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DEPARTMENT OF CIVIL ENGINEERING

Correlating Dynamic Cone Penetration Index (DCPI) with Undrained Shear Strength for Clayey Soils

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A Thesis Submitted to the School of Graduate Studies of Addis Ababa University in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering [Geotechnical Engineering]

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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
GPS	Global Positioning System
G_s	Specific Gravity
γ	Unit weight of soil
h	Depth of cone tip while recording
kPa	kilo Pascal
LI	Liquidity Index
LL	Liquid Limit
m	meter
MH	Inorganic Silt of High Plasticity
mm	Millimetre
NGL	Natural Ground Level
NMC	Natural Moisture Content
N/A	Not Available
N_c	Bearing capacity factor for cohesion
N_q	Bearing capacity factor for surcharge
N_γ	Bearing capacity factor for unit weight
ORN	Overseas Road Note
ω	Water Content
PI	Plasticity Index/ Penetration 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

SNNP	South Nations, Nationalities and Peoples
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 engineers practicing in developing countries. This usually leads to usage of unreliable designing methods. 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 were conducted. After analysing the data by categorising in to three categories, it has been found out that parameters like unconfined compression strength, liquidity index, depth, natural moisture content and bulk density have influence on the Dynamic Cone Penetration Index (DCPI). Undrained shear strength can be estimated by UCS=-197ln(DCPI)+735.5 with coefficient of determination (R^2) of 71.1% for red clay soils of Addis Ababa and by $UCS=895.8*DCPI^{-0.56}$ with R^2 of 52.4%, for clayey soils including both red clay and black cotton soils of Addis Ababa combined. These good correlations were used to develop bearing capacity equation based on bearing capacity theory. The equation found were $q_{ult}=-506.5\ln(DCPI)+\gamma h+1891$ for red clay soils of Addis Ababa and $q_{ult}=2303.1*DCPI^{-0.56}+\gamma h$ for clayey soils including both red clay and black cotton soils of Addis Ababa combined. 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 ours, where construction of small scale buildings like residential, warehouses, local market places, health centres, which are thought to solve the problem of the society in general, is growing in a fast rate. The design trend should be based on a more reliable mode of determining the bearing properties and performance of soils.

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 one.

There are various types of DCPs available in the world. They are operated on the same principle. In the current research, a light weight DCP device was manufactured and used for evaluation of the undrained shear strength of clayey soils. The device consists is of an 8 kg mass dropping through a height of 575 mm and a 60^0 cone having a base diameter of 20 mm. The penetration of the cone is measured using a calibrated scale.

1.2. OBJECTIVE OF THE STUDY

1.2.1. General Objective

The general objective of this research is to use Dynamic Cone Penetration (DCP) equipment for determination of undrained shear strength (s_u) of clayey soils.

1.2.2. Specific Objective

The specific objectives of the research are:

- **a)** to establish relations between undrained shear strength and DCP penetration per blow for clayey soils.
- **b)** to develop equations based on bearing capacity theory.

c) to study factors and parameters of soil property that affect DCP result.

d) to introduce DCP as a simple test device that is inexpensive, portable, and easy to operate by manufacturing it from locally available materials.

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 foundations in the *areas of study* which have mostly clayey soils, both red and black cotton /expansive/.

> *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 [1].*

In addition to the estimation of bearing capacity, the equipment once manufactured 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 these areas.

1.4. ORGANIZATION OF THE THESIS

The thesis is organized into six parts. The first part 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 second part. The review also covers bearing capacity theory and shear resistance of penetrometers. The third part presents test methods, data collection and test result analysis. The analyses of parameters under study are presented in the fourth part. Discussions on analysis are presented in the fifth part. Finally, conclusions are given in sixth part.

2. LITERATURE REVIEW

2.1. General

The primary purpose of this literature review is to describe DCP's historical background and its acceptance for the practice. The review also covers the following areas:

- summary of hardware, procedures and specifications that exist on the device;
- discussion on soil parameters that affect DCP result;
- discussion on shear strength of clay 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;

2.2. Dynamic Cone Penetrometer

2.2.1. History of DCP Equipment

Soil penetration testing devices like the DCP have a long, but subdued 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 [2].

The earliest record of a subsoil penetration testing device similar to the DCP is a "ram penetrometer," developed in Germany at the end of 17^{th} century by Nicholas Goldman. The next major development again came from Germany, when Künzel in 1936 developed what was known as a "Prüfstab". This device was later used by Paproth in 1943, and eventually become standardized in 1964 as the "Light Penetrometer", German Standard DIN 4094 [2].

Concurrent with the German standardization of the "Light Penetrometer", several other countries developed their own standard penetration devices. The DCP used by several Departments of Transportation in the United States and Canada, was originally developed by Scala in Australia in 1956. 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. Following its adoption as the Central African Standard DCP, it was later simplified and modified by Van Vuuren in South Africa [2, 3, 4 and 5].

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 [3].

2.2.2. Description of the Device

The Dynamic Cone Penetrometer (DCP) used in this research is based on the widely accepted Central African Standard or South African as modified by the Transvaal Road Department [6]. The device was manufactured locally for the purpose of conducting this research and used for collecting dynamic cone penetration data for sites inside Addis Ababa. 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 device is calibrated before use and the penetration of the cone is measured using a calibrated scale. Since the manufactured device does not support extension rods, it will only be possible to measure up to 1.2m with this arrangement.

Figure 2-1 shows the equipment used for current research. For detail construction notes refer to User Guide to the Dynamic Cone Penetrometer and detail drawings prepared by Minnesota Department of Transportation [6]. The equipment 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 though 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) **Cone:** The cone measures 20 mm in diameter and has a 60^0 cone.

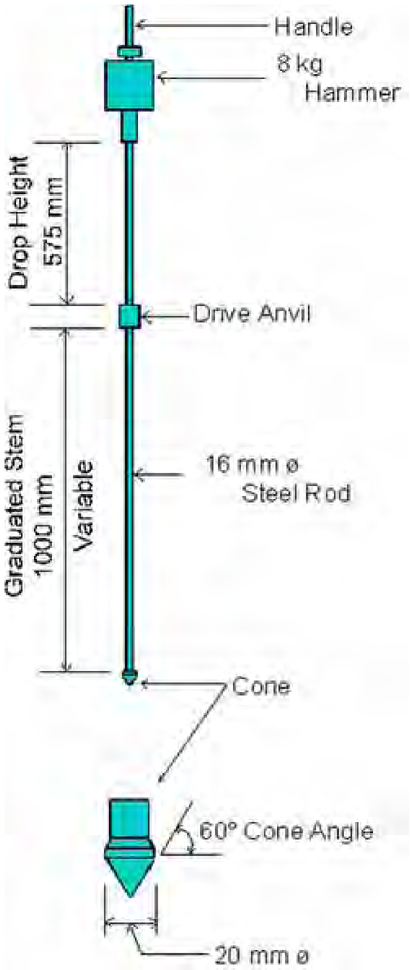


Figure 2-1 Dynamic Cone Penetration Equipment

This device has undergone some minor modifications in its design, since it's frequently used by many departments of transportation in different countries, with the most significant change occurring at the connection between the upper and lower rods. Originally a threaded connection is now changed to a simple slip plug and bolt connection. Other notable modifications include an increase in the weld size at all junctions, for prolonged device life.

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 is 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 two persons, one to drop 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. 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.

Depth which the cone penetrates with each drop of the hammer, penetration index, is expressed in terms of millimetres per blow. Small penetration rate represent better soil material in terms of shear strength. The depth of penetration can be plotted versus number of blows to identify the thickness of the different underlying materials (Figure 2.2), and the penetration index can directly be correlated with a number of common design parameters which are used to determine the bearing properties and performance of the underlying soil. Some of these correlations have been discussed in more detail later in the literature review.

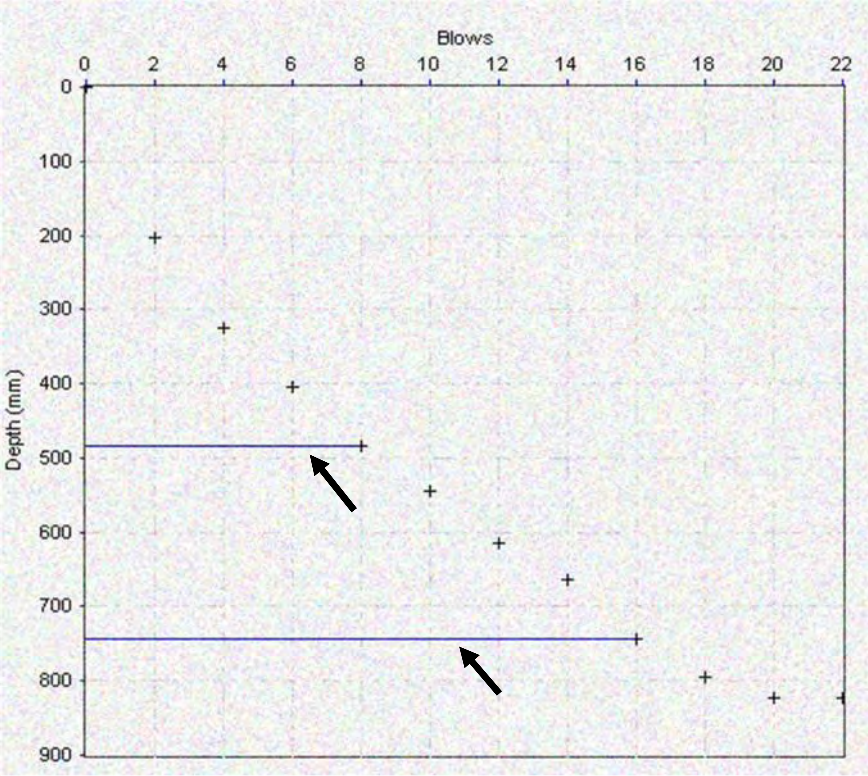


Figure 2-2 Depth of penetration versus number of blows for Imperial Hotel area

2.2.4. Factors Affecting DCP Results

There are some factors that affect the applicability of the equipment and reliability of the test results obtained from the dynamic cone penetrometer. Several investigators have studied the influence of several factors on the Dynamic Cone Penetration Index (DCPI) and they have implied that the following are the factors affecting the outcome of the DCP results.

a) **Material Effects:** Klein and Savageas cited by Amini [7] indicated that moisture content, gradation, density, and plasticity were important material properties influencing the DCPI. Hassan [8] performed a study on the effects of several variables on the DCPI. He concluded that for fine-grained soils, moisture content, soil classification and dry density affect the DCPI. For coarse-grained soils, coefficient of uniformity and confining pressures were important variables.

b) **Vertical Confinement Effect:** Livneh, et al. [9] performed a comprehensive study of the vertical confinement effect on dynamic cone penetrometer strength values in pavement and sub grade evaluations. The results have shown that there is no vertical confinement effect by upper cohesive layers on the DCP values of lower cohesive sub grade layers. In addition, their findings have indicated that no vertical confinement effect exists by the upper granular layer on the DCP values of the cohesive sub grade beneath them. 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.

c) **Side Friction Effect:** Because 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 [8].

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 lightweight structures.

b) **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.

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

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

- Compaction control or verification in embankment, drainage and pavement construction.
- Verification or control using penetrometer to check individual foundations during construction where the shear strength characteristics range is generally known.
- Determine the bearing properties and performance of soils.

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

Other than manpower errors there are 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 of soils.

The maximum depth suggested for this test is about 6m. 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. UNDRAINED SHEAR STRENGTH OF CLAYEY SOILS

2.3.1. General

It is usually assumed that the shear strength of soils is governed by the Mohr-Coulomb failure criterion:

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

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

σ'= (σ'-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.

For most saturated clays, tested under quick undrained conditions, the angle of shearing resistance is zero. This means that the shear strength of the clay is a fixed value and is equal to the 'apparent cohesion' (i.e., the response of pore water pressure to imposed loads). For drained conditions, or in terms of effective stresses, it is found that the shear strength of soils is principally a frictional phenomenon. This does not appear to be the case for over consolidated clays which have a built-in pre-stress or for partially saturated clays, in which the particles are drawn together by surface tension effects, giving them some cohesion.

Partially saturated soils, tested in undrained conditions, will show a behaviour which is intermediate between that for drained conditions and for saturated undrained conditions, depending on the degree of saturation [10].

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.

2.3.2. Predicting Undrained Shear Strength of Clays

I. From Simple Hand Tests

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. One 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 [11]

Description	q_u(kPa)	SPT N-Value	Remark
Very Soft	<25	0-2	Squishes between finger when squeezed
Soft	25-50	3-5	Very easily deformed by squeezing
Medium Stiff (firm)	50-100	6-9	?? (*Thumb makes impression easily*)

Description	q_u(kPa)	SPT N-Value	Remark
Stiff	100-200	10-16	Hard to deform by hand squeezing
Very Stiff	200-400	17-30	Very hard to deform by hand
Hard	>400	>30	Nearly impossible to deform by hand

II. From Simple Classification Tests

The other 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 [$LI = \frac{Wn-PI}{LL-PI}$] have been established by Skempton and Northey (1952) as cited by [10] and these are given in Figure 2-3.

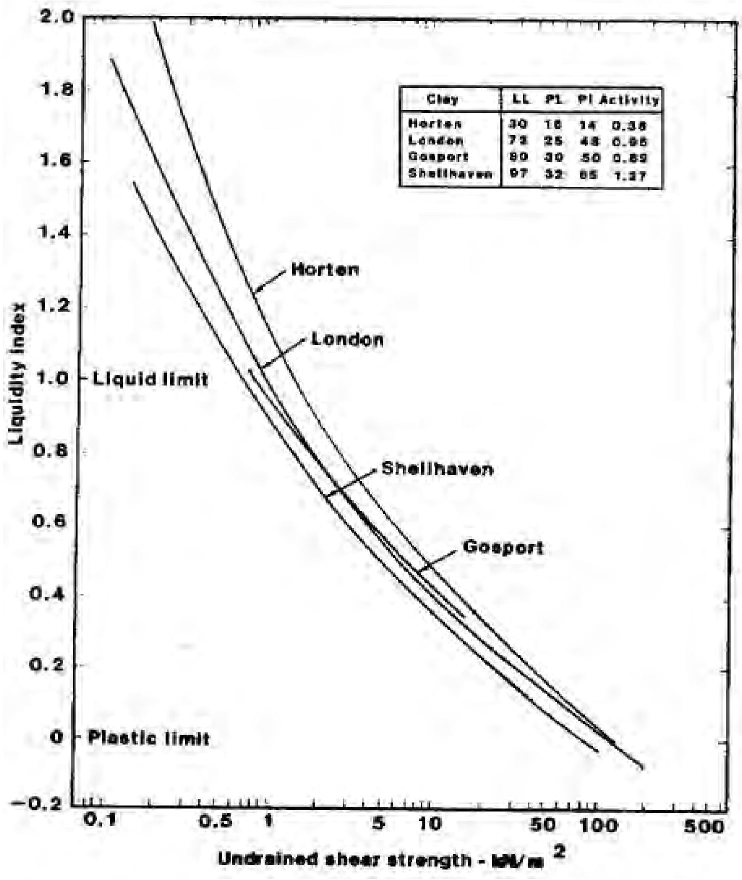


Figure 2-3 Correlation between shear strength and liquidity index established by Skempton and Northey (1952) as cited by [10]

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 [10] relates mainly to clays of relatively moderate sensitivity with natural moisture contents below the liquid limit. Their findings are given in Figure 2-4.

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 2-5).

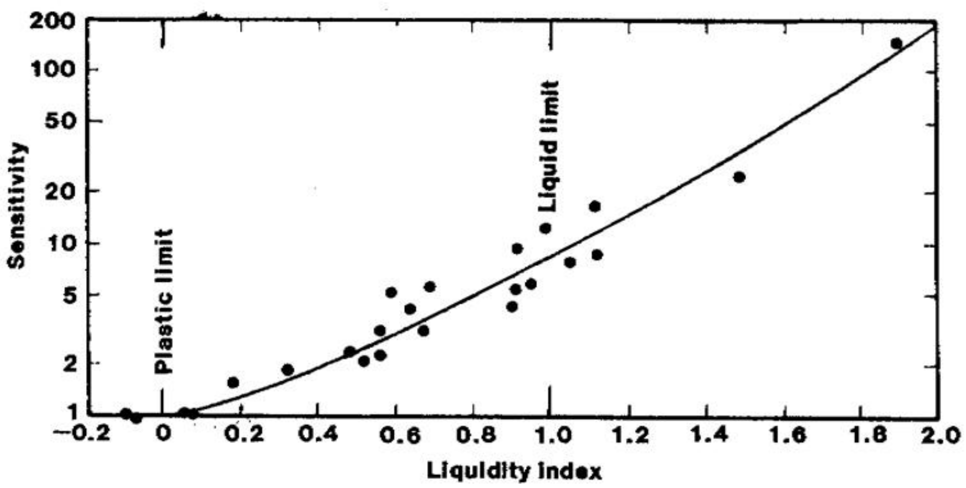


Figure 2-4 Correlation between sensitivity and liquidity index after Skempton and Northey (1952) as cited by [10]

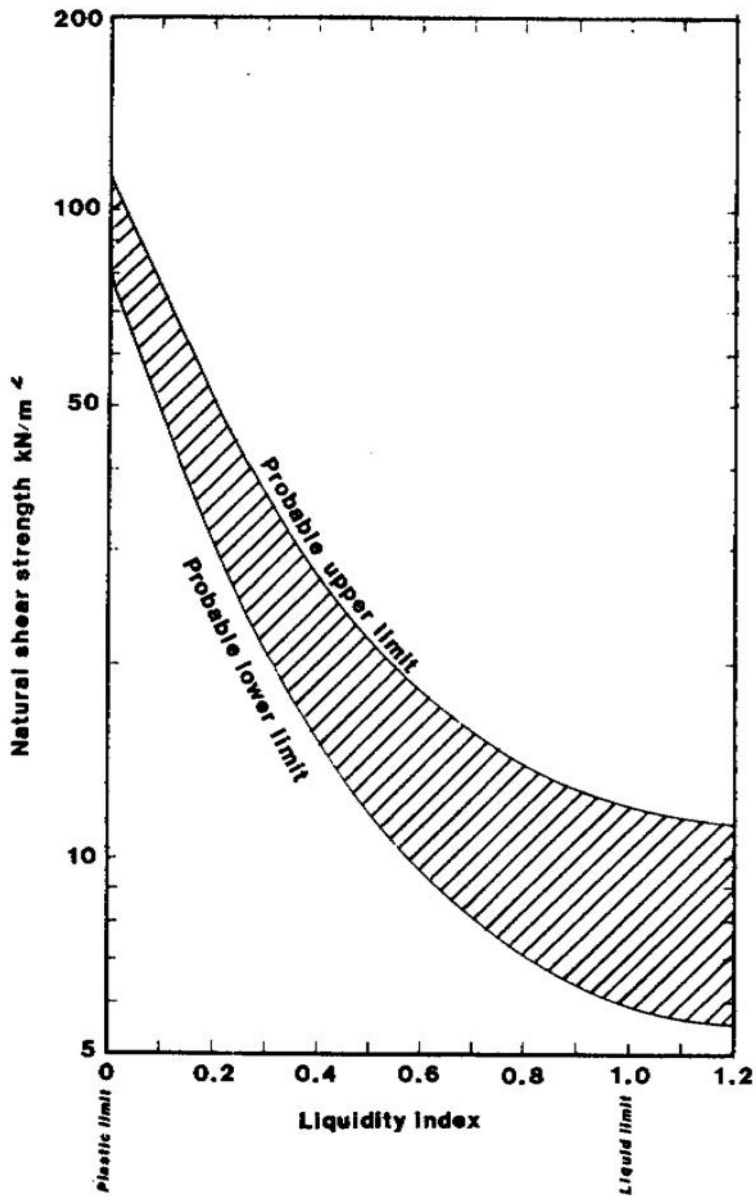


Figure 2-5 Correlation between natural shear strength and liquidity index [10]

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 (1957) as cited by [10]:

$$\sigma'_v = \frac{s_u}{0.11 + 0.0037PI} \quad \text{……………………………………(2.5)}$$

where PI is the plasticity index

s_u is shear strength and

σ'_v is over burden pressure

III. From SPT N-Value

For over consolidated clays, Stroud (1974) as cited by Clayton [12] 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 triaxial compression test on 102-mm diameter specimens.

$$c_u = f_1 N_{60} \quad \text{…………………………………………(2.7)}$$

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 2.6). 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 2.7 will under-estimate the undrained shear strength [12].

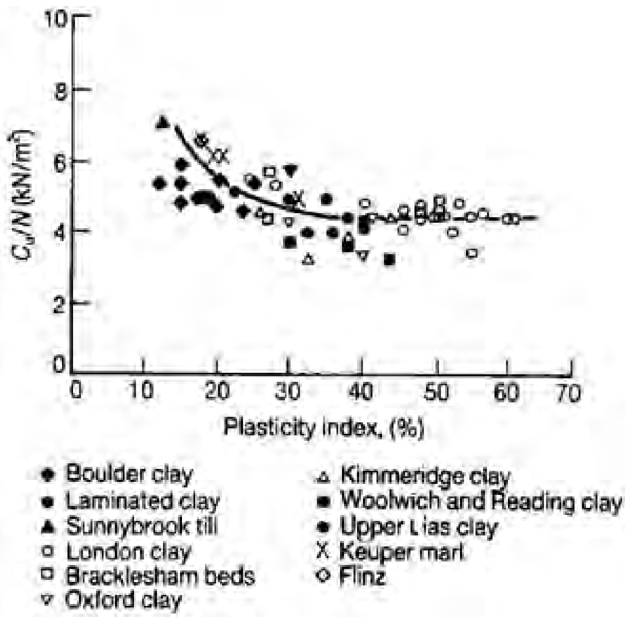


Figure 2-6 Correlation between N value and undrained shear strength for insensitive clays Stroud (1974) as cited by [12]

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 (1971) as cited by Carter [10] and Clayton [12] shows values with c_u/N ratios apparently varying between 0.4 and 20 (Figure 2-7).

