05	57	0.0809	0.001275	63.48
8.60	11.32	59	0.0838	0.001279	65.51
8.80	11.58	60	0.0852	0.001283	66.43
9.00	11.84	62	0.0880	0.001286	68.44
9.20	12.11	64	0.0909	0.001290	70.43
9.40	12.37	65	0.0923	0.001294	71.32
9.60	12.63	66	0.0937	0.001298	72.20
9.80	12.89	67	0.0951	0.001302	73.07
10.00	13.16	68	0.0966	0.001306	73.94
10.20	13.42	69	0.0980	0.001310	74.80
10.40	13.68	70	0.0994	0.001314	75.65
10.60	13.95	71	0.1008	0.001318	76.50
10.80	14.21	70	0.0994	0.001322	75.19
11.00	14.47	69	0.0980	0.001326	73.89
11.20	14.74	68	0.0966	0.001330	72.59

Unconfined Compressive Strength , kPa = **76.50**

Figure B-5.2 (a) Unconfined Compression Strength for stadium [S1] at 1m

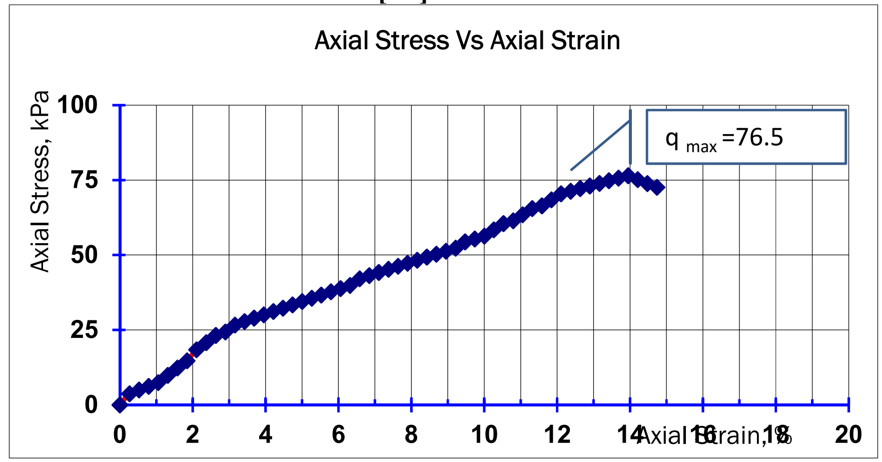

Table B – 5.3(a) Unconfined Compression Strength for Coran condominium [C2] at 2m

Test Pit :	**Coran condominium**	**Cross- Sectional Area , m^2**	**0.001134**
Depth, m :	2.00	Ring Calibration Factor, kN/div	0.00142
Sampling	Undisturbed	Moisture content, %	35.72
Diameter of sample , mm	38	Wet unit weight, kN/m^3	16.25
Length of sample , mm	91	Dry Unit Weight, kN/m^3	11.98
		Rate of Strain, mm/min	1.70

Axial Deformation [mm]	Axial Strain [%]	Proving Ring Reading [div]	Axial Load [kN]	Corrected Area [m2]	Axial Stress [kPa]
0.00	0.00	0	0.0000	0.001134	0
0.20	0.22	22	0.0312	0.001137	27.49
0.40	0.44	31	0.0440	0.001139	38.64
0.60	0.66	40	0.0568	0.001142	49.75
0.80	0.88	48	0.0682	0.001144	59.57
1.00	1.10	54	0.0767	0.001147	66.87
1.20	1.32	58	0.0824	0.001149	71.66
1.40	1.54	63	0.0895	0.001152	77.67
1.60	1.76	66	0.0937	0.001154	81.18
1.80	1.98	69	0.0980	0.001157	84.68
2.00	2.20	72	0.1022	0.001160	88.17
2.20	2.42	75	0.1065	0.001162	91.64
2.40	2.64	78	0.1108	0.001165	95.09
2.60	2.86	80	0.1136	0.001167	97.30
2.80	3.08	82	0.1164	0.001170	99.51
3.00	3.30	84	0.1193	0.001173	101.71
3.20	3.52	85	0.1207	0.001175	102.68
3.40	3.74	86	0.1221	0.001178	103.66
3.60	3.96	83	0.1179	0.001181	99.81
3.80	4.18	71	0.1008	0.001184	85.19
					-

Unconfined Compressive Strength , kPa = **104**

Figure B-5.3 (a) Unconfined Compression Strength for Coran condominium [C2] at 2m