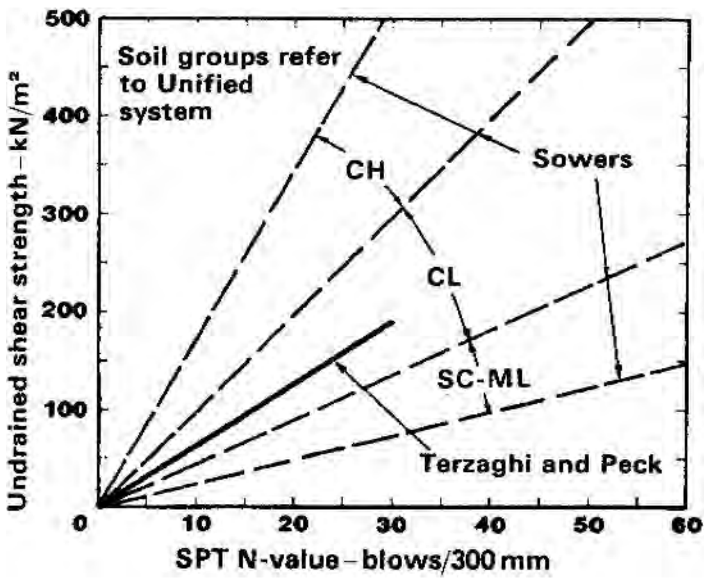


Figure 2-7 Approximate correlations between undrained shear strength and SPT N-values De Mello (1971) as cited by [12]

2.4. DETERMINATION OF GEOTECHNICAL PARAMETERS USING DCP

2.4.1. Prediction of Undrained Shear Strength

It is known that undrained shear strength of clays can be predicted from many indices including the SPT N-value as discussed by section 2.3.2 (III). Another correlation that is used for prediction is that of DCPI versus unconfined compressive strength (UCS). Several graphs of the correlation between UCS and DCPI can be found in literature (Figure 2-8)[13].

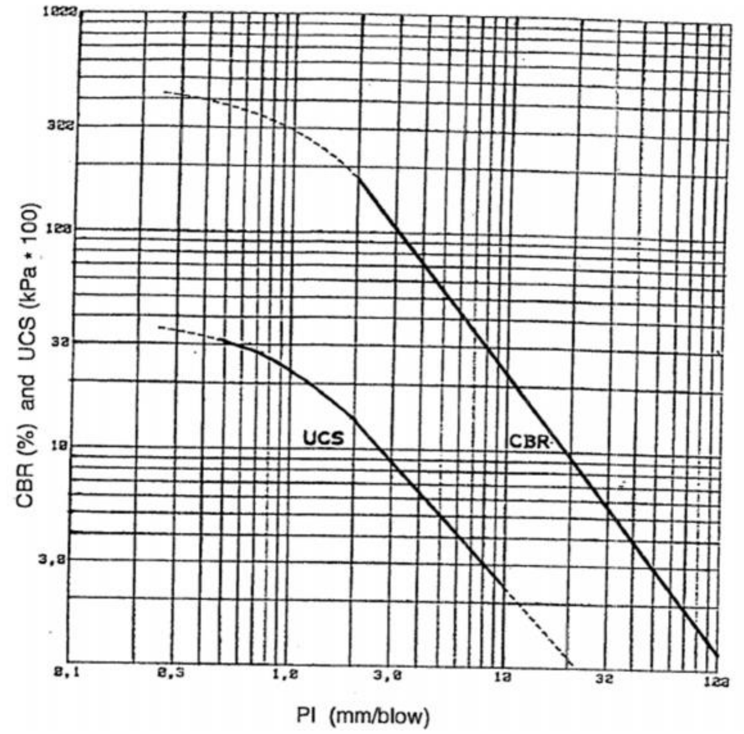


Figure 2-8 Relationship between Penetration Index (PI), California Bearing Ratio (CBR) and Unconfined Compressive Strength (UCS) [13]

Another research done on lime stabilized soils by McElvaney and Djatnika (1991) as cited by Amini [7] indicated that DCPI values can be correlated to the unconfined compressive strength (UCS) of soil-lime mixtures. They considered both individual and combined soil types in their analysis. They have concluded that the inclusion of data on mixtures from material with zero lime content has negligible effects on the correlation equations, indicating that the correlation is mainly a function of strength and not of the way in which strength is achieved.

This observation was valid only for lower range of strain values. For the combined data, three relationships, with each model permitting estimated unconfined compressive strength to a predetermined reliability level, were developed. These relationships are summarized below [7]:

$$Log_{10}(UCS) = 3.56 - 0.807Log_{10}(DCPI) \quad (2.6)$$

50% probability of underestimation

$$Log_{10}(UCS) = 3.29 - 0.809Log_{10}(DCPI) \quad (2.7)$$

95% confident that probability of underestimation will not exceed 15 percent

$$Log_{10}(UCS) = 3.21 - 0.809Log_{10}(DCPI) \quad (2.8)$$

99% confident that probability of underestimation will not exceed 15 percent

Where UCS is Unconfined Compressive Strength (kPa) and DCPI is Dynamic Cone Penetration Index (mm/blow).

2.4.2. Prediction of SPT N-value

Sowers and Hedges [1] and later Livneh and Ishai [14], developed a correlation between DCPI and standard penetration test (SPT) results which are only valid for SPT < 10mm/blow (Figure 2-9). The correlation equation took the form:

$$Log_{10}(DCPI)=-A+ B\ Log_{10}(SPT) \quad (2.10)$$

It should be noted that both studies involved the use of DCP's having slightly different type of light penetrometer. Refer to Livneh and Ishai [14] for more detail.

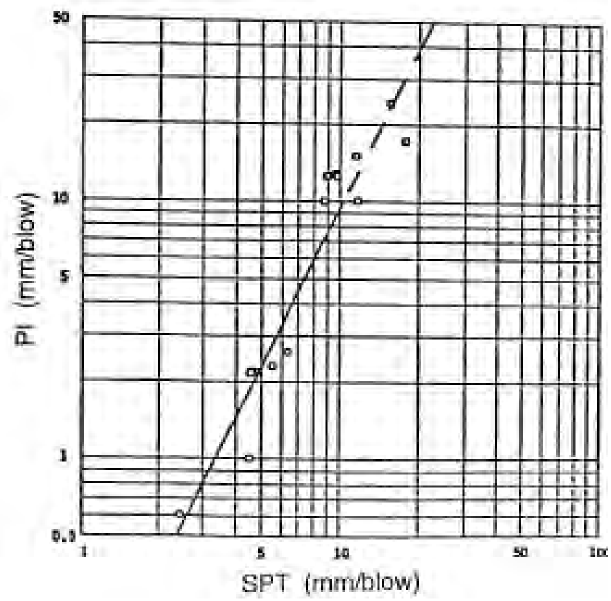


Figure 2-9 Relationship between Penetration Index (PI) and SPT [14]

Currently, the most widely used correlation between DCP and SPT N-value is done by Transport Road Research Laboratory (TRRL), Overseas Road Note (ORN) 9, Design of small bridges [15] (Table 2-2).

Table 2-2 Typical correlation between DCP and SPT values [15]

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

2.4.3. Prediction of Other Widely Used Parameters

I. California Bearing Ratio (CBR)

The most common correlation of the DCPI is with California Bearing Ratio (CBR). The CBR is defined as the ratio of the resistance to penetration developed by a sub grade soil to that developed by a specimen of standard crushed-rock base material. Numerous graphs of DCPI-CBR correlation can be found. Form these graphs, a typical equation in the form of equation 2.10 may be written. Although moisture content and dry density can have great effects on shear strength in fine soils, these properties are typically neglected in this correlation since they were found to have a similar effect on both CBR and DCPI result. During the studies of the correlation of DCPI and CBR, it was found that DCP testing can be an excellent substitute for field CBR determination [16].

$$\text{Log}_{10}(\text{CBR}) = A - B\,\text{Log}_{10}(\text{DCPI}) \quad (2.10)$$

There are many parties which tried to determine the values of the constant and the coefficient in the above equation. Two equations are listed below.

Equation by Kleyn (1983) as cited by [17]:

$$\text{Log}_{10}(\text{CBR}) = 2.632 - 1.28\,\text{Log}_{10}(\text{DCPI}) \quad (2.11)$$

Equation by TRRL [17]:

$$\text{Log}_{10}(\text{CBR}) = 2.48 - 1.057\,\text{Log}_{10}(\text{DCPI}) \quad (2.12)$$

II. Elastic Modulus

Some researchers have tried to establish correlation between DCPI and the elastic modulus of a soil. De Beer [18] proposed equations of a form similar to CBR equations (Figure 2-10).

$$Log_{10}(E_{eff}) = A - B\ Log_{10}(DCPI) \qquad (2.13)$$

Where E_{eff} is effective elastic modulus (MPa) and DCPI is Dynamic Cone Penetration Index (mm/blow).

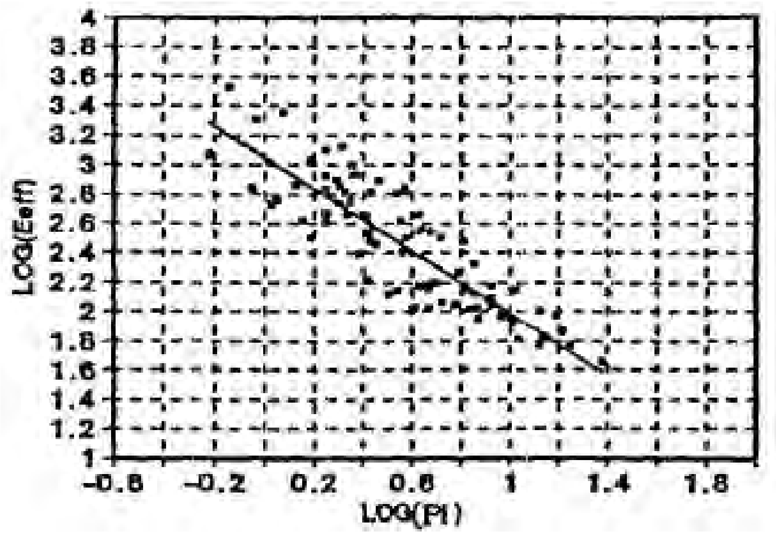


Figure 2-10 Empirical relationship between effective elastic modulus (MPa) and DCPI (mm/blow) [18]

2.5. Bearing Capacity Theory and Shear Resistance of Penetrometers

2.5.1. Bearing Capacity Theory

An important problem of foundation engineering is the computation of the maximum load (the bearing capacity) and Coulomb's method 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 [19].

Based on this theory, Prandtl (1920) as cited by [19] described the punching resistance of an ideal plastic medium. 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.

$$p_c = (\pi + 2)c = 5.14c \qquad (2.14)$$

After this finding, many others incorporated the influence of the depth of the foundation and other parameters in to the equation. Influence of depth of the foundation was accounted for by considering a surcharge at the foundation level, to the left and the right of the applied load. The foundation pressure is denoted by p. The surcharge q, next to the foundation, is supposed to be given (refer to Figure 2-11). It can be used to represent the effect of the depth of the foundation (d) below the soil surface. In that case q = γd, where γ is the unit weight of the soil.

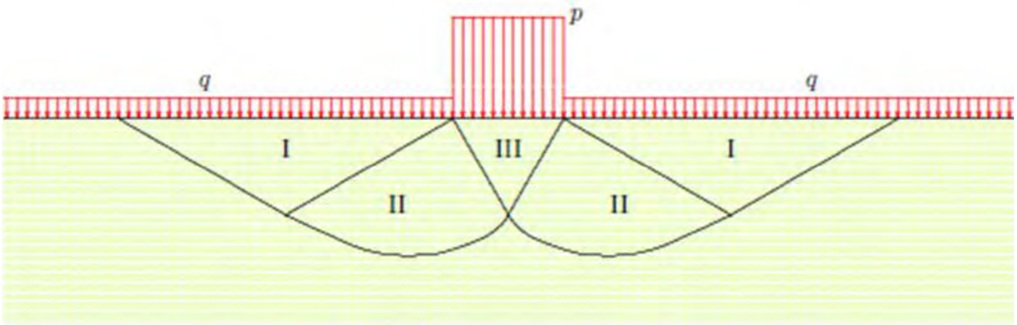


Figure 2-11 Schematization Prandtl's of strip foundation as cited by [19]

The results of the analysis of the three zones can be written as

$$p = cN_c + qN_q \qquad (2.15)$$

Where the coefficients N_c and N_q are dimensionless constants, for which Prandtl (1920) as cited by [19] obtained the following expressions,

$$N_q = \frac{1 + \sin\phi}{1 - \sin\phi} \exp(\pi \tan\phi) \qquad (2.16)$$

$$N_c = (N_q - 1)\cot\phi \qquad (2.17)$$

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 [19]

$$p = cN_c + qN_q + 0.5B\gamma N_\gamma \qquad (2.18)$$

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 [19]

$$N_\gamma = 2(N_q - 1)\tan\phi \qquad (2.19)$$

or

$$N_\gamma = 1.5(N_q - 1)\tan\phi \qquad (2.20)$$

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 $N_\gamma = 0$. The following ultimate bearing capacity equation is used for the current thesis:

$$q_{ult} = 5.142c + \gamma h \qquad (2.21)$$

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

c = undrained shear strength (c_u)

h = depth to cone tip

γ = average unit weight of the soil

2.5.2. Shear Resistance of Penetrometers

This topic states about the theoretical principles that exist on shear resistance of penetrometers. As a cone penetration device, the DCP provides some measurement of the shear strength of a soil. Research has been conducted looking at both the forces imparted by a DCP cone tip, and the behaviour of the soil caused by the application of these forces.

DCP tip to soil interaction behaviour models are various and these models are developed to analyze soil failure caused by air-dropped projectiles. While projectiles begin with velocities of several hundred meters per second, DCP tip penetrations are considered "slow" penetrations [16].

Chua [20] formulated his modelling solution by considering the penetration of an axi-symmetric soil disc with a thickness equal to the height of the cone, similar to work by Yankexevsky and Adin as cited by [16] for projectiles. Using stresses and strains from the model, Chua developed a correlation of Penetration Index (PI) versus elastic modulus for various types of soils.

Chua and Lytton [21] also performed a "structural system" type dynamic analysis including both the DCP and its soil interaction. In the analysis, the DCP is modelled as a series of springs and masses, and the soil as dashpot. Acceleration and damping analyses were conducted, along with measuring the peak acceleration of the device (1400 G). It was also shown that it is possible to determine damping properties of in-situ pavement materials through DCP testing.

Basically the theoretical aspect of the successive penetrations caused by the hammer drop is that outlined in the classic study of bearing capacity failure as discussed in section 2.5.1. Before the cone point is forced into the level of the soil to be tested, the soil is in a state of elastic equilibrium. When the cone point is forced to the test level the soil passes into a state of plastic equilibrium with the cone point becoming the element forming part or all of Zone I (Figure 2-12). Assuming an ideal soil and a smooth cone point, the zone of plastic equilibrium is subdivided into a cone-shaped zone (later displaced by the penetrometer point), an annular zone of radial shear emanating from the outer edges of the cone, and an annular passive Rankine zone. The dashed lines on the right-hand side of the same indicate the boundaries of Zones I to III at the failure stage or Penetrometer movement, and the solid lines represent the same boundaries after the cone point has moved into the level being tested [1].

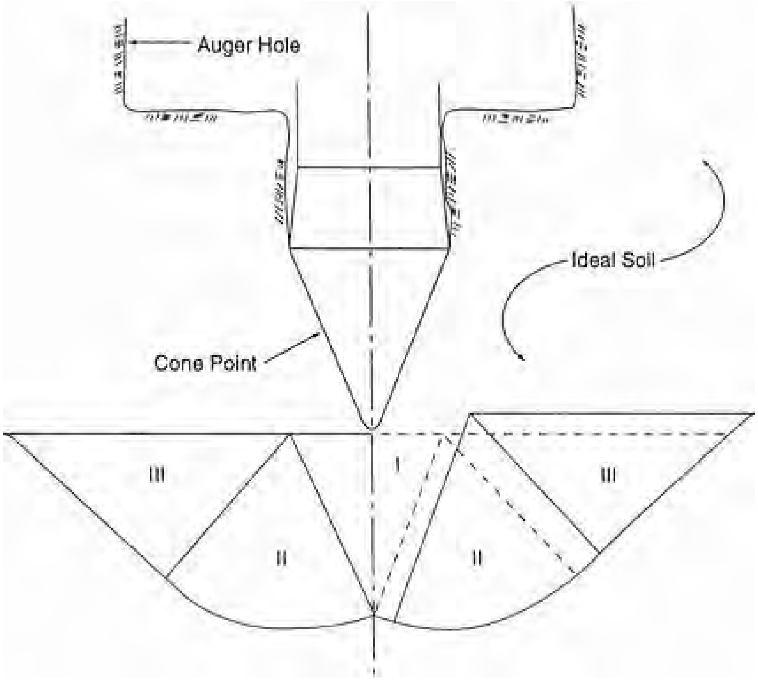


Figure 2-12 Theoretical principles of penetration equipment [1]

3. METHODS, DATA COLLECTION ANDRESULTANALYSIS AND DISCUSSION

3.1. GENERAL

The subsurface strata encountered in the test pits, test methods used and other supplementary information is provided in this section. The visual description and identification of soils are done on the site and stated as "soil data" at the top of penetration data report on appendix A. After the visual description and identification of soils, the samples, both disturbed and undisturbed, were taken to the laboratory as per the procedure stated in Table 3.1. Other tests were also done according to the methods and standards stated in section 3.2.

3.2.TEST METHODS

3.2.1. Field Identification, Tests and Sampling Methods

Table 3-1 Field test description and sampling methods used [17 and 22]

Test Description	Test Procedure
Description and Identification of Soils (Visual-Manual Procedure)	ASTM D 2488
Dynamic Cone Penetration test	Overseas Road Note 8, a User Manual for a Program to Analyse Dynamic Cone Penetration Data, TRRL, 1992[17]/ASTM D 6951-03
Soil Sampling	Representative disturbed soil samples were collected using Polythene bag 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)

3.2.2. Laboratory Test Methods

Table 3-2 Laboratory test description and sampling methods used [22, 23 and 24]

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

Test Descriptions	Test Procedure
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 [24]
Classification System	ASTM D-2487

3.3. DATA COLLECTION AND TEST RESULTS

The locations of site to be investigated were selected based on previous researches done on the engineering properties of Addis Ababa soils. The locations were made to cover the two main clayey soil categories, i.e., area covered by red clay and black cotton /expansive/ soils (Figure 3-1).

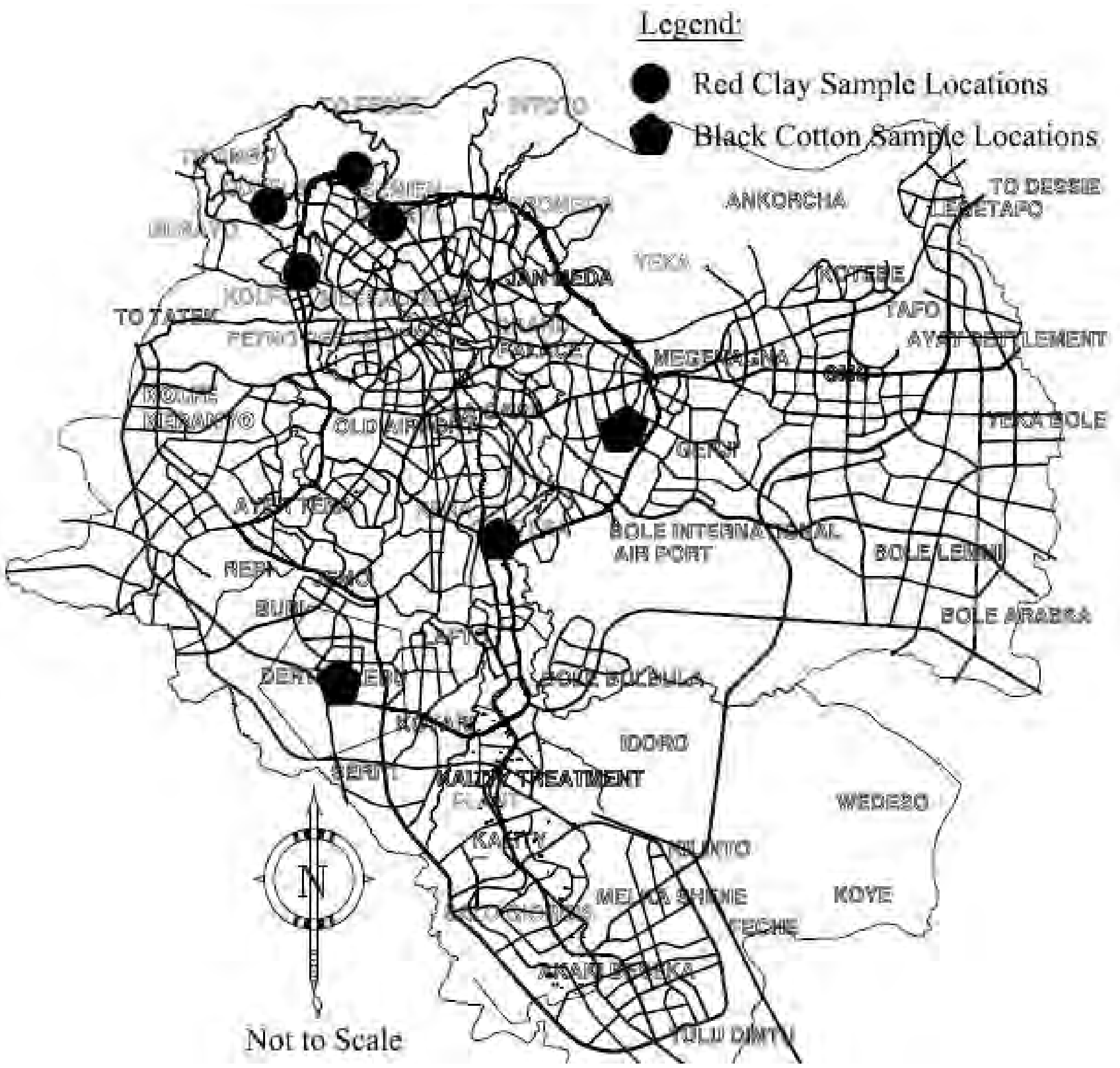


Figure 3-1 Locations of data collection in Addis Ababa

The field and laboratory data conducted outside Addis Ababa were also used to study the effect of parameters like plasticity index, liquidity index, natural water content, bulk density and depth on the dynamic cone penetration index and were taken from Tetra Tech, American consulting firm hired by the USAID for designing 92 new health centres and renovating 300 existing health centres in Amhara, Oromiya and SNNP regional states. The firm that took the sub-consultancy of the detail site investigation (EDGE consulting firm) conducted insitu field test using DCP and other equipment like Shelby and augur for sampling. Classification and shear strength tests were also conducted in the laboratory.

The researcher has participated in most of these site investigations and the results were found to add to the betterment of the research. So, the results were analysed accordingly and whenever the site is found to lie within the scope of the study it is incorporated into the study. Therefore, for analysis and correlation development purpose the three categories will be

Category-1) clayey soils including both red clay and black cotton soils of Addis Ababa combined

Category-2) Red clay soils of Addis Ababa and

Category-3) Clayey soils after incorporating data outside of Addis Ababa

For sites outside of Addis Ababa, equipment manufactured by Gonduwana Engineering PLC was used. This equipment, unlike the DCP manufactured by the author, supports extension rods. Refer to Appendix C for general overview of work done.

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 (Figure 3-2).The figure shows how the first drops do not accurately represent the average dynamic cone penetration index (DCPI) at 0.0, 1.0, and 1.6m. Thus, the first two drops, the seating drops, are disregarded. 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 [25].

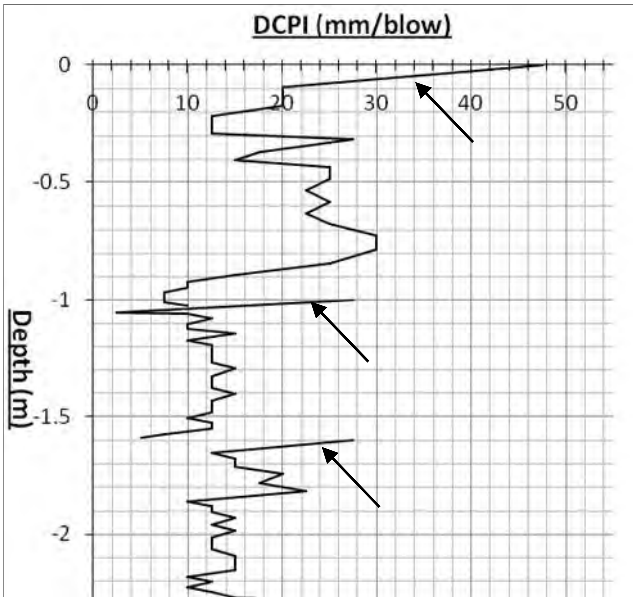


Figure 3-2 Dynamic Cone Penetration Index (DCPI) versus depth for Addisu Gebeya indicating the unreliability of initial drops

The Penetration data obtained in the field was conducted using the procedure stated in Table 3.1. The data was analyzed 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 insitu characteristics in an enhanced way. Refer to Figure 3-3 for the software interface. The details of field penetration data are presented in Appendix A.

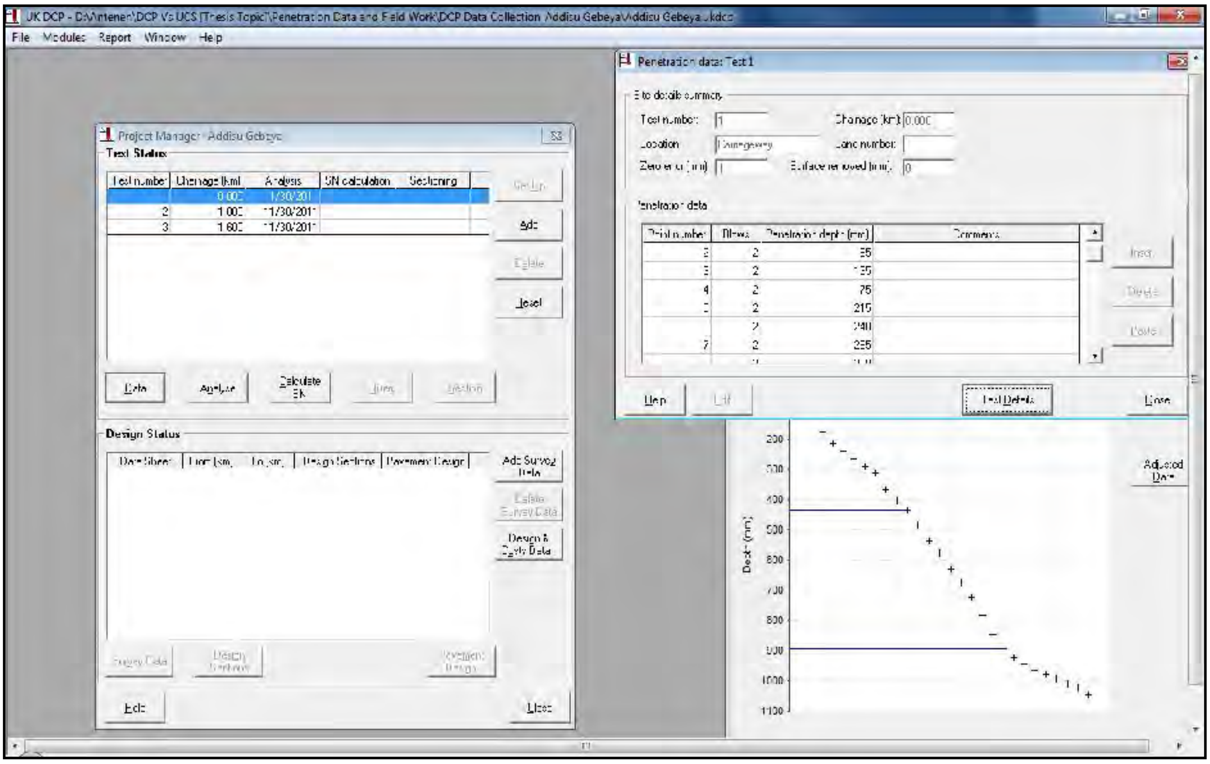


Figure 3-3 Penetration data analysis software interface

The summery of all the field and laboratory data used are presented in Table 3-3 and 3-4. Table 3-3 is the result of tests conducted by the researcher in Addis Ababa. Table 3-4 is the result obtained from the consulting firm EDGE outside of Addis Ababa. As stated previously, the data obtained outside Addis Ababa was only used to study the effect of parameters like plasticity index, liquid limit, natural water content, bulk density and depth on the dynamic cone penetration index. The details of laboratory test results are presented in Appendix B.

3.4. SUMMARY OF TEST RESULTS

Table 3-3 Summary of test results for sites inside Addis Ababa

Site Designation	Depth (m)	Natural Moisture content (%)	Specific gravity	Clay Fraction (%)	Liquid Limit (%)	Plasticity Index (%)	Liquidity index $=[\mu_L-\mu_n]/PI$	Free swell (%)	γ_{bulk} (kN/m^3)	UCS (kN/m^2)	DCPI (mm/blow)	USCS Classif.
01) Near Imperial hotel_[I_1]	3.0	23.56	2.46	57.0	102.70	50.82	1.56	190	16.51	326	20	*MH*
02) Near Imperial hotel_[I_2]	3.0	32.87	2.49	*57.0*	107.75	54.87	1.36	240	15.30	144	30	*MH*
03) Lebu site (near the road to Alem Tena)_[L_1]	3.00	26.53	2.50	N/A	81.00	38.00	1.43	130.00	15.33	195.00	35.00	*MH*
04) Lebu site (near the road to Alem Tena)_[L_2]	3.00	31.56	2.41	41.0	109.00	52.00	1.49	160.00	14.99	86.00	48.00	*MH*
05) Addisu Gebeya_[AG]	2.00	18.65	2.64	N/A	70.80	34.80	1.50	*N/A*	18.32	248.18	13.00	*MH*
06) Atari_[AT_1]	1.50	30.20	2.78	N/A	65.50	28.80	1.23	*N/A*	19.06	181.27	21.00	*MH*
07) Atari_[AT_2]	3.00	29.20	2.79	N/A	70.30	27.70	1.48	*N/A*	18.81	108.26	19.00	*MH*
08) Aweliya_[AW_1]	1.50	27.00	2.85	45.7	56.30	24.10	1.22	*N/A*	17.63	128.74	12.50	*MH*
09) Aweliya_[AW_2]	2.40	35.20	2.90	55.3	68.40	28.70	1.16	*N/A*	17.97	226.86	14.00	*MH*
010) Aweliya_[AW_3]	2.50	31.59	2.90	55.3	68.40	28.70	1.28	*N/A*	17.89	257.24	11.00	*MH*
011) Aweliya_[AW_4]	3.10	32.74	2.85	76.0	58.30	28.70	0.89	*N/A*	16.98	146.06	22.00	*CH*
012) Addis Ketema (Woreda 10)_[AK_1]	0.25	21.00	*N/A*	*N/A*	*N/A*	*N/A*	*N/A*	*N/A*	15.09	341.15	8.00	*MH*
013) Addis Ketema (Woreda 10)_[AK_2]	0.35	21.80	*N/A*	*N/A*	*N/A*	*N/A*	*N/A*	*N/A*	15.11	341.01	9.00	*MH*
014) Addis Ketema (Woreda 10)_[AK_3]	1.55	32.50	2.82	60.5	60.00	19.00	1.45	*N/A*	18.75	222.93	13.00	*MH*
015) Addis Ketema (Woreda 10)_[AK_4]	1.65	32.80	2.82	60.5	60.00	19.00	1.43	*N/A*	18.25	200.23	15.00	*MH*
016) Addis Ketema (Woreda 10)_[AK_5]	2.60	39.70	2.84	56.6	63.80	20.00	1.21	*N/A*	17.47	141.20	20.00	*MH*
017) Meskel Flower (Woreda 03)_[MF_1]	3.0	36.97	2.56	82.9	109.80	67.00	1.09	250.00	17.40	139.30	20.11	*CH*
018) Meskel Flower (Woreda 03)_[MF_2]	2.60	31.56	*2.56*	*82.9*	*109.80*	*67.00*	*1.17*	*250.00*	18.04	163.85	11.11	*CH*

Table 3-4 Summary of test results for sites outside of Addis Ababa [EDGE Consulting Firm]

Site Designation	Depth (m)	Natural Moisture content (%)	% Passing 75«m	Liquid Limit (%)	Plasticity Index (%)	Liquidity index =[μ_L-μ_n]/PI	Free swell (%)	¥$_{bulk}$ (kN/m^3)	UCS (kN/m^2)	DCPI (mm/blow)	USCS Classification
01) Ego (Arisi) [E_1]	1.1	N/A	91	79	38		140	N/A	N/A	40	MH
02) Ego(Arisi) [E_2]	1.5	N/A	93	79	40		130	N/A	N/A	21.85	MH
03) Ego(Arisi) [E_3]	1.05	N/A	93	75	38		140	N/A	N/A	29.62	MH
04) Gado Guna (Arisi) [GG_{1-1}]	0.5	N/A	96	88	53		140	N/A	N/A	37	CH
05) Gado Guna (Arisi) [GG_{1-2}]	2.15	N/A	89	87	44		170	N/A	N/A	9.13	MH
06) Gado Guna (Arisi) [GG_{2-1}]	0.6	42.83	97	96	59	0.90	150	22.56	N/A	36.76	CH
07) Gado Guna (Arisi) [GG_{2-2}]	2.15	N/A	89	94	58		150	N/A	N/A	8.97	CH
08) Gado Guna (Arisi) [GG_{3-1}]	0.6	N/A	94	85	48		130	N/A	N/A	37	CH
09) Gado Guna (Arisi) [GG_{3-2}]	2.15	N/A	90	88	45		160	N/A	N/A	18.27	MH
10) Gebrekirstos (Arisi) [GK_2]	0.4	N/A	69	53	16		20	N/A	N/A	50.94	MH
11) Solo Madero (Arisi) [SM_{1-1}]	0.75	39.88	95	83	43	1.00	140	19.13	N/A	18	MH
12) Solo Madero (Arisi) [SM_{1-2}]	2.3	N/A	98	78	43		140	N/A	N/A	16	CH
13) Solo Madero (Arisi) [SM_{2-1}]	0.75	N/A	96	85	42		120	N/A	N/A	25	MH
14) Solo Madero (Arisi) [SM_{2-2}]	2.3	N/A	97	99	57		130	N/A	N/A	18.33	CH
15) Solo Madero (Arisi) [SM_{3-1}]	0.75	N/A	92	80	42		120	N/A	N/A	20	MH
16) Solo Madero (Arisi) [SM_{3-2}]	2.3	N/A	97	83	41		130	N/A	N/A	19.3	MH
17) Sacha (Kafa) [SA_1]	1	N/A	93	65	23		N/A	N/A	N/A	27	MH
18) Sacha (Kafa) [SA_2]	2.35	N/A	65	61	24		N/A	N/A	N/A	16	MH
19) Garbicho (Sidama) [GR_1]	0.65	N/A	93	55	17		N/A	N/A	N/A	45	MH
20) Garbicho (Sidama) [GR_2]	2.1	N/A	97	61	19		N/A	N/A	N/A	13.75	MH
21) Kabate (Arisi) [KA_1]	1	N/A	86	70	36		80	N/A	N/A	7	CH
22) Kabate (Arisi) [KA_2]	0.95	N/A	86	66	31		90	N/A	N/A	12	MH
23) Kabate (Arisi) [KA_3]	0.95	N/A	88	64	29		80	N/A	N/A	12	MH
24) Sibeya (Hadiya) [SB_1]	2	N/A	93	56	21		30	N/A	N/A	16	MH
25) Sibeya (Hadiya) [SB_{2-2}]	75	N/A	93	67	30		50	N/A	N/A	18	MH
26) Sibeya (Hadiya) [SB_{3-2}]	2.1	N/A	92	58	21		50	N/A	N/A	13	MH
27) Mehal Jarso (Hadiya) [MJ_{1-2}]	1.6	48.43	91	73	39	0.63	180	19.91	N/A	62	CH
28) Mehal Jarso (Hadiya) [MJ_{2-2}]	2.05	N/A	65	95	61		110	N/A	N/A	48	CH
29) Mehal Jarso (Hadiya) [MJ_{3-2}]	2	N/A	68	67	36		100	N/A	N/A	62	CH
30) Mehal Jarso (Hadiya) [MJ_{4-2}]	2.1	N/A	90	87	54		120	N/A	N/A	50	CH
31) Andero (Gamo Gofa) [AN]	1.45	N/A	71	67	19		60	N/A	N/A	11	MH
32) Gelsha (South Wollo) [GL_1]	0.7	N/A	90	60	19		60	N/A	N/A	10	MH
33) Gelsha (South Wollo) [GL_2]	0.7	N/A	91	61	16		40	N/A	N/A	10	MH
34) Kosero (South Wollo) [KS_{1-1}]	0.8	N/A	89	77	32		130	N/A	N/A	11.2	MH
35) Kosero (South Wollo) [KS_{1-2}]	2.6	N/A	92	75	33		90	N/A	N/A	17	MH
36) Babasit (South Wollo) [BB_2]	0.55	N/A	97	52	11		40	N/A	N/A	70	MH
37) Tardiat (South Wollo) [TA]	0.95	N/A	57	51	14		30	N/A	N/A	2.2	MH
38) Wahilo (NorthWollo) [WH_{1-2}]	1.9	N/A	62	55	16		40	N/A	N/A	5	MH
39) Kusilegna (Arisi) [KU_{1-1}]	0.7	N/A	95	73	28		130	N/A	N/A	27.3	MH
40) Kusilegna (Arisi) [KU_{1-2}]	1.6	N/A	96	60	28		60	N/A	N/A	27.3	MH
41) Kusilegna (Arisi) [KU_2]	1.4	N/A	93	60	31		60	N/A	N/A	20	CH
42) Wacho (Gurage) [W_{1-1}]	0.5	N/A	95	85	48		100	N/A	N/A	76.9	CH

Site Designation	Depth (m)	Natural Moisture content (%)	% Passing 75«m	Liquid Limit (%)	Plasticity Index (%)	Liquidity index $=[\mu_L-\mu_n]/PI$	Free swell (%)	γ_{bulk} (kN/m^3)	UCS (kN/m^2)	DCPI (mm/blow)	USCS Classification
43) Wacho (Gurage) [W_{1-2}]	1.55	N/A	95	67	21		20	N/A	N/A	31.86	MH
44) Wacho (Gurage) [W_{2-1}]	0.45	N/A	91	72	35		90	N/A	N/A	76.44	MH
45) Wacho (Gurage) [W_{2-2}]	1.55	N/A	63	60	20		N/A	N/A	N/A	21.8	MH
46) Wacho (Gurage) [W_{3-1}]	0.4	N/A	91	70	35		80	N/A	N/A	67.85	MH
47) Wacho (Gurage) [W_4]	0.4	N/A	91	80	47		90	N/A	N/A	38.27	CH
48) Mukaye Haro (East Shoa)[MH_1]	1.8	N/A	81	87	51		100	N/A	N/A	32.4	CH
49) MukayeHaro(East Shoa) [MH_{2-1}]	0.6	N/A	94	83	42		110	N/A	N/A	18.8	MH
50) MukayeHaro(East Shoa) [MH_{2-2}]	1.2	N/A	92	87	47		80	N/A	N/A	22	MH
51) MukayeHaro(East Shoa) [MH_3]	1.35	N/A	84	88	50		100	N/A	N/A	23	CH
52) Hula Arba (Arisi) [HA_1]	1	N/A	98	61	26		30	N/A	N/A	6.8	MH
53) Hula Arba (Arisi) [HA_2]	0.6	N/A	92	59	21		10	N/A	N/A	11.8	MH
54) Hula Arba (Arisi) [HA_3]	0.6	N/A	93	74	38		110	N/A	N/A	10.5	MH

3.5. Discussion of Test Results

3.5.1. General

The primary purpose of this sub-section is to 1) review all the tests results obtained in this research and compare them with previous results in the study area; 2) identify test results with erroneous values and if found necessary exclude them from the correlation.

3.5.2. Clayey Soils of Addis Ababa

Clay samples collected from the northern and north western parts of Addis Ababa are characteristically red clay soils. Those collected from southern and south eastern parts of Addis Ababa are generally found to be black cotton (expansive) clay soils. Most of the sites, according to USCS, are found to be classified as inorganic SILT with high plasticity (MH) (Refer to Figure 3-4).

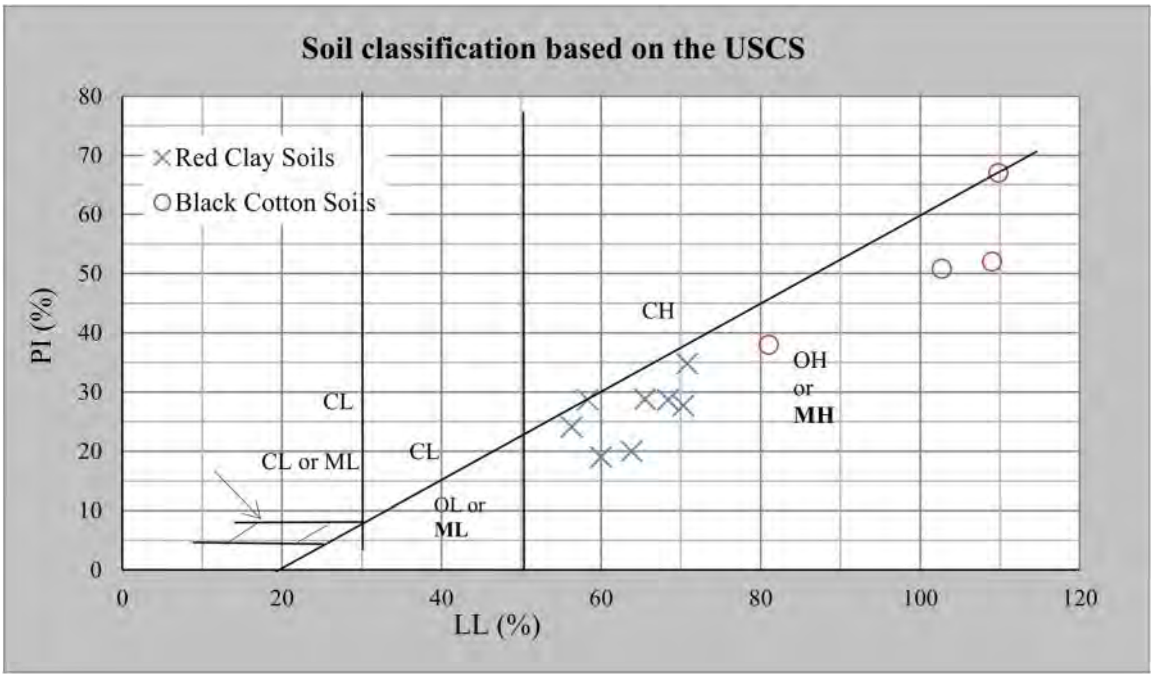


Figure 3-4 Soil classification of sites inside Addis Ababa based on USCS

Grain size analysis and other index tests were conducted on twelve (12) representative samples (i.e., eight (8) red clay soils and four (4) black cotton soils) in the current research. The plot of soil gradations conducted in the current research is presented in Figure 3-5.

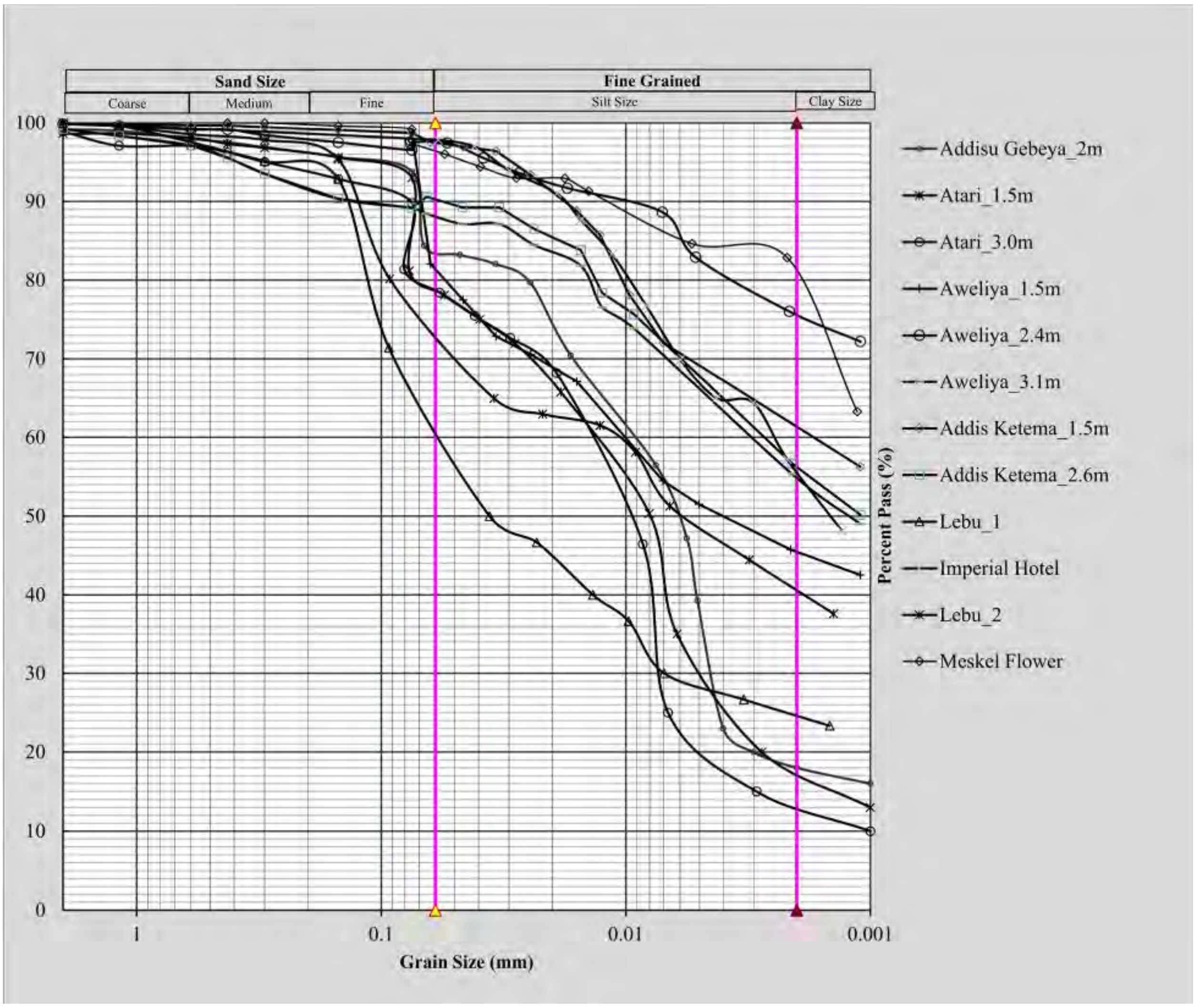


Figure 3-5 Plot of soil gradations

The index properties of Addis Ababa red clay soils from current research and previous researches are presented in Table 3-6 and that of black cotton /expansive/ soils are presented in Table 3-7 for comparison purpose.