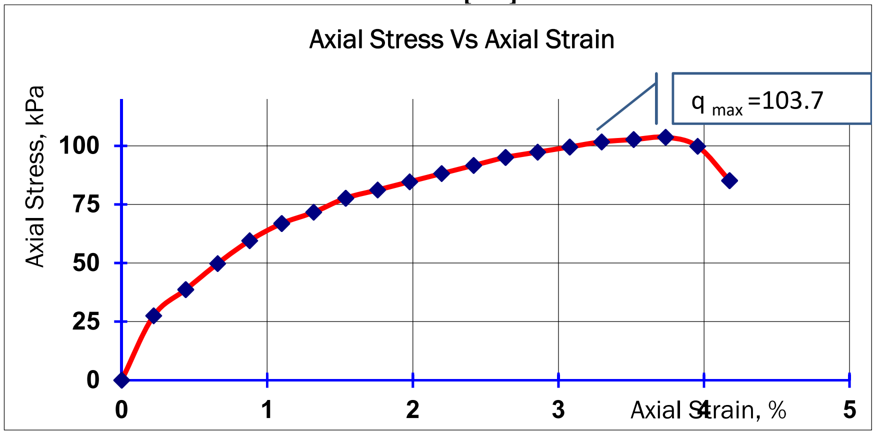

Table B – 5.4(a) Unconfined Compression Strength for Efoyta [E2] at 3m

Test Pit No:	Efoyta	Cross- Sectional Area , m^2	0.001134
Depth, m :	3.00	Ring Calibration Factor, kN/div	0.00142
Sampling	Undisturbed	Moisture content, %	32.58
Diameter of sample , mm	38	Wet unit weight, kN/m^3	17.71
Length of sample , mm	90	Dry Unit Weight, kN/m^3	13.36
		Rate of Strain, mm/min	1.70

Table B – 5.4(a) Unconfined Compression Strength for Efoyta [E2] at 3m

Test Pit No:	Efoyta	Cross- Sectional Area , m^2	0.001134
Depth, m :	3.00	Ring Calibration Factor, kN/div	0.00142
Sampling	Undisturbed	Moisture content, %	32.58
Diameter of sample , mm	38	Wet unit weight, kN/m^3	17.71
Length of sample , mm	90	Dry Unit Weight, kN/m^3	13.36
		Rate of Strain, mm/min	1.70

Axial Deformation [mm]	Axial Strain [%]	Proving Ring Reading [div]	Axial Load [kN]	Corrected Area [m2]	Axial Stress [kPa]
0.00	0.00	0	0.0000	0.001134	0
0.20	0.22	11	0.0156	0.001137	13.74
0.40	0.44	17	0.0241	0.001139	21.19
0.60	0.67	25	0.0355	0.001142	31.09
0.80	0.89	32	0.0454	0.001144	39.71
1.00	1.11	41	0.0582	0.001147	50.76
1.20	1.33	51	0.0724	0.001149	63.00
1.40	1.56	63	0.0895	0.001152	77.65
1.60	1.78	73	0.1037	0.001155	89.78
1.80	2.00	82	0.1164	0.001157	100.62
2.00	2.22	92	0.1306	0.001160	112.63
2.20	2.44	98	0.1392	0.001163	119.70
2.40	2.67	104	0.1477	0.001165	126.74
2.60	2.89	110	0.1562	0.001168	133.75
2.80	3.11	115	0.1633	0.001171	139.51
3.00	3.33	119	0.1690	0.001173	144.03
3.20	3.56	122	0.1732	0.001176	147.32
3.40	3.78	125	0.1775	0.001179	150.60
3.60	4.00	128	0.1818	0.001181	153.86
3.80	4.22	131	0.1860	0.001184	157.10
4.00	4.44	133	0.1889	0.001187	159.13
4.20	4.67	135	0.1917	0.001190	161.14
4.40	4.89	136	0.1931	0.001192	161.96
4.60	5.11	137	0.1945	0.001195	162.77
4.80	5.33	135	0.1917	0.001198	160.02
5.00	5.56	133	0.1889	0.001201	157.27
5.20	5.78	130	0.1846	0.001204	153.37