Table 3-5 Index properties of Addis Ababa red clay soil from previous researches

Researches	Site	Clay Content (%)	LL (%)	PI (%)	Gs
Samuel Tadesse [26]	Kolfe	58 - 70	61 - 75	15 – 21	2.66 - 2.73
	Semen Gebeya	53 – 68	59 – 72	33 – 47	2.7 – 2.77
Merihun Lukas [27]	Kolfe	12 - 25	61 - 71	34 – 38	2.72 - 2.73
	Addisu Gebeya	62 – 69	63 – 72	35 – 39	2.71
Tesfay Neare [28]	Red Clay	51 - 62	47.3 - 82	23.5 – 55.5	N/A
Current Research	Red Clay	13 – 76	56.3 – 70.8	19 – 34.8	2.64 – 2.9

Table 3-6 Index properties of Addis Ababa black cotton /expansive/ clay soil from previous researches

Researches	Clay Content (%)	LL (%)	PI (%)
Tesfay Neare [28]	N/A	85 - 118	23.35 – 55.5
Daniel Teklu [30]	48 – 82	79 – 121	38 – 84
Tewold E. (1989) as cited by [31]	55-68	91-106	64-80
Legesse M. (2004) as cited by [31]	45-80	96-109	55- 77
Current Research	25 – 83	81 – 110	38 – 67

Results of DCP conducted on the clayey soils of Addis Ababa converted to an equivalent SPT N-value indicated a range between 5.4 and 33.9 (Table 2-2). In terms of shear strength, based on the SPT N-value as shown in Table 2-1, it could be described to have q_u ranging from 50 to 400kPa. In the laboratory unconfined compression test results indicate that the soils have a q_u value ranging from 86 to 341kPa.

To avoid erroneous result, the consistency of the soil (i.e., hard, very stiff, stiff, medium stiff or firm, soft and very soft) is determined from insitu SPT N-value and liquidity index according to the widely accepted procedure Bowles [11] and Janbu (1963) as cited by [26]. Then the range of expected value for undrained shear strength is estimated based on Bowles [11] and BS 5930 [27]. If the actual laboratory result obtained is found to be different from both the estimates, the value will not be included in the correlation (i.e., the value will be rejected).

For example, for the site located near Imperial Hotel (I_1) (Table 3.8) the soil has a VERY STIFF consistency based on the values obtained from the liquidity index and in a STIFF consistency based on the values obtained from in situ penetration according to Janbu (1963) as cited by [28] and Bowles [11], respectively. The consistencies obtained have estimated shear strength ranging from 150kPa to 300kPa and 100kPa to 200kPa according to BS 5930 [32] and Bowles [11], respectively. The actual value of shear strength obtained using unconfined compression strength was 326.0kPa. This indicates that actual value fall outside the estimated range.

Table 3-8 shows consistency values of the soil based on SPT N-value and liquidity index and Table 3-9 gives expected range of undrained shear strength.

Table 3-7 Consistency determinations for clayey soils of Addis Ababa

Site designation	DCPI (mm/blow)	SPT N-value based on Overseas road note*	Range of Consistency Suggested by Bowles [SPT N-value]	Range of Consistency Suggested by Jambu [%]	Consistency according to *Bowles [11], S=1*	Consistency based on Liquidity index Janbu (1963) as cited by [28]
I_1	20.0	14.0	10 - 16	>100	STIFF	VERY STIFF
I_2	30.0	8.6	6 – 9	>100	MEDIUM-STIFF	VERY STIFF
L_1	35.0	7.5	6 – 9	>100	MEDIUM-STIFF	VERY STIFF
L_2	48.0	5.4	3 – 5	>100	SOFT	VERY STIFF
AG	13.0	20.6	17 – 30	>100	VERY STIFF	VERY STIFF
AT_1	21.0	12.7	10 – 16	>100	STIFF	VERY STIFF
AT_2	19.0	14.0	10 – 16	>100	STIFF	VERY STIFF
AW_1	12.5	21.5	17 – 30	>100	VERY STIFF	VERY STIFF
AW_2	14.0	18.0	17 – 30	>100	VERY STIFF	VERY STIFF
AW_3	11.0	24.5	17 – 30	>100	VERY STIFF	VERY STIFF
AW_4	22.0	12.1	10 – 16	75 – 100	STIFF	STIFF
AK_1	8.0	33.0	>30	>100	HARD	HARD
AK_2	9.0	28.0	>30	>100	HARD	HARD
AK_3	13.0	20.6	17 – 30	>100	VERY STIFF	VERY STIFF
AK_4	15.0	17.8	17 – 30	>100	VERY STIFF	VERY STIFF
AK_5	20.0	14.0	10 – 16	>100	STIFF	VERY STIFF
MF_1	20.0	14.0	10 – 16	>100	STIFF	VERY STIFF
MF_2	11.0	24.1	17 - 30	>100	VERY STIFF	VERY STIFF

* To judge the consistency of soil from DCPI value the general practice is to convert it in to SPT N-value (Table 2-2). Then use developed relationship between SPT N-Value and consistency.

Table 3-8 Shear strength estimation based on consistency for clayey soils of Addis Ababa

Site Designation	Estimated q_u (kPa) based on Consistency		Actual q_u (kPa)
	According to Bowles [11] using SPT N-value	Using Liquidity index *BS 5930 [32]*	
I_1	*100-200*	*150-300*	***326.0 ****
I_2	50-100	150-300	144.0
L_1	50-100	150-300	195.0
L_2	25-50	150-300	86.0
AG	200-400	150-300	248.18
AT_1	100-200	150-300	181.27
AT_2	100-200	150-300	108.26
AW_1	200-400	150-300	128.74
AW_2	200-400	150-300	226.86
AW_3	200-400	150-300	257.24
AW_4	100-200	75-150	146.06
AK_1	>400	>300	341.15
AK_2	>400	>300	341.01
AK_3	200-400	150-300	222.93
AK_4	200-400	150-300	200.23
AK_5	100-200	150-300	141.2
MF_1	100-200	150-300	139.3
MF_2	100-200	150-300	163.85

G Excluded from correlation development stage

3.5.3. Clayey Soils after Incorporating Data Outside of Addis Ababa

Data selected from different part of the country were plotted along the A-line (Figure 3-6). According to USCS, the soils are classified as mostly as inorganic SILT with high plasticity (MH) and inorganic CLAY with high plasticity (CH). This criterion was used for data acceptance/rejection for this category (i.e., data that have high plasticity and more than 50% fines were incorporated).

The data collected outside of Addis Ababa were used to strengthen/support the parametric study that affect the result of dynamic cone penetration index like bulk unit weight, dry unit weight, natural moisture content, liquidity index, plasticity index, liquid limit and depth.

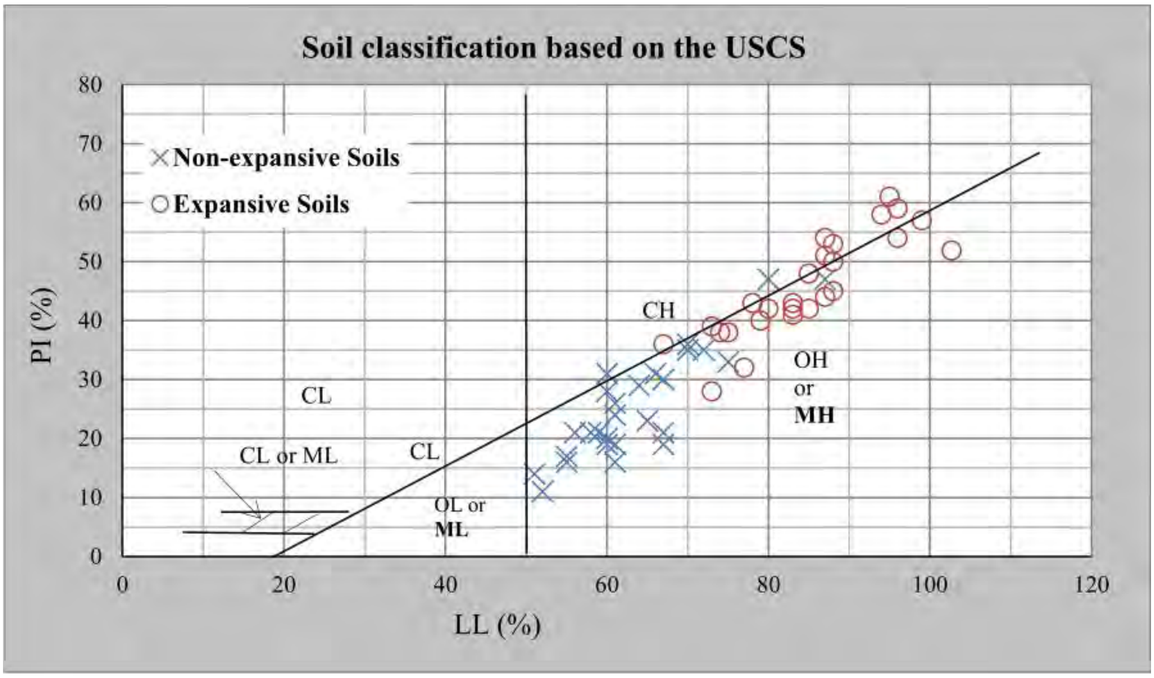


Figure 3-6 Soil classification of sites outside Addis Ababa based on USCS

4. ANALYSIS

4.1. GENERAL

Regression analysis is concerned with how the values of Y depend on the corresponding values of X. Y, whose value is to be predicted, is known as dependent variable or response 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 *single 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 observations are uncorrelated random variables with zero mean and constant variance. Tests of hypotheses and interval estimation require that the errors be normally distributed. In addition, one assumes that the order of the model is correct; that is, if one fits a simple linear regression model, one is assuming that the phenomenon actually behaves in a linear or first order manner. During regression analysis, a regression model with higher value of coefficient of determination (R^2), which quantifies the proportion of the variance of one variable by the other, is usually accepted [33].*

In this study two sets of investigations are conducted. The first set considers UCS as the dependent variable whereas DCPI, γ_{dry}, γ_{bulk}, NMC, LI, LL, PI and Depth 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 data being analyzed. 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 included unconfined compressive strength, dynamic cone penetration index, bulk unit weight, dry unit weight, natural water content, liquidity index, plasticity index, liquid limit and depth. Table 4-1present the data of the researcher used for analysis while Table 4-2 present the data used for analysis from external source.

Table 4-1 Data of the researcher used for analysis

Site Designation	UCS (kN/m^2)	DCPI (mm/blow)	$¥_{bulk}$ (kN/m^3)	$¥_{dry}$ (kN/m^3)	NMC (%)	Liquidity index =[w_L-w_n]/PI	LL (%)	PI (%)	Depth (m)
02) I_2*	**144.0**	**30.0**	**15.30**	**11.51**	**32.87**	**1.36**	**107.75**	**54.87**	**3.00**
03) L_1*	**195.00**	**35.0**	**15.33**	**12.12**	**26.53**	**1.43**	**81.0**	**38.0**	**3.00**
04) L_2*	**86.00**	**48.0**	**14.99**	**11.85**	**26.53**	**1.59**	**109.0**	**52.0**	**3.00**
05) AG	248.18	13.0	18.32	15.44	18.65	1.50	70.8	34.8	2.00
06) AT_1	181.27	21.0	19.06	14.64	30.20	1.23	65.5	28.8	1.50
07) AT_2	108.26	19.0	18.81	14.56	29.20	1.48	70.3	27.7	3.00
08)AW_1	128.74	12.5	17.63	13.88	27.00	1.22	56.3	24.1	1.50
09) AW_2	226.86	14.0	17.97	13.29	35.20	1.16	68.4	28.7	2.40
10) AW_3	257.24	11.0	17.89	13.60	31.59	1.28	68.4	28.7	2.50
11) AW_4	146.06	22.0	16.98	12.79	32.74	0.89	58.3	28.7	3.10
12) AK_1	341.15	8.0	15.09	12.47	21.00	2.05	60.0	19.0	0.25
13) AK_2	341.01	9.0	15.11	12.41	21.80	2.01	60.0	19.0	0.35
14) AK_3	222.93	13.0	18.75	14.15	32.50	1.45	60.0	19.0	1.55
15) AK_4	200.23	15.0	18.25	13.74	32.80	1.43	60.0	19.0	1.65
16) AK_5	141.20	20.0	17.47	12.51	39.70	1.21	63.8	20.0	2.60
17) MF_1*	**139.30**	**20.0**	**17.40**	**12.70**	**36.97**	**1.09**	**109.8**	**67.0**	**3.00**
18) MF_2*	**163.85**	**11.0**	**18.04**	**13.71**	**31.56**	**1.17**	**109.8**	**67.0**	**2.60**

* Black cotton (expansive) clayey soils

Table 4-2 Data from external source used for analysis [EDGE Consulting Firm]

Site designation	DCPI (mm/blow)	$¥_{bulk}$ (kN/m^3)	$¥_{dry}$ (kN/m^3)	NMC (%)	Liquidity index =(μ_L-μ_n)/PI	LL (%)	PI (%)	Depth (m)
01) E_1	40	n/a		n/a		79	38	1.1
02) E_2	21.85	n/a		n/a		79	40	1.5
03) E_3	29.62	n/a		n/a		75	38	1.05
04) GG_{1-1}	37	n/a		n/a		88	53	0.5
05) GG_{1-2}	9.13	n/a		n/a		87	44	2.15
06) GG_{2-1}	36.76	22.56	15.80	42.83	0.90	96	59	0.6
07) GG_{2-2}	8.97	n/a		n/a		94	58	2.15
08) GG_{3-1}	37	n/a		n/a		85	48	0.6

Site designation	DCPI (mm/blow)	γ_{bulk} (kN/m^3)	γ_{dry} (kN/m^3)	NMC (%)	Liquidity index =(μ_L-μ_n)/PI	LL (%)	PI (%)	Depth (m)
09) GG_{3-2}	18.27	n/a		n/a		88	45	2.15
010) GK_2	50.94	n/a		n/a		53	16	0.4
011) SM_{1-1}	18	19.13	13.68	39.88	1.00	83	43	0.75
12) SM_{1-2}	16	n/a		n/a		78	43	2.3
13) SM_{2-1}	25	n/a		n/a		85	42	0.75
14) SM_{2-2}	18.33	n/a		n/a		99	57	2.3
15) SM_{3-1}	20	n/a		n/a		80	42	0.75
16) SM_{3-2}	19.3	n/a		n/a		83	41	2.3
17) SA_1	27	n/a		n/a		65	23	1
18) SA_2	16	n/a		n/a		61	24	2.35
19) GR_1	45	n/a		n/a		55	17	0.65
20) GR_2	13.75	n/a		n/a		61	19	2.1
21) KA_1	7	n/a		n/a		70	36	1
22) KA_2	12	n/a		n/a		66	31	0.95
23) KA_3	12	n/a		n/a		64	29	0.95
24) SB_1	16	n/a		n/a		56	21	2
25) SB_{2-2}	18	n/a		n/a		67	30	0.75
26) SB_{3-2}	13	n/a		n/a		58	21	2.1
27) MJ_{1-2}	62	19.91	13.41	48.43	0.63	73	39	1.6
28) MJ_{2-2}	48	n/a		n/a		95	61	2.05
29) MJ_{3-2}	62	n/a		n/a		67	36	2
30) MJ_{4-2}	50	n/a		n/a		87	54	2.1
31) AN	11	n/a		n/a		67	19	1.45
32) GL_1	10	n/a		n/a		60	19	0.7
33) GL_2	10	n/a		n/a		61	16	0.7
34) KS_{1-1}	11.2	n/a		n/a		77	32	0.8
35) KS_{1-2}	17	n/a		n/a		75	33	2.6
36) BB_2	70	n/a		n/a		52	11	0.55
37) TA	2.2	n/a		n/a		51	14	0.95
38) WH_{1-2}	5	n/a		n/a		55	16	1.9
39) KU_{1-1}	27.3	n/a		n/a		73	28	0.7
40) KU_{1-2}	27.3	n/a		n/a		60	28	1.6
41) KU_2	20	n/a		n/a		60	31	1.4
42) W_{1-1}	76.9	n/a		n/a		85	48	0.5
43) W_{1-2}	31.86	n/a		n/a		67	21	1.55
44) W_{2-1}	76.44	n/a		n/a		72	35	0.45
45) W_{2-2}	21.8	n/a		n/a		60	20	1.55
46) W_{3-1}	67.85	n/a		n/a		70	35	0.4
47) W_4	38.27	n/a		n/a		80	47	0.4
48) MH_1	32.4	n/a		n/a		87	51	1.8
49) MH_{2-1}	18.8	n/a		n/a		83	42	0.6
51) MH_{2-2}	22	n/a		n/a		87	47	1.2
51) MH_3	23	n/a		n/a		88	50	1.35
52) HA_1	6.8	n/a		n/a		61	26	1
53) HA_2	11.8	n/a		n/a		59	21	0.6
54) HA_3	10.5	n/a		n/a		74	38	0.6

4.2. SINGLE REGRESSION

4.2.1. Scatter Plot for Category-1 (Clayey Soils of Addis Ababa)

In developing correlations, the first step is creating a scatter plot of the data, to visually assess the strength and form of some type of relationship. In the figures below (Figure 4-1 to 4-15) the scatter plot of UCS and DCPI with γ_{dry}, γ_{bulk}, NMC, LI, PI, LL and depth are presented.

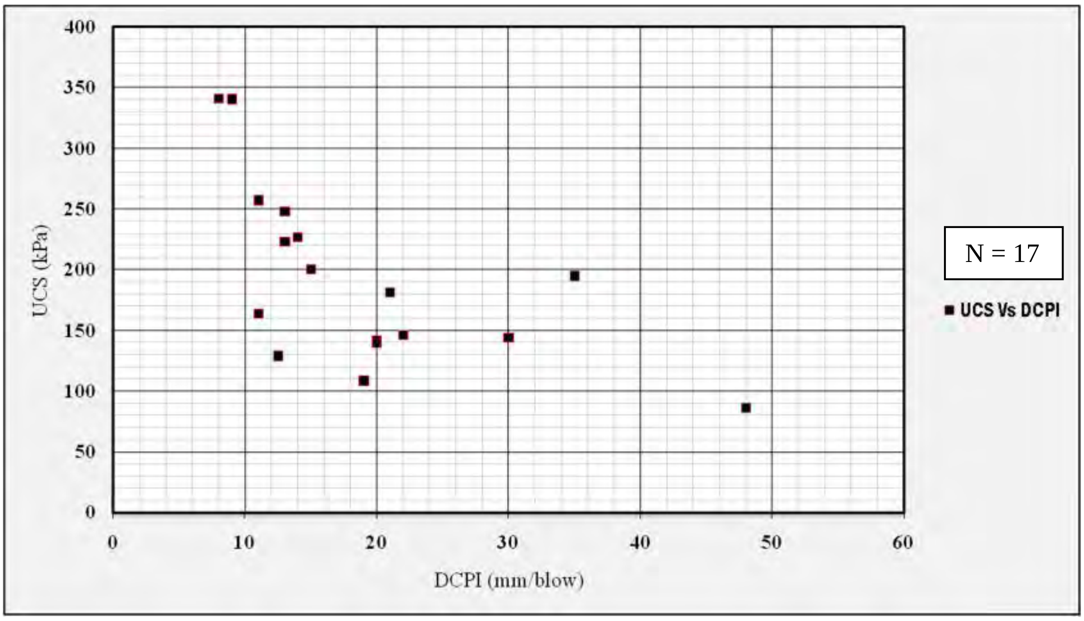


Figure 4-1 Scatter plot of UCS with DCPI for clayey soils of Addis Ababa

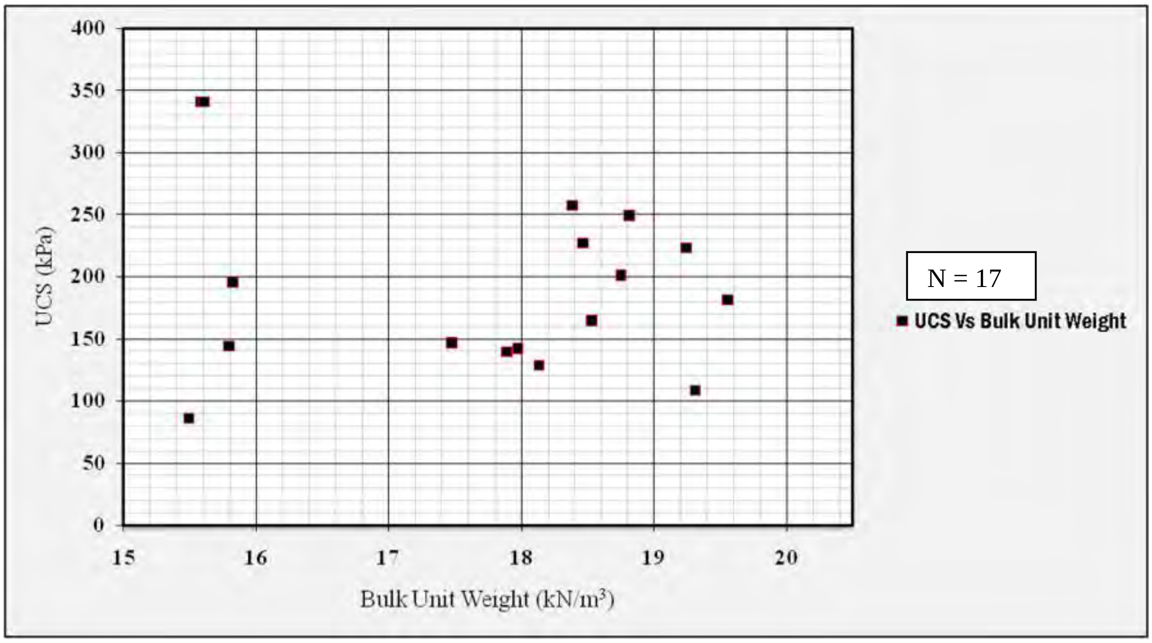


Figure 4-2 Scatter plot of UCS with bulk unit weight for clayey soils of Addis Ababa

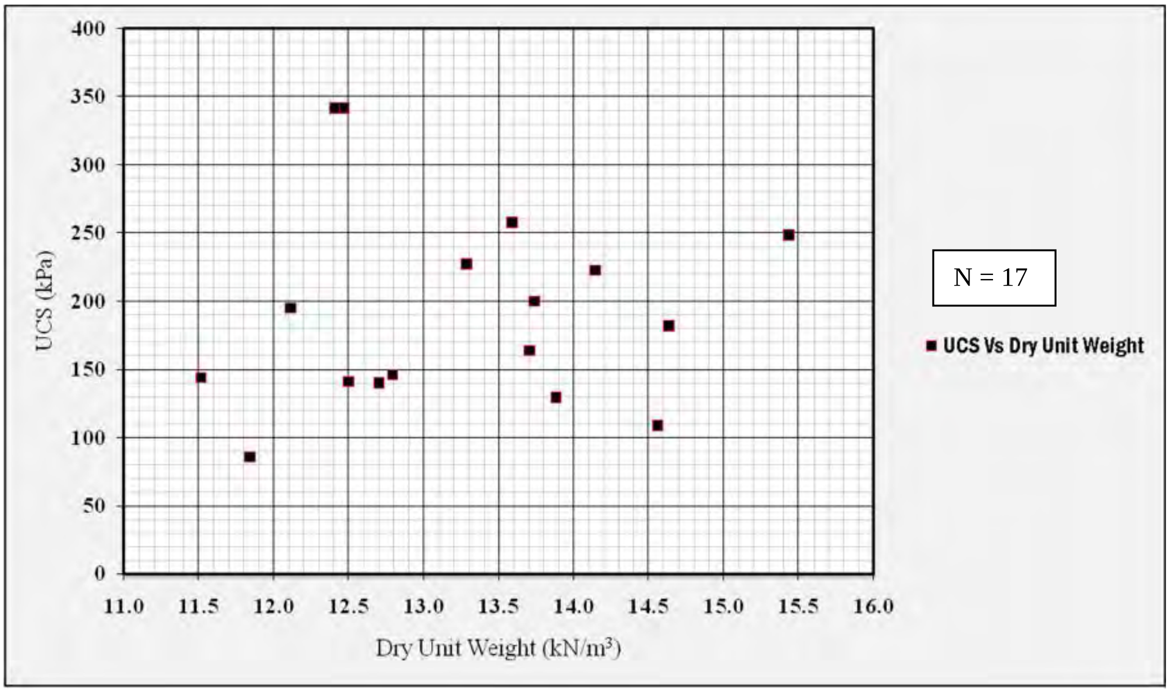


Figure 4-3 Scatter plot of UCS with dry unit weight for clayey soils of Addis Ababa

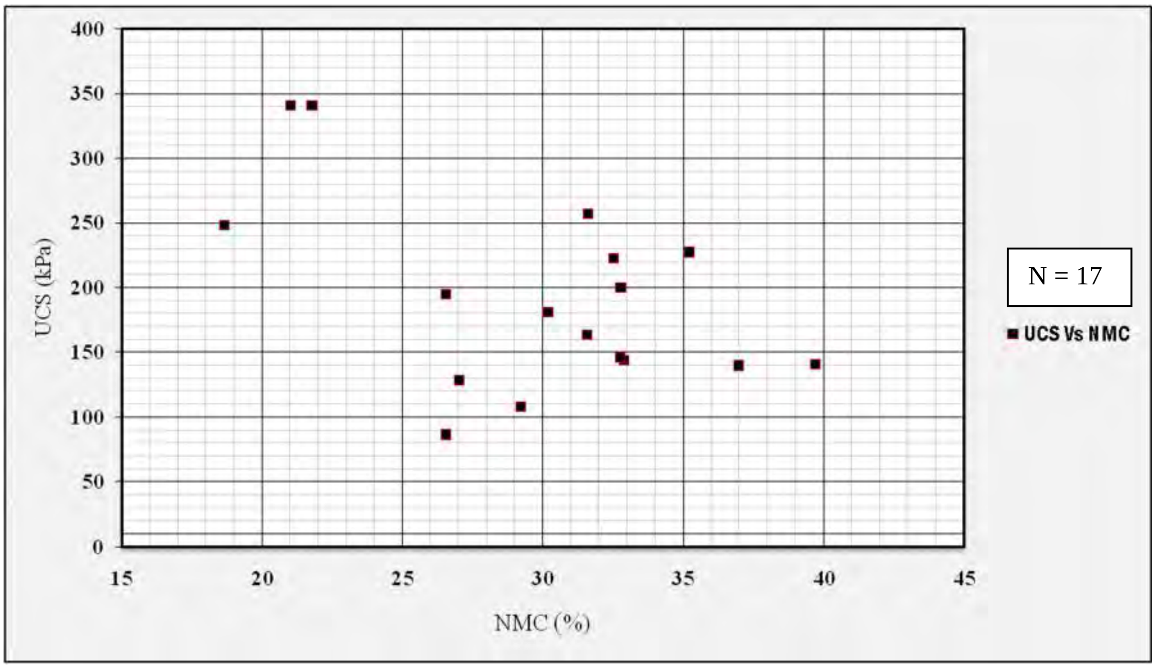


Figure 4-4 Scatter plot of UCS with natural moisture content for clayey soils of Addis Ababa

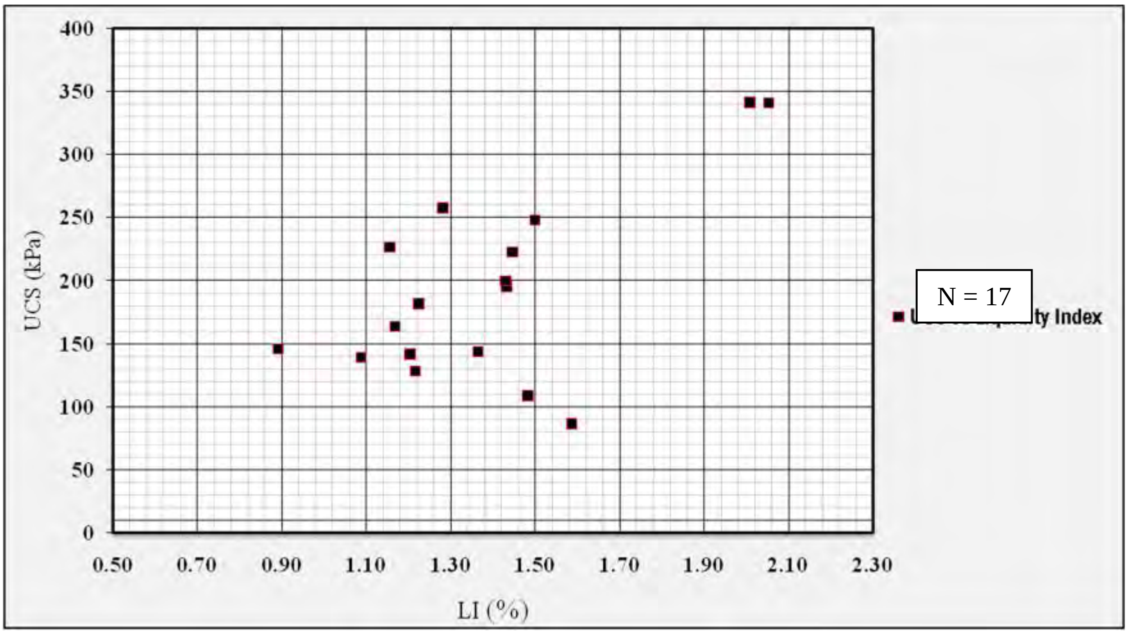


Figure 4-5 Scatter plot of UCS with liquidity index for clayey soils of Addis Ababa

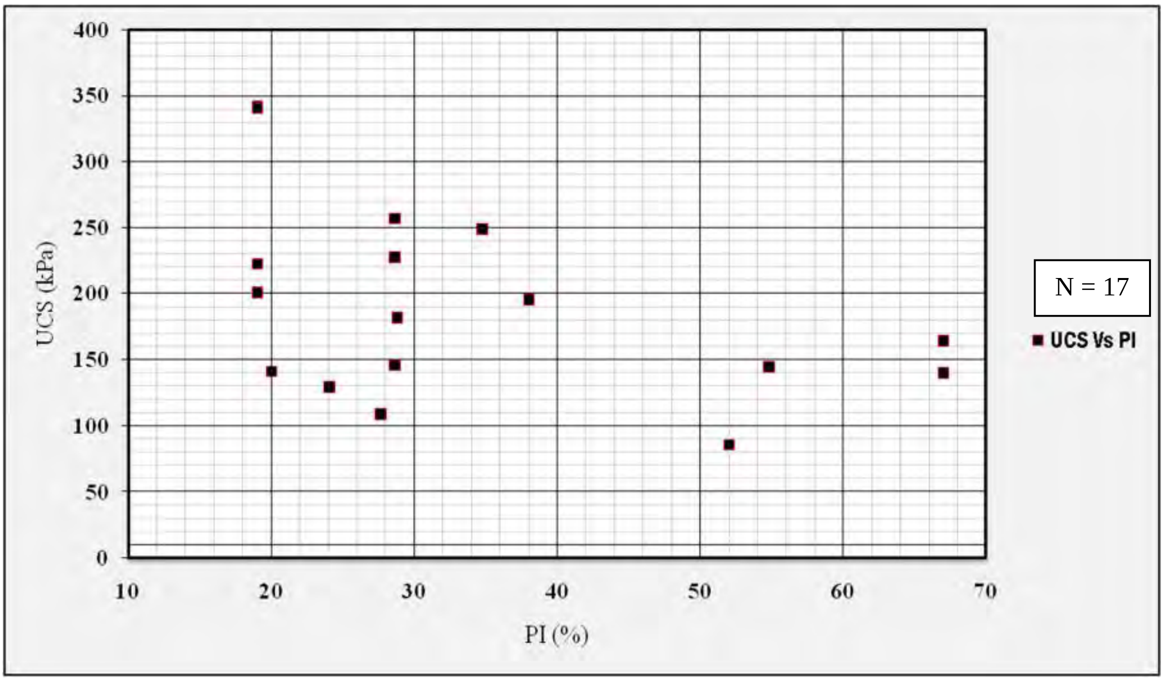


Figure 4-6 Scatter plot of UCS with plasticity index for clayey soils of Addis Ababa

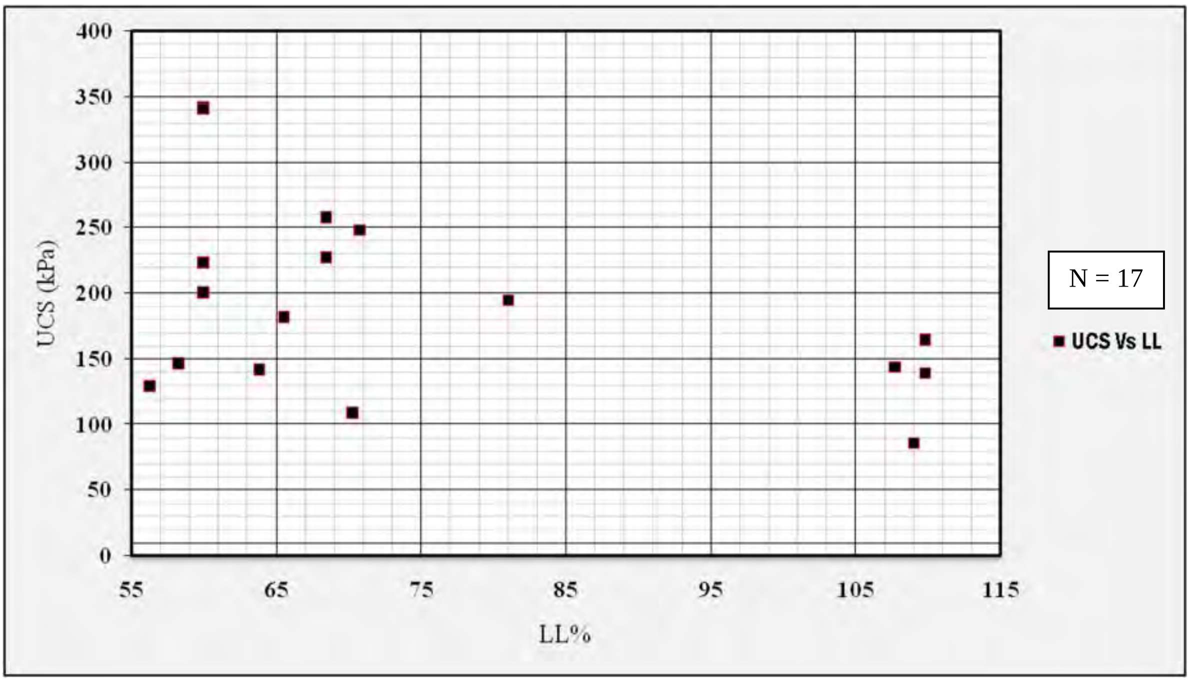


Figure 4-7 Scatter plot of UCS with liquid limit for clayey soils of Addis Ababa

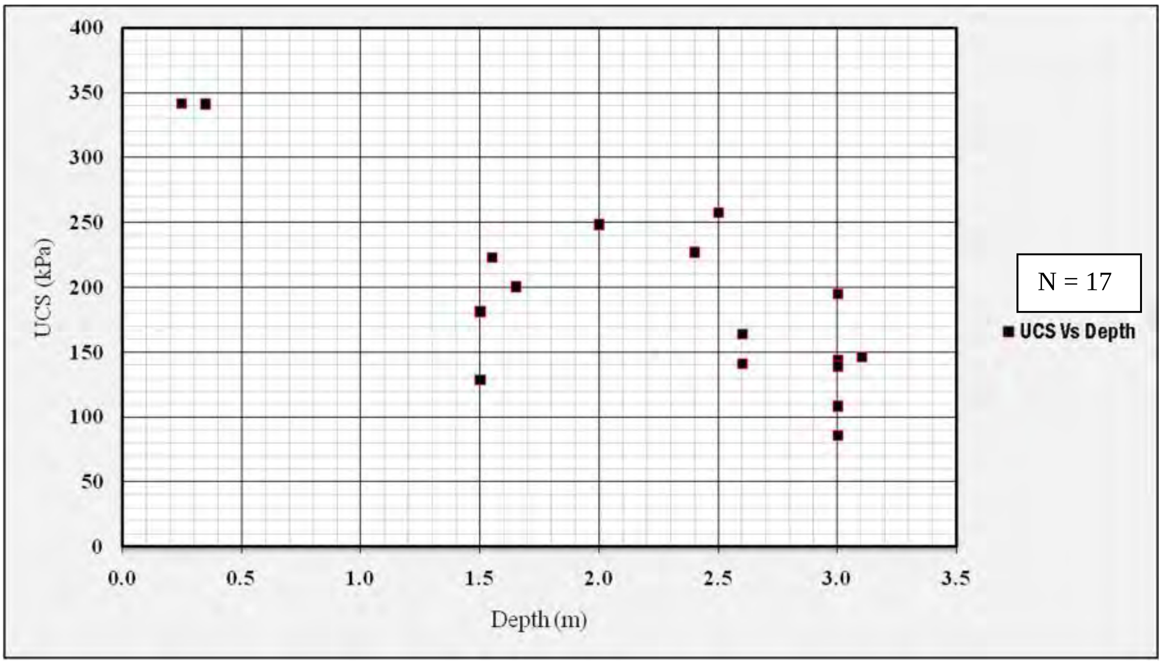


Figure 4-8 Scatter plot of UCS with depth for clayey soils of Addis Ababa

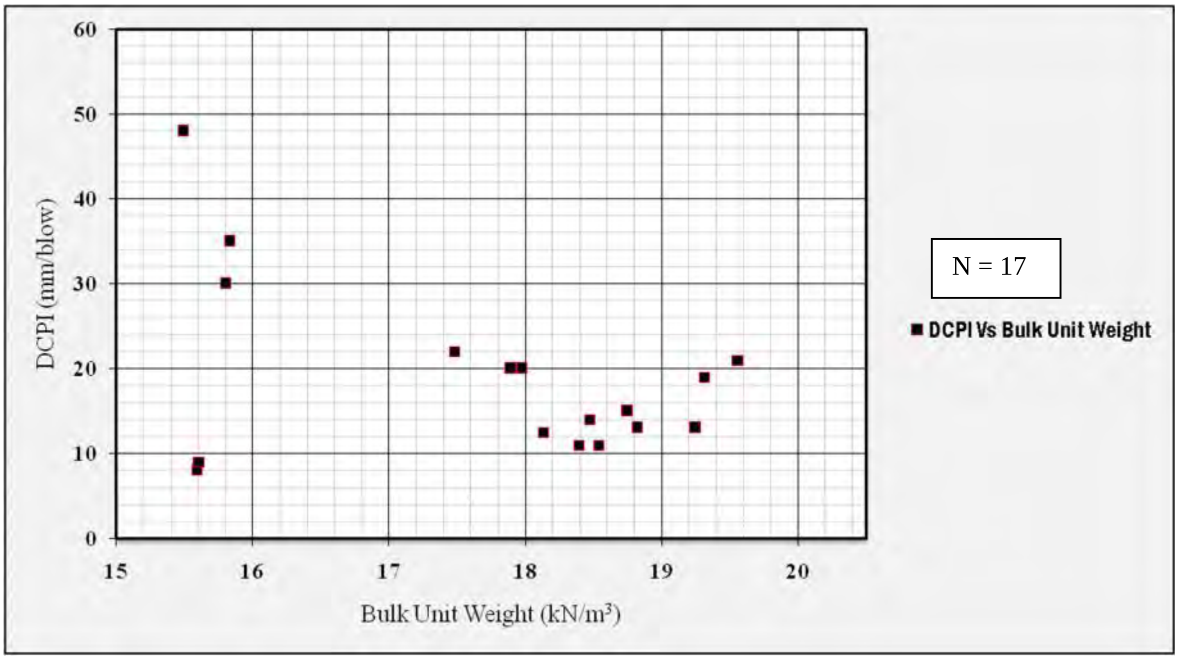


Figure 4-9 Scatter plot of DCPI with bulk unit weight for clayey soils of Addis Ababa

Figure 4-10 Scatter plot of DCPI with dry unit weight for clayey soils of Addis Ababa

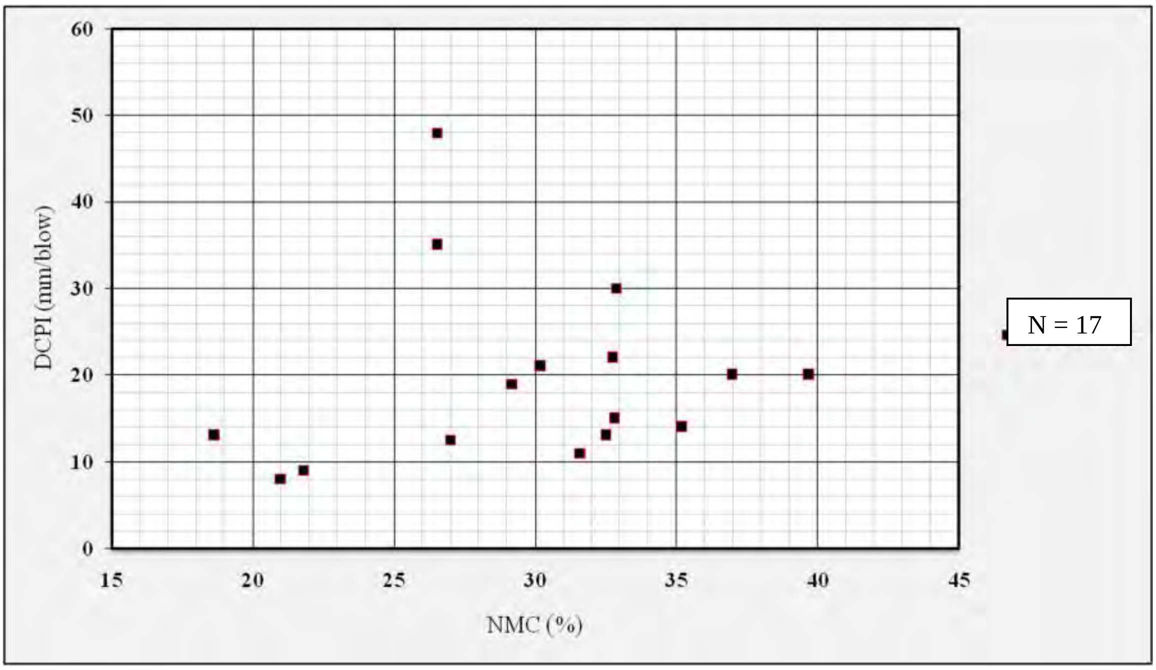


Figure 4-11 Scatter plot of DCPI with natural moisture content for clayey soils of Addis Ababa

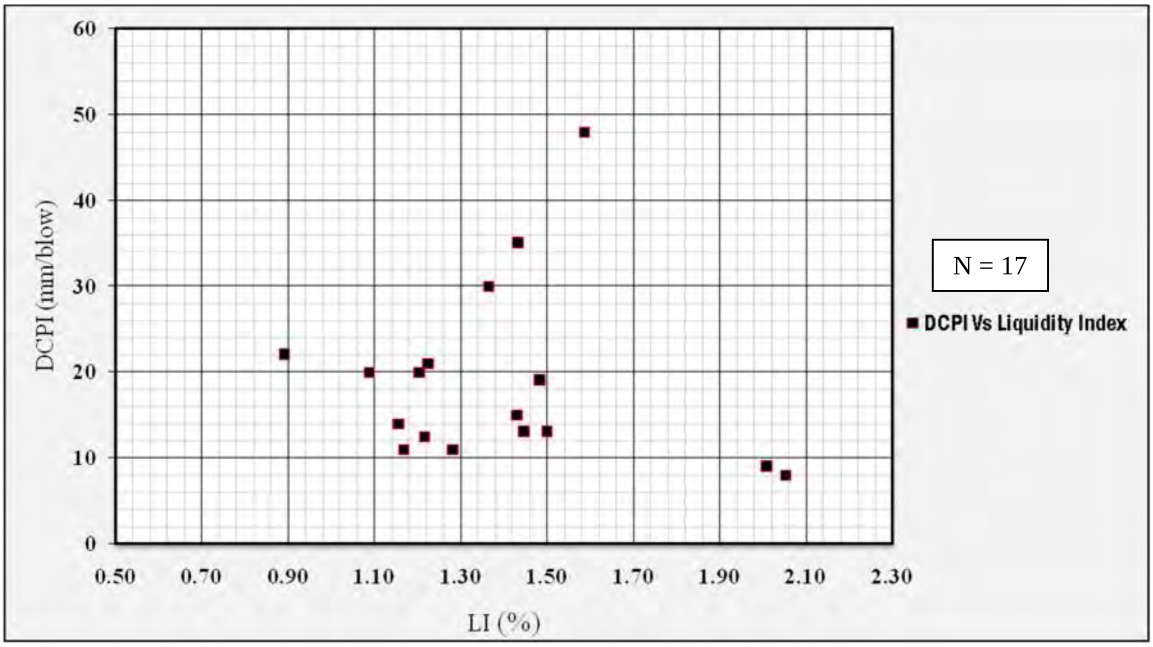


Figure 4-12 Scatter plot of DCPI with liquidity index for clayey soils of Addis Ababa

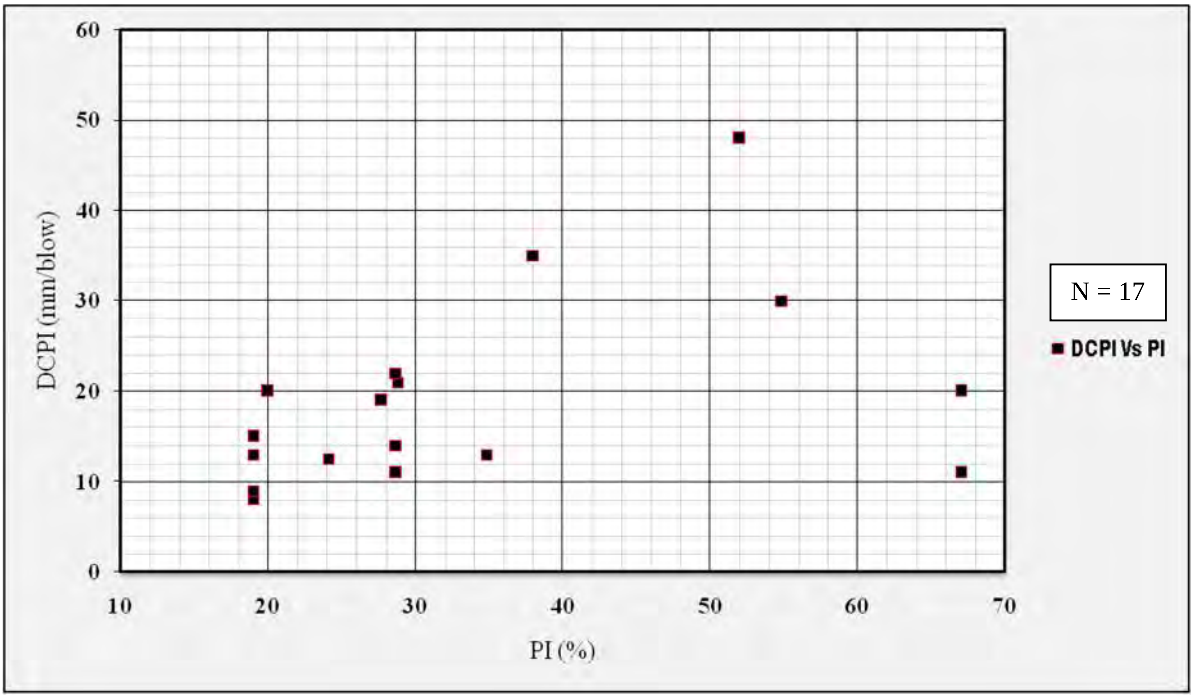


Figure 4-13 Scatter plot of DCPI with plasticity indexfor clayey soils of Addis Ababa

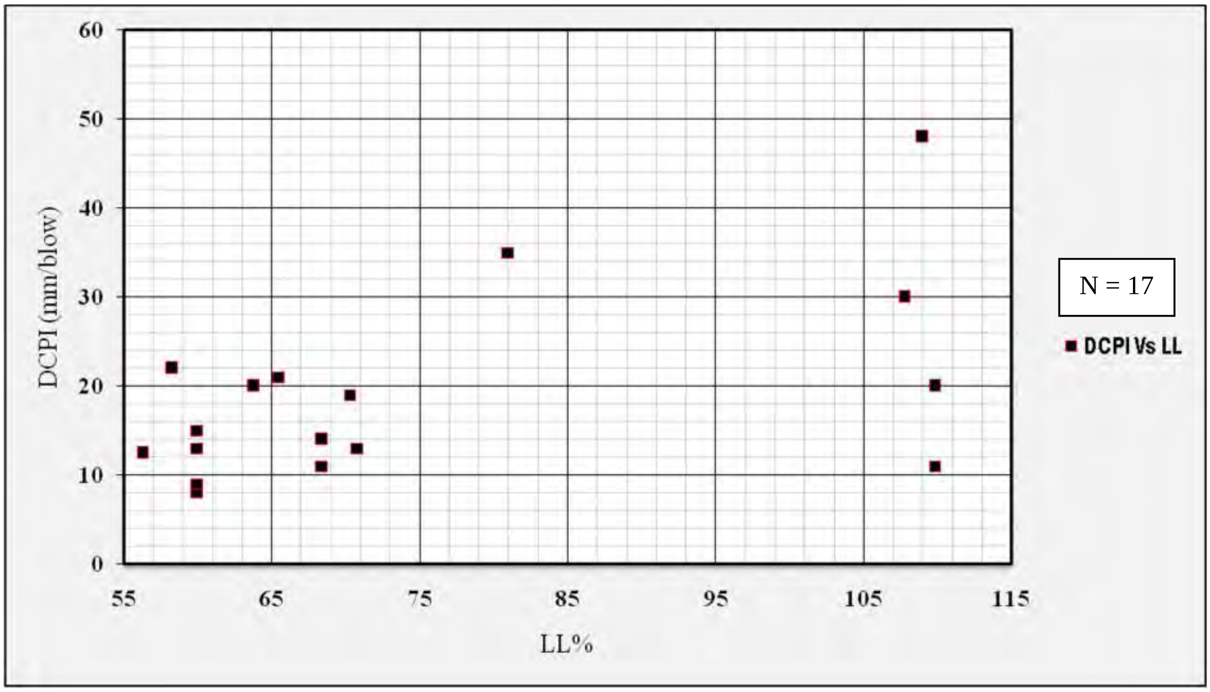


Figure 4-14 Scatter plot of DCPI with liquid limit for clayey soils of Addis Ababa

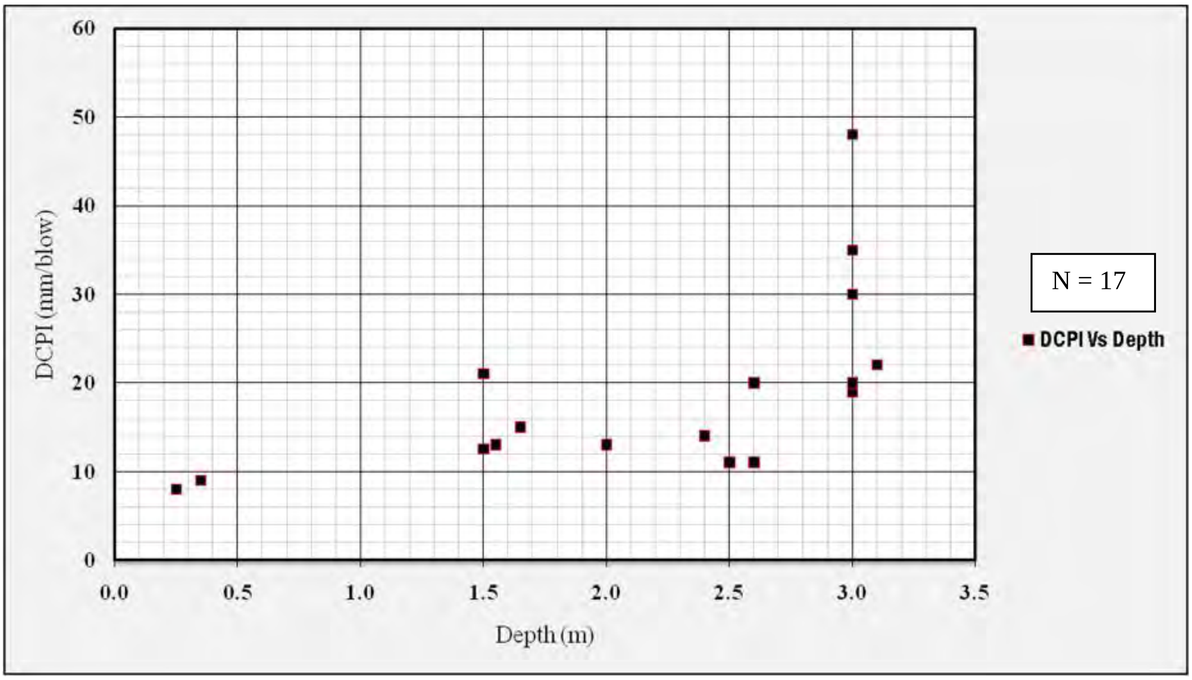


Figure 4-15 Scatter plot of DCPI with depth for clayey soils of Addis Ababa

4.2.2. Scatter Plot for Category-2 (Red Clay Soils of Addis Ababa)

Similar to 4.2.1 the scatter plots of UCS and DCPI with γ_{dry}, γ_{bulk}, NMC, LI, PI, LL and depth are presented for red clay soils of Addis Ababa (Figure 4-16 – 4-30).

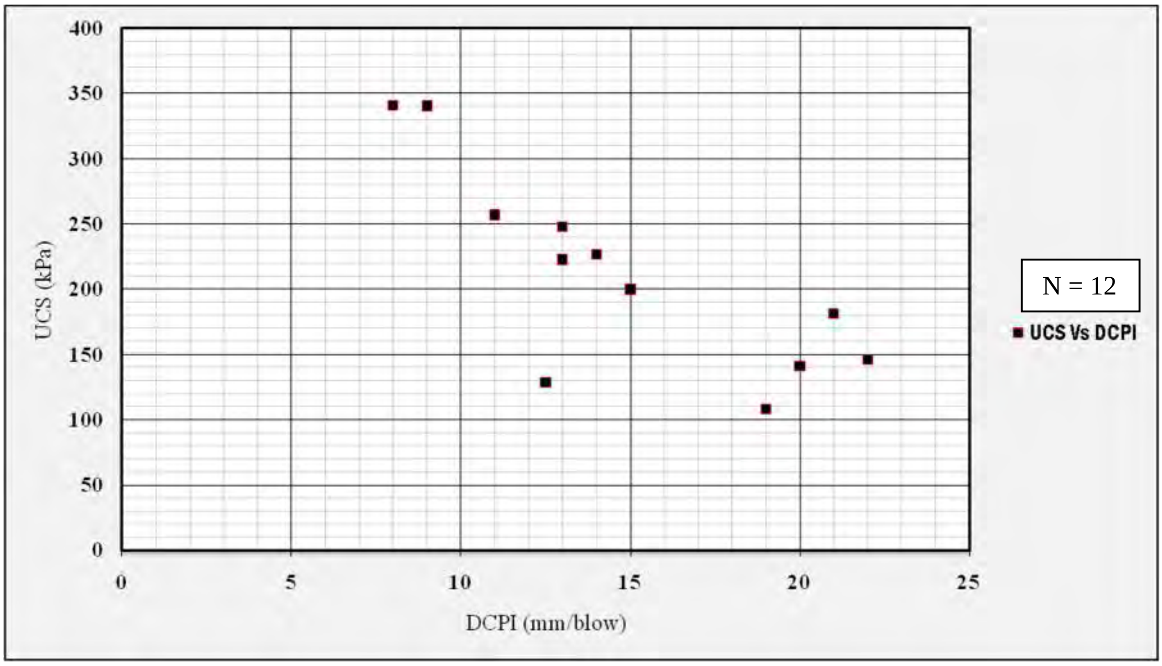


Figure 4-16 Scatter plot of UCS with DCPI for red clay soils of Addis Ababa

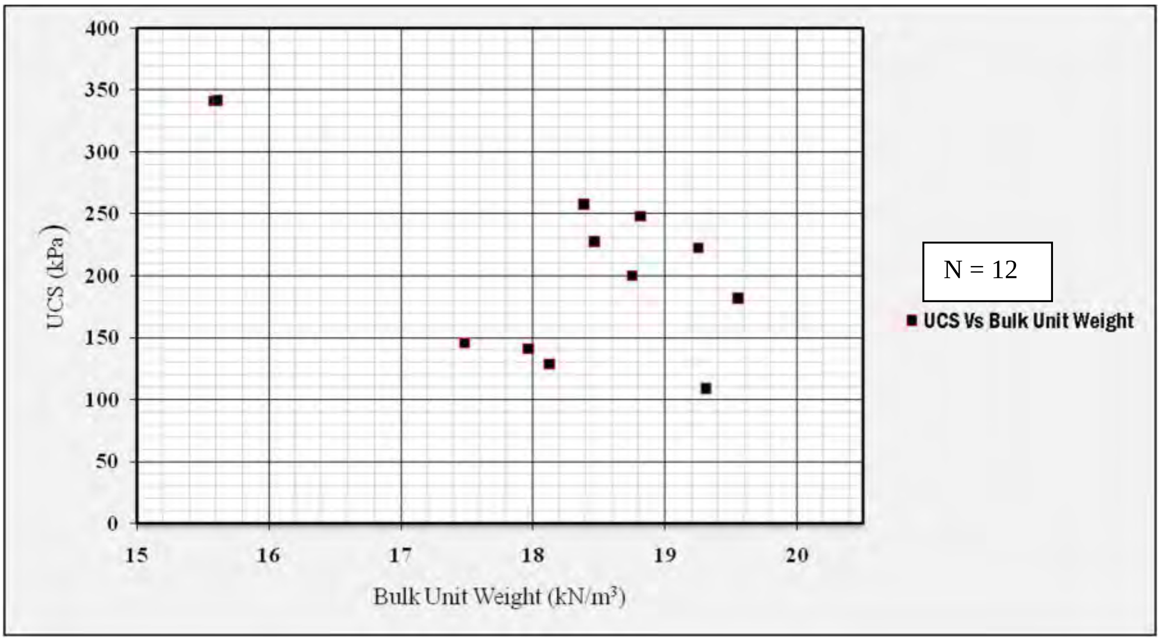


Figure 4-17 Scatter plot of UCS with bulk unit weight for red clay soils of Addis Ababa

Figure 4-18 Scatter plot of UCS with dry unit weight for red clay soils of Addis Ababa

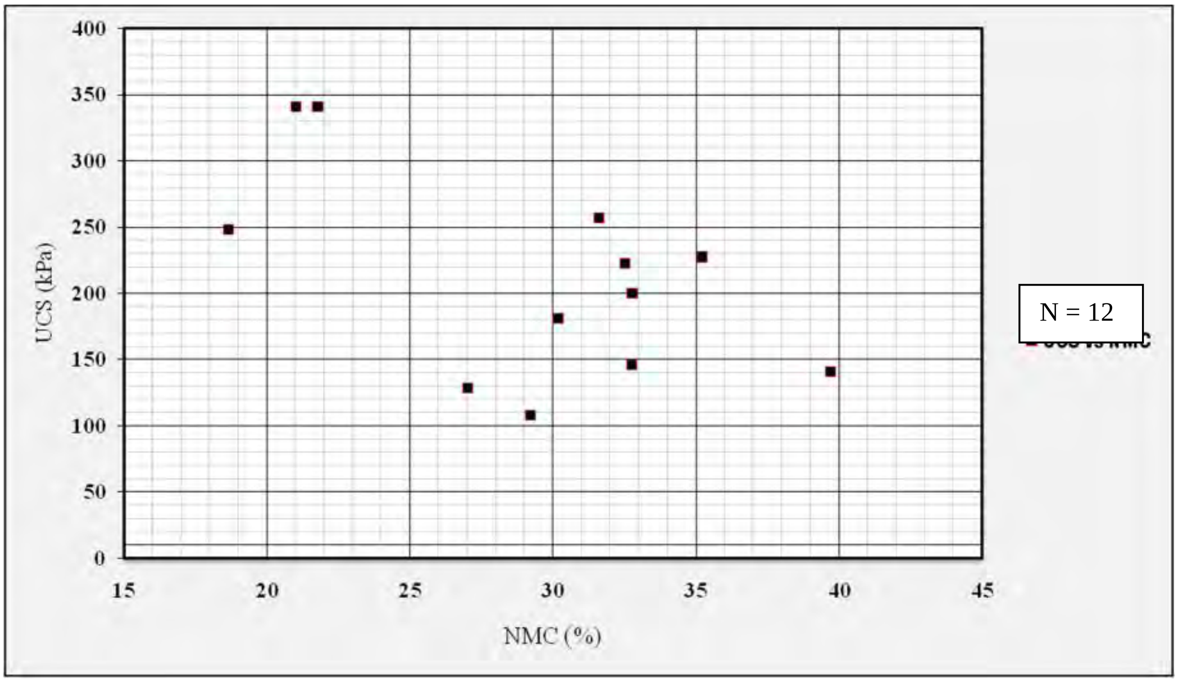


Figure 4-19 Scatter plot of UCS with natural moisture content for red clay soils of Addis Ababa

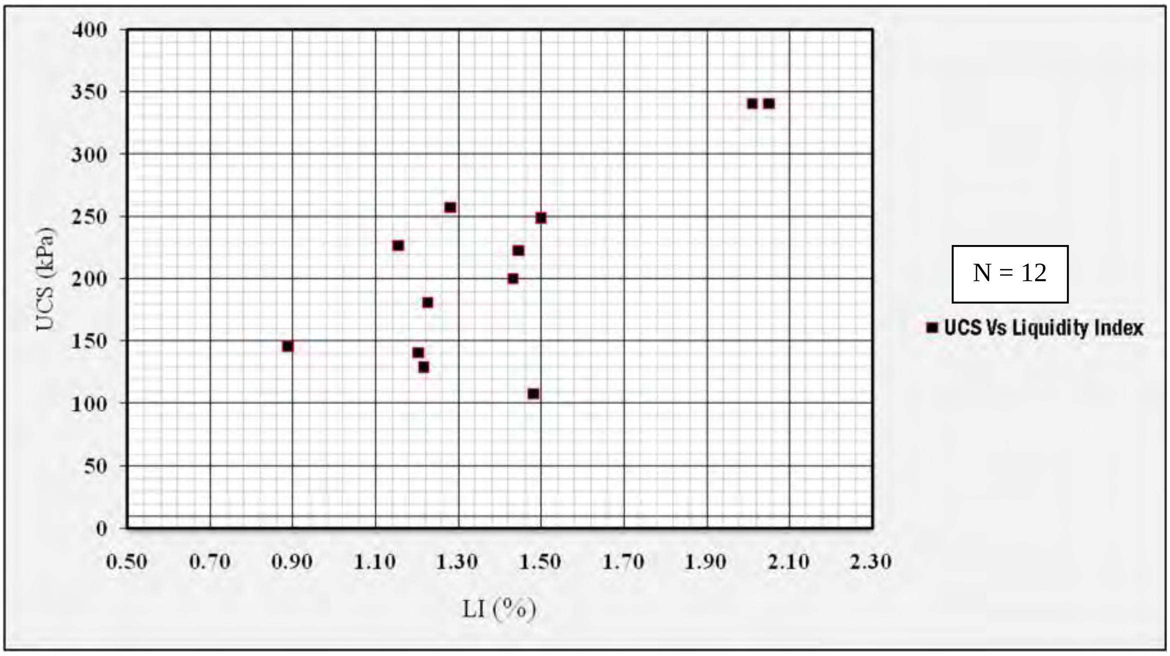


Figure 4-20 Scatter plot of UCS with Liquidity index for red clay soils of Addis Ababa

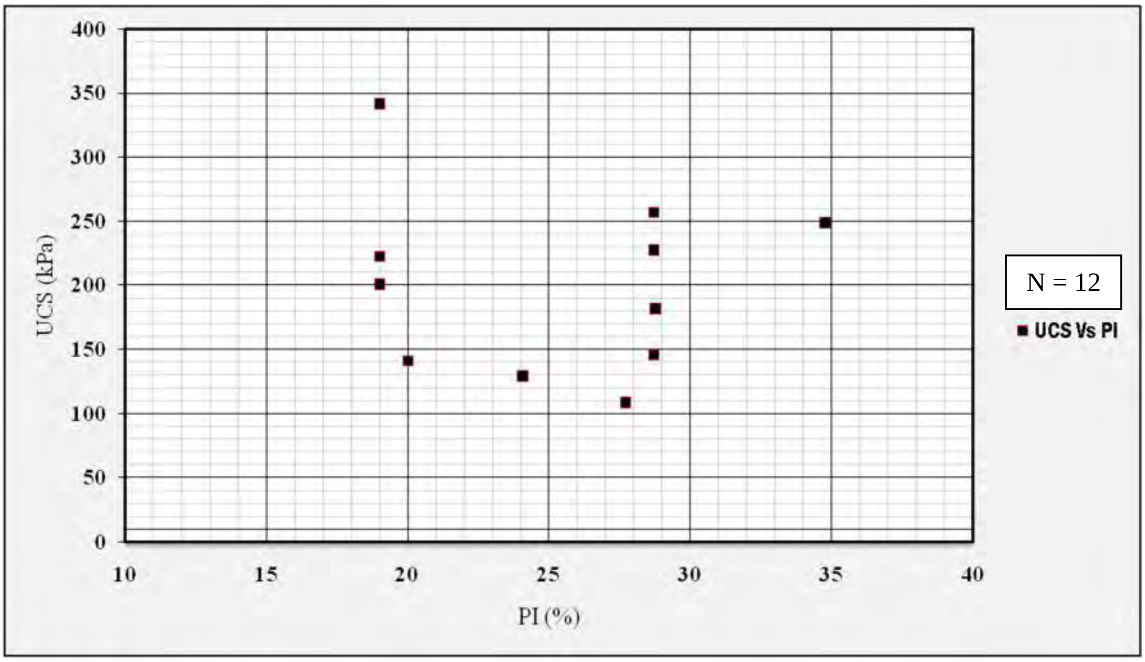


Figure 4-21 Scatter plot of UCS with Plasticity Index for red clay soils of Addis Ababa

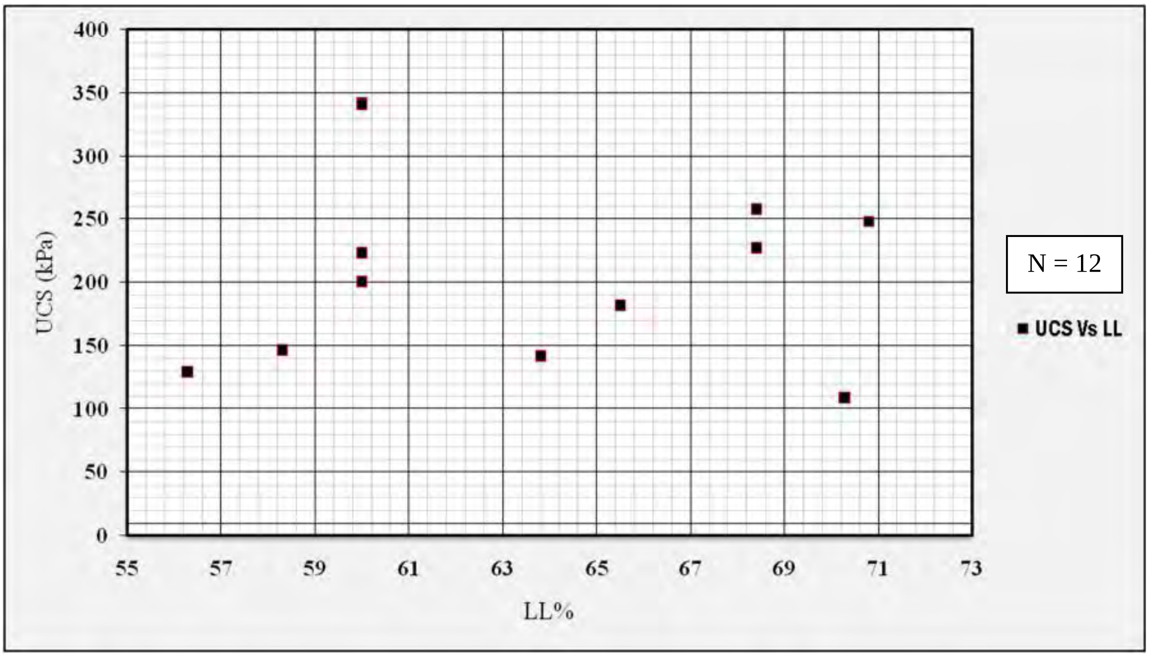


Figure 4-22 Scatter plot of UCS with Liquid Limit for red clay soils of Addis Ababa

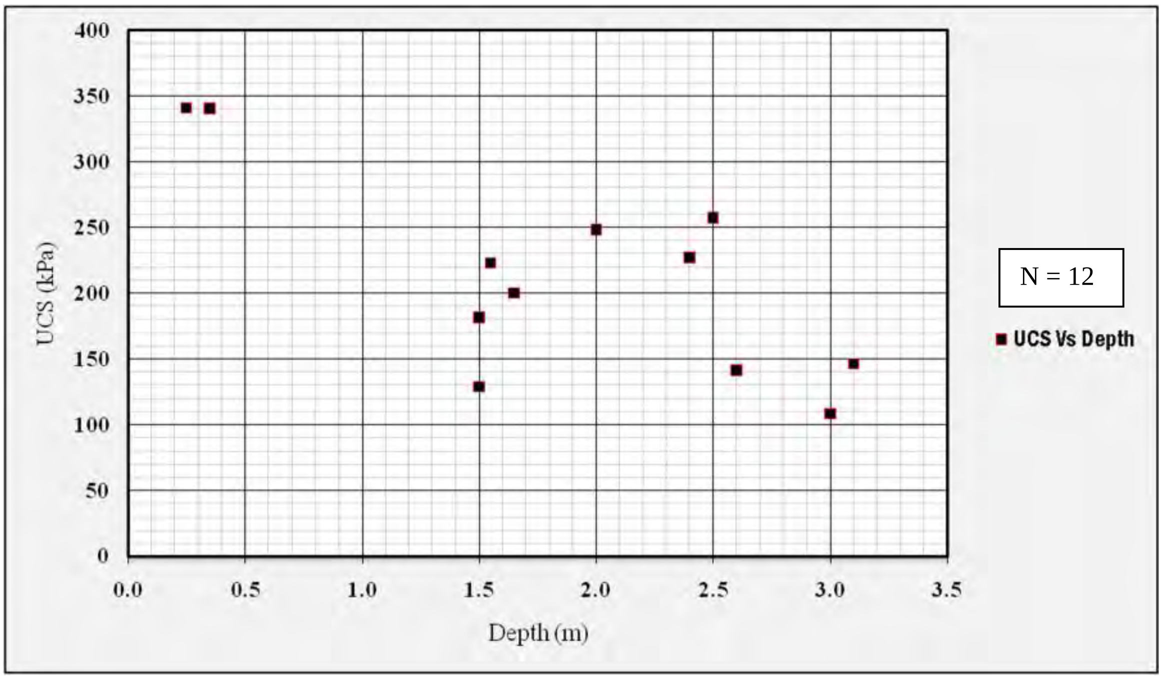


Figure 4-23 Scatter plot of UCS with depth for red clay soils of Addis Ababa

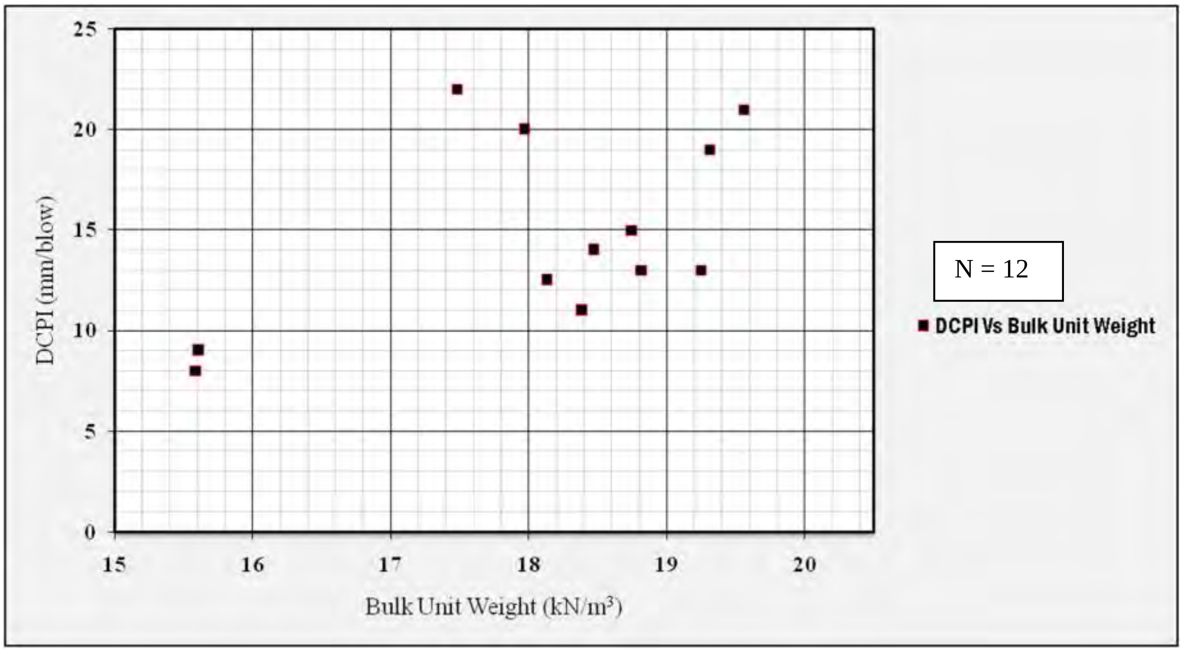


Figure 4-24 Scatter plot of DCPI with bulk unit weight for red clay soils of Addis Ababa

Figure 4-25 Scatter plot of DCPI with dry unit weight for red clay soils of Addis Ababa

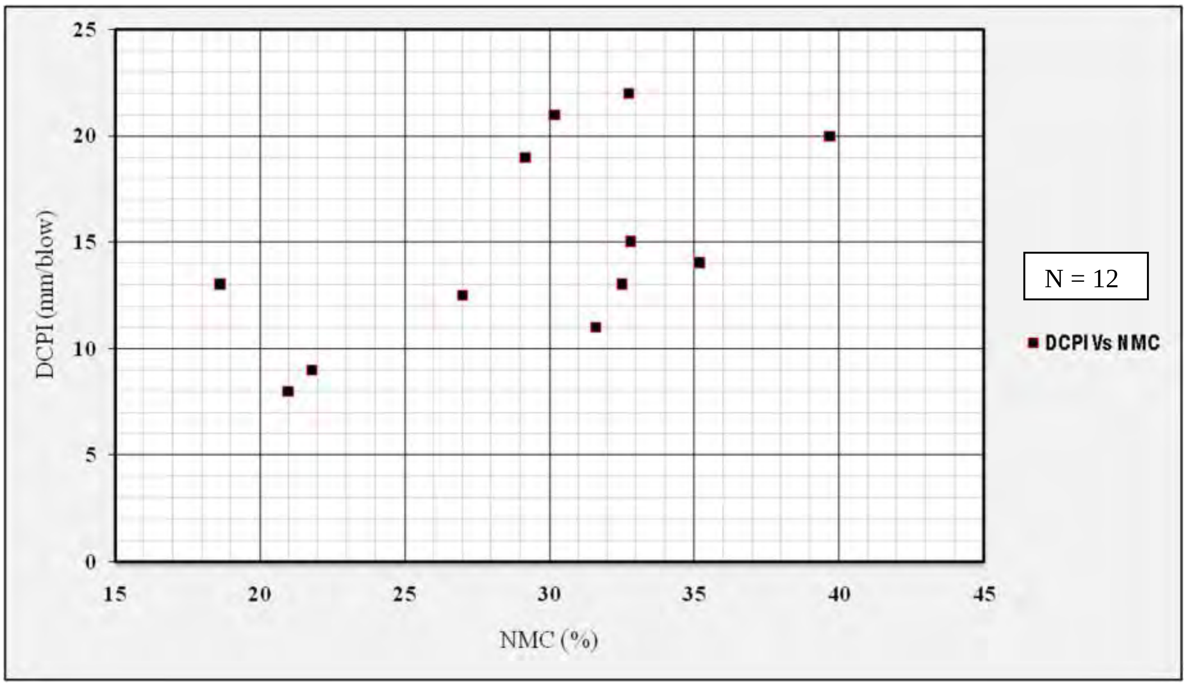


Figure 4-26 Scatter plot of DCPI with natural moisture content for red clay soils of Addis Ababa

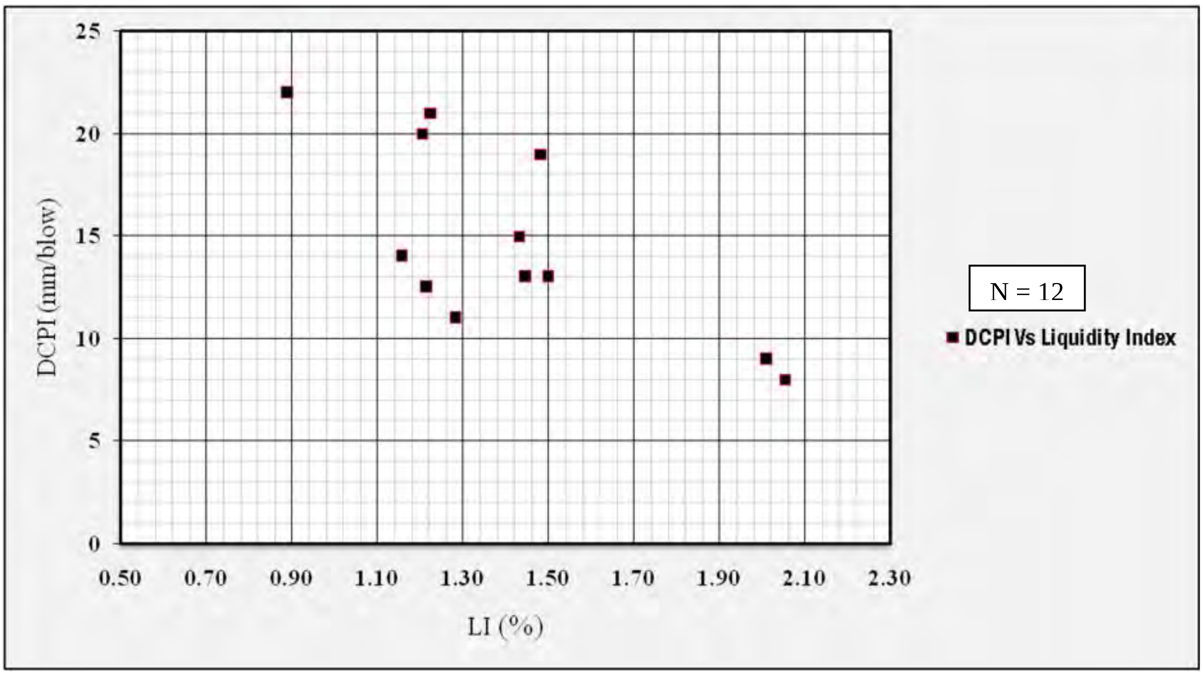


Figure 4-27 Scatter plot of DCPI with liquidity index for red clay soils of Addis Ababa

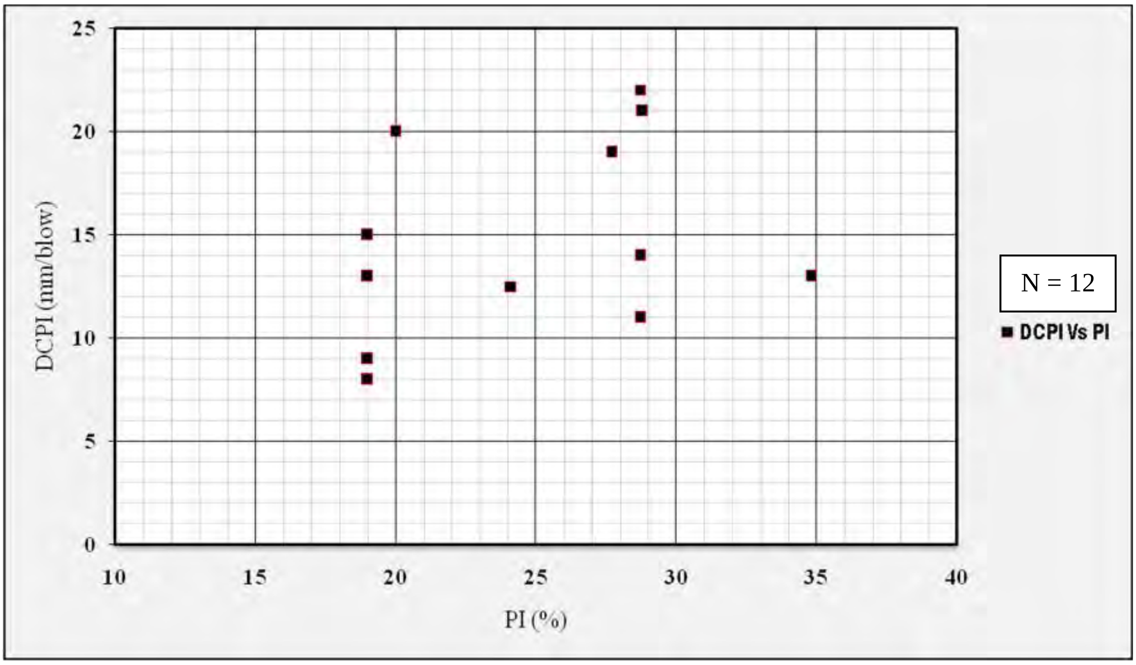


Figure 4-28 Scatter plot of DCPI with plasticity index for red clay soils of Addis Ababa

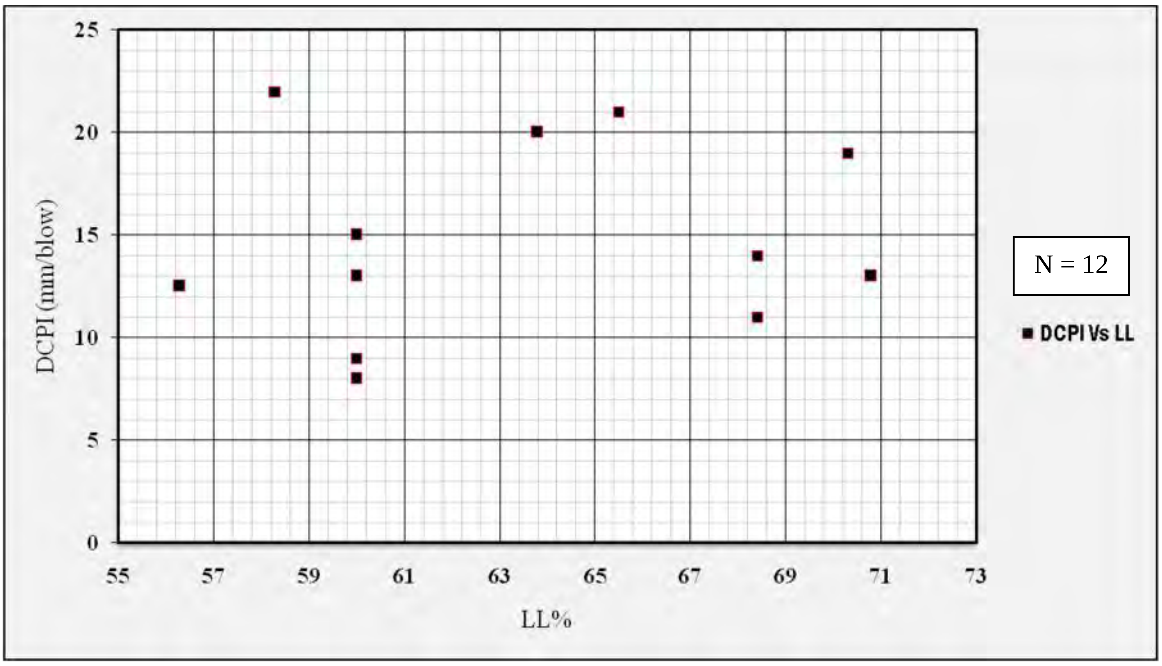


Figure 4-29 Scatter plot of DCPI with liquid limit for red clay soils of Addis Ababa

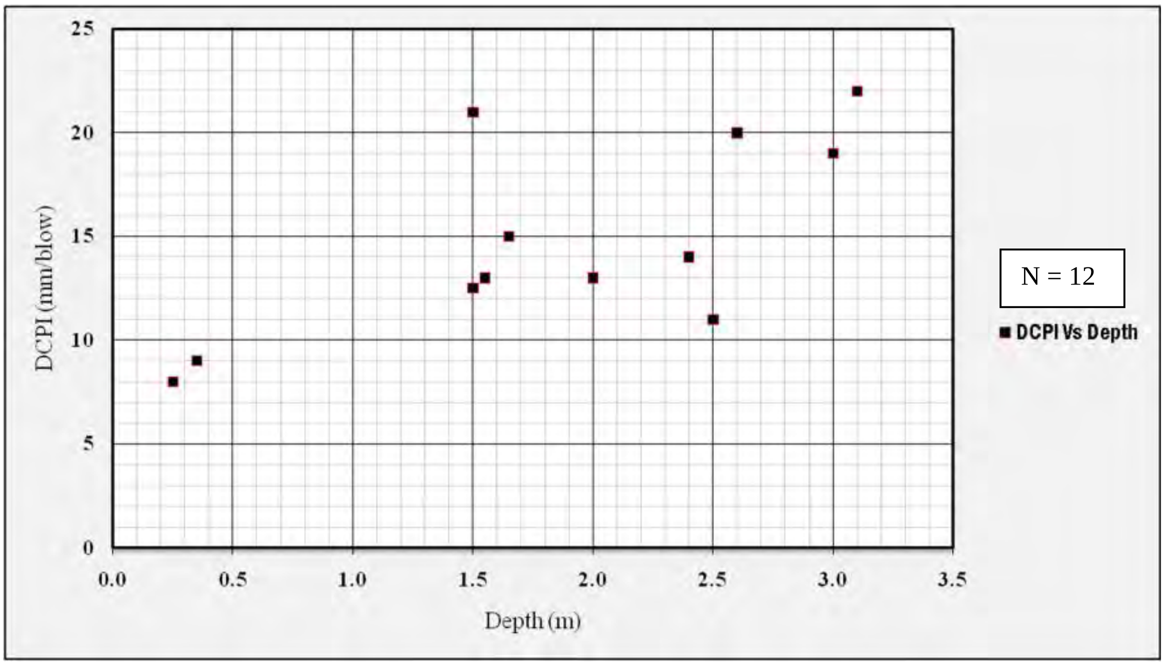


Figure 4-30 Scatter plot of DCPI with Depth for red clay soils of Addis Ababa

4.2.3. Scatter Plot for Category-3 (Clayey Soils after Incorporating Data Outside of Addis Ababa)

The scatter plot of DCPI with γ_{dry}, γ_{bulk}, NMC, LI, LL, PI and Depth to identify parameters that affect DCPI are presented for clayey soils after incorporating data outside of Addis Ababa (Figure 4-31 – 4-37).

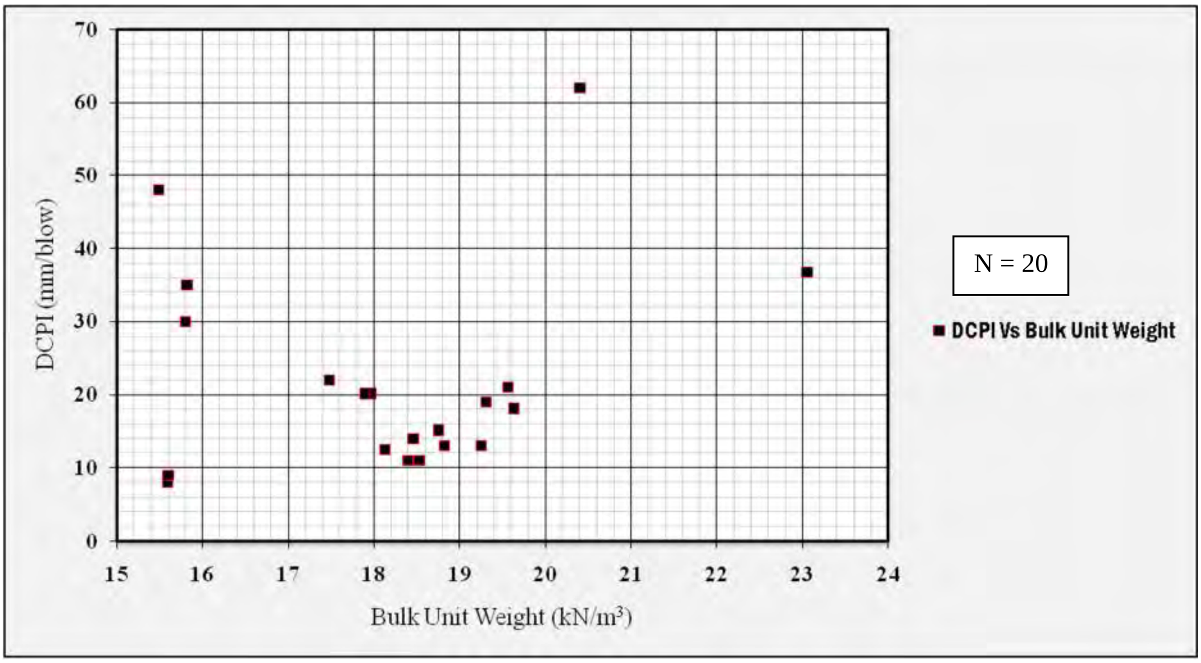


Figure 4-31 Scatter plot of DCPI with bulk unit weight for clayey soils

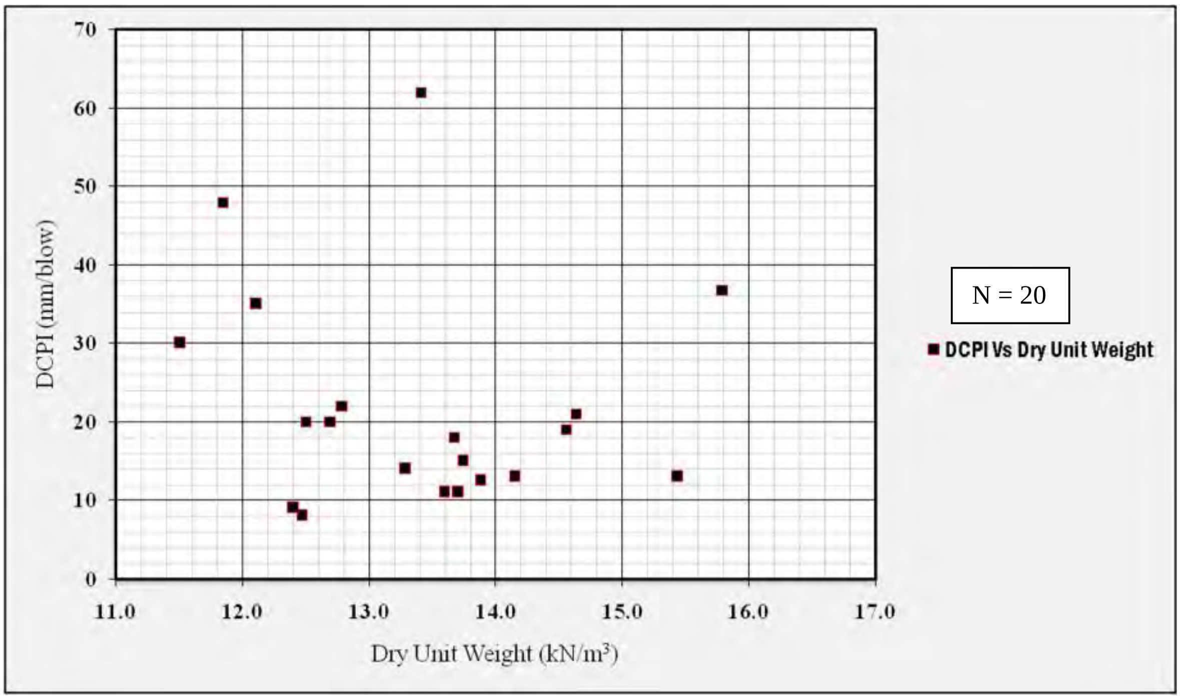


Figure 4-32 Scatter plot of DCPI with dry unit weight for clayey soils

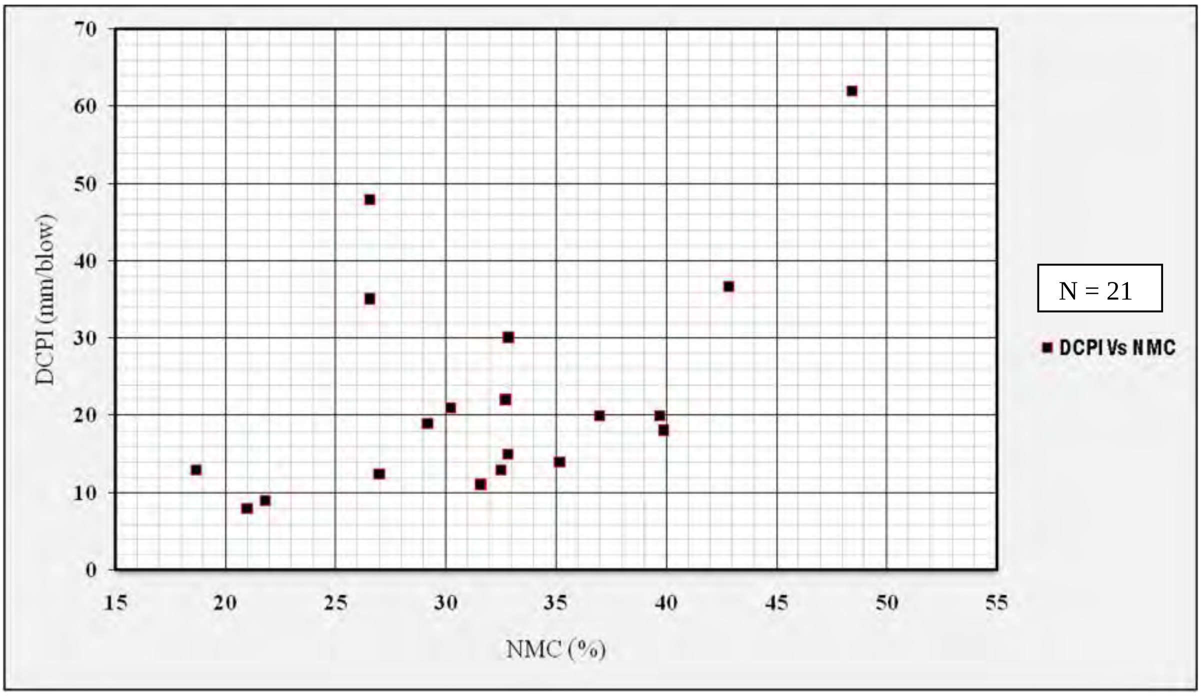


Figure 4-33 Scatter plot of DCPI with natural moisture content for clayey soils

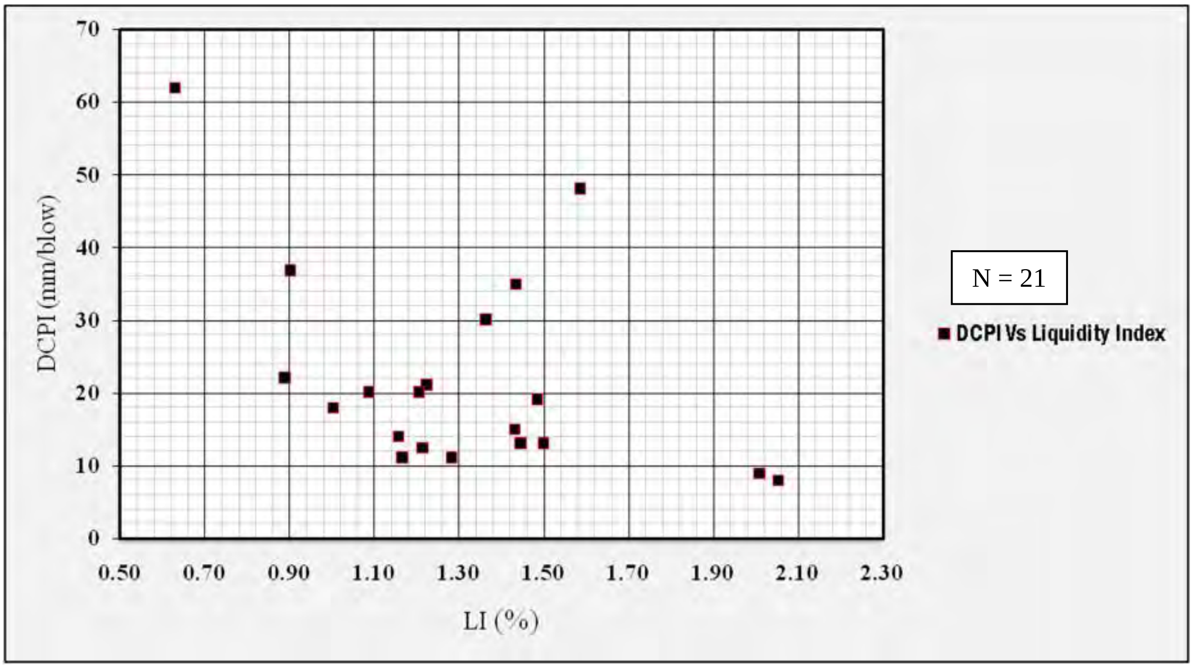


Figure 4-34 Scatter plot of DCPI with liquidity index for clayey soils

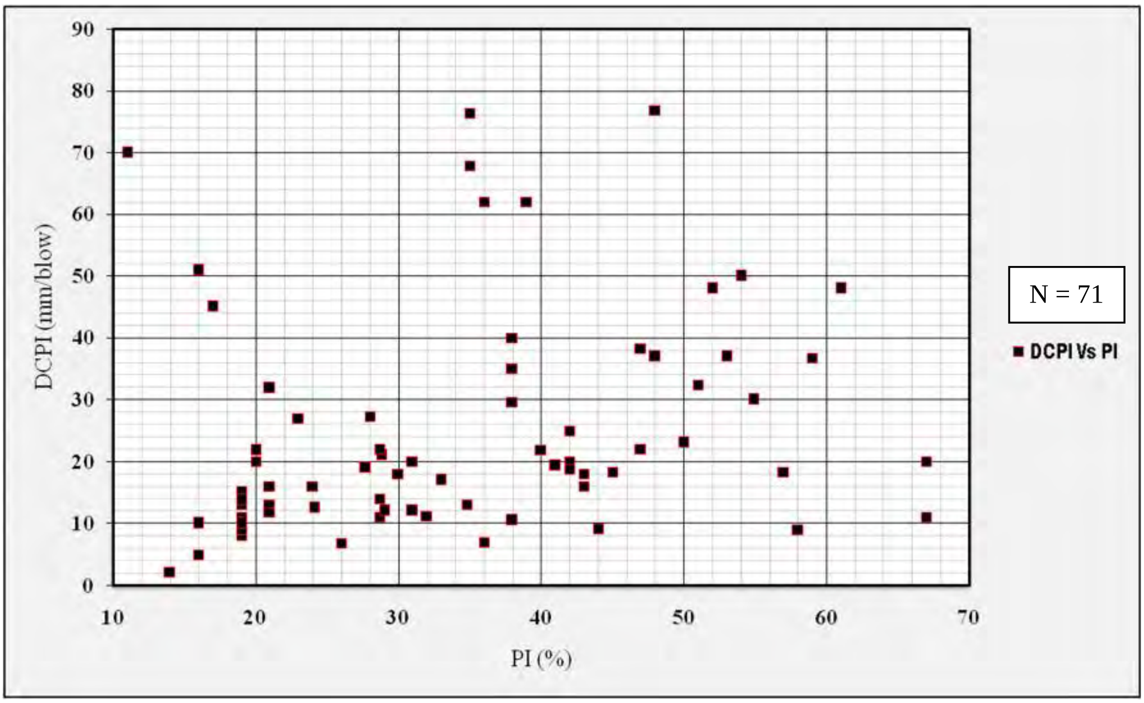


Figure 4-35 Scatter plot of DCPI with plasticity index for clayey soils

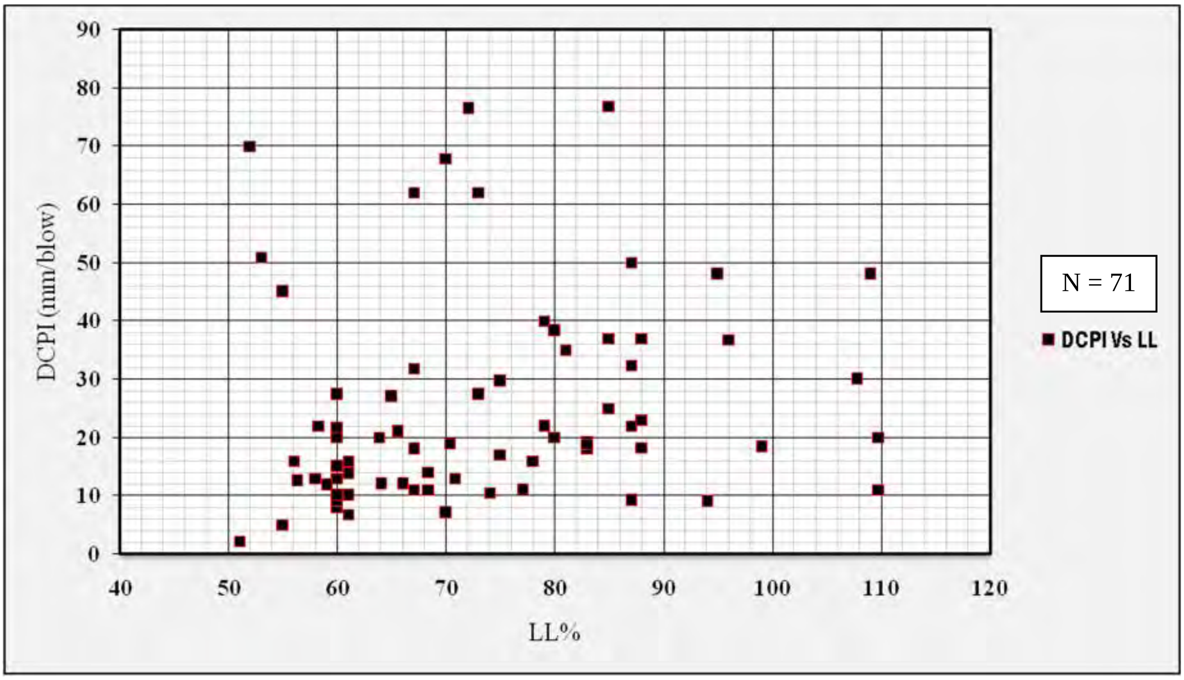


Figure 4-36 Scatter plot of DCPI with liquidlimit for clayey soils

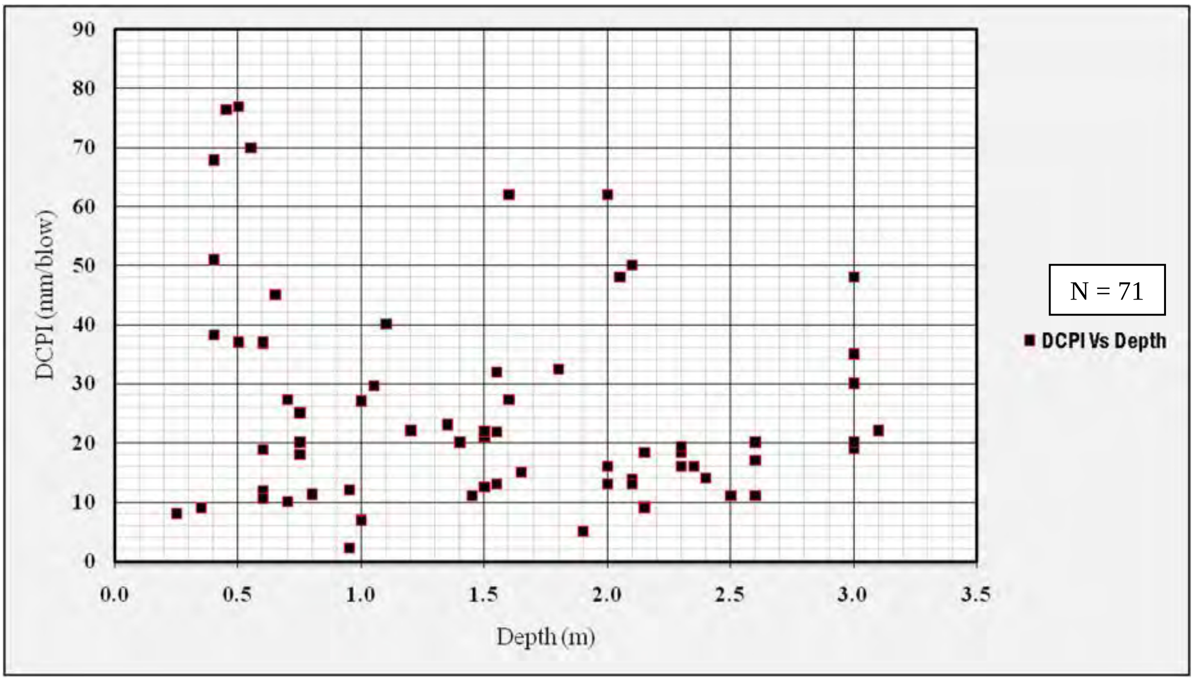


Figure 4-37 Scatter plot of DCPI with depth for clayey soils

4.2.4. Summary of Correlations for Category-1 (Clayey Soils of Addis Ababa)

After carefully studying the data trend on the scatter plot from Figure 4-1 to Figure 4-15 and applying different models, correlations were developed for this category. The summery of the correlations is presented in Tables 4-3.

Table 4-3 Summery of Correlations for Category-1 (Clayey Soils of Addis Ababa)

Equation	R^2	Sample Size
$UCS= 895.8*DCPI^{-0.56}$	0.524	17
$DCPI =-53.3\ln(\gamma_b)+170.5$	0.196	17
$DCPI =-67.3\ln(\gamma_d)+192.6$	0.277	17
$DCPI=1.699*NMC^{0.678}$	0.082	17
$DCPI=33.44e^{-0.49LI}$	0.094	17
$DCPI=11.48\ln(PI)-20.44$	0.234	17
$DCPI =23.2\ln(LL)-80.65$	0.300	17
$DCPI =7.422e^{0.37d}$	0.506	17
$UCS= -170\ln(\gamma_b)+676.8$	0.040	17
$UCS = 33.52*\gamma_d^{0.68}$	0.019	17
$UCS = -6.547*NMC+387.6$	0.253	17
$UCS =156*LI - 23.69$	0.402	17
$UCS =-83.7\ln(PI)+479.3$	0.248	17
$UCS = 3889*LL^{-0.71}$	0.215	17

Equation	R^2	Sample Size
UCS =-61.6*d+326.5	0.585	17

4.2.5. Summary of Correlations for Category-2 (Red Clay Soils of Addis Ababa)

After carefully studying the data trend on the scatter plot Figure 4-16 to Figure 4-30 and applying different models, correlations were developed for this category. The summery of the correlations is presented in Tables 4-4.

Table 4-4 Summery of Correlations for Category-2 (Red Clay Soils of Addis Ababa)

Equation	R^2	Sample Size
UCS=-197ln(DCPI)+735.5	0.711	12
DCPI = $0.006*\gamma_b^{2.673}$	0.402	12
DCPI = $0.569*\gamma_{dry}^{1.229}$	0.071	12
DCPI = $5.366*e^{0.032*NMC}$	0.390	12
DCPI = $40.57*e^{-0.75*LI}$	0.596	12
DCPI = $2.43PI^{0.551}$	0.138	12
DCPI = $0.623*LL^{0.751}$	0.032	12
DCPI = $12.39d^{0.31}$	0.588	12
UCS=-651ln(γ_b)+2077	0.433	12
UCS =352ln(γ_d)+1130	0.105	12
UCS =-199ln(NMC)+881.3	0.349	12
UCS = 173.4*LI - 32.35	0.578	12
UCS =-103ln(PI)+541.4	0.088	12
UCS =-0.671LL-254.5	0.001	12
UCS = -74.6ln(d)+242.9	0.616	12

4.2.6. Summary of Correlations for Category-3 (Clayey Soils after incorporating data outside of Addis Ababa)

After carefully studying the data trend on the scatter plot from Figure 4-31 to Figure 4-37 and applying different models, correlations were developed for this category. The summery of the correlations is presented in Tables 4-5.

Table 4-5 Summery of Correlations for Category-3 (Clayey Soils after incorporating data outside of Addis Ababa)

Equation	R^2	Sample Size
DCPI =$7.057e^{0.055\gamma b}$	0.035	20
DCPI =-29.5ln(γ_d)+98.57	0.032	20

Equation	R^2	Sample Size
$DCPI=5.181e^{0.04*NMC}$	0.290	21
$DCPI=24.28LI^{-1.12}$	0.311	21
$DCPI=3.322PI^{0.518}$	0.102	71
$DCPI =0.252LL^{1.021}$	0.083	71
DCPI =-6.52ln(d)+26.42	0.056	71

4.3. Multiple Regression

4.3.1. General

To examine the combined effect of these parameters on UCS and also on DCPI, a multiple regression analysis is conducted. The basic form of the equation is as follows:

$$y = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + \ldots + \beta_k x_k + \varepsilon \quad \text{.. (4.1)}$$

The single regression discussed previously (i.e., the regression between UCS and DCPI and with other parameters) had shown that the undrained shear strength is significantly affected by some parameters like dynamic cone penetration index, bulk unit weight, natural moisture content, liquidity index and depth. The remaining parameters such as dry unit weight, plasticity index and liquidity index also affect the undrained shear strength but not in a significant amount.

The multiple regressions were conducted for two of three categories since the category under clayey soils after incorporating data outside of Addis Ababa is only used for parametric study on dynamic cone penetration index.

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

4.3.2. Multiple Regression for Category-1(Clayey Soils of Addis Ababa)

Developed equation for multiple regression of UCS(kPa) with DCPI(mm/blow), γ_b(kN/m^3), γ_d(kN/m^3), NMC(%),LI(%), PI(%), LL(%)and d (m), for clayey soils of Addis Ababa, with N=17 and adjusted R^2=0.520 is:

$$UCS = 397.23 - 4.05*DCPI - 46.82*\gamma_b + 39.3*\gamma_d + 3.75*NMC + 83.1*LI + 0.74*PI - 1.06*LL - 7.22*d \quad \text{..(4.2)}$$

Developed equation for multiple regression of DCPI (mm/blow) with PI(%) and LL(%) for clayey soils of Addis Ababa, with N=17 and adjusted R^2=0.331 is:

$$DCPI = -21.449 - 0.795*PI + 0.895*LL \qquad (4.3)$$

4.3.3. Multiple Regression for Category-2(Red Clay Soils of Addis Ababa)

Developed equation for multiple regression of UCS(kPa) with