Unconfined Compressive Strength , kPa = **162.77**

Figure B-5.4 (b) Unconfined Compression Strength for Efoyta[E2] at 3m

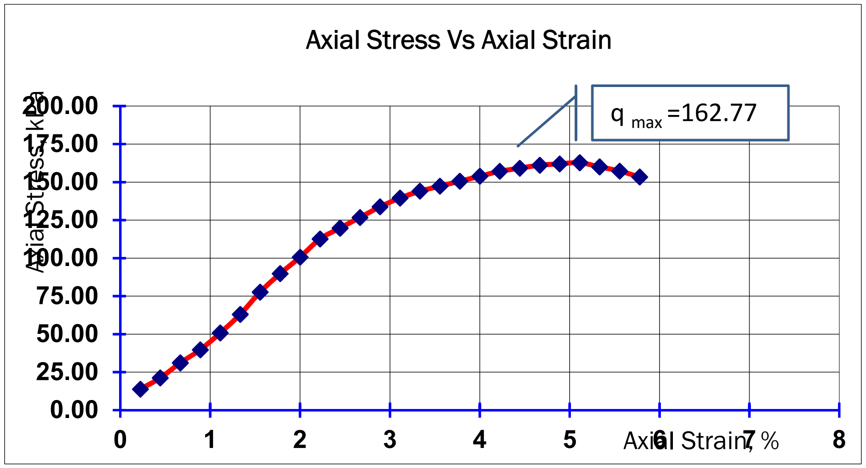

Table B – 5.5(a) Unconfined Compression Strength for Yebragy [Y2] at 2m

Test Pit No:	Yebragy	Cross- Sectional Area , m^2	0.001134
Depth, m :	2.00	Ring Calibration Factor, kN/div	0.00142
Sampling	Undisturbed	Moisture content, %	34.35
Diameter of sample , mm	38	Wet unit weight, kN/m^3	17.41
Length of sample , mm	81	Dry Unit Weight, kN/m^3	12.96
		Rate of Strain, mm/min	1.70

Axial Deformation [mm]	Axial Strain [%]	Proving Ring Reading [div]	Axial Load [kN]	Corrected Area [m2]	Axial Stress [kPa]
0.00	0.00	0	0.0000	0.001134	0
0.20	0.25	20	0.0284	0.001137	24.98
0.40	0.49	34	0.0483	0.001140	42.36
0.60	0.74	45	0.0639	0.001143	55.93
0.80	0.99	55	0.0781	0.001145	68.18
1.00	1.23	65	0.0923	0.001148	80.38
1.20	1.48	75	0.1065	0.001151	92.51
1.40	1.73	85	0.1207	0.001154	104.59
1.60	1.98	95	0.1349	0.001157	116.60
1.80	2.22	108	0.1534	0.001160	132.22
2.00	2.47	117	0.1661	0.001163	142.88
2.20	2.72	125	0.1775	0.001166	152.26
2.40	2.96	132	0.1874	0.001169	160.38
2.60	3.21	136	0.1931	0.001172	164.82
2.80	3.46	130	0.1846	0.001175	157.14
3.00	3.70	125	0.1775	0.001178	150.71

Unconfined Compressive Strength , kPa = **164.82**

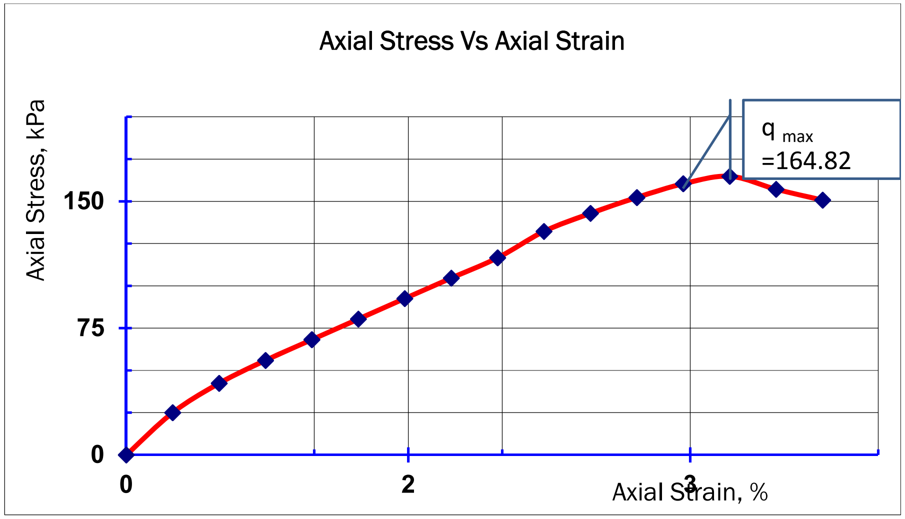

Figure B-5.5 (a) Unconfined Compression Strength for yebragy[Y2] at 2m

Appendix C: General Overview of Work done

Habitat DCP Test

Addis Alem DCP test

Dinbza high school DCP test

Disturbed Samples

DECLARATION

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

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Date: September, 2015

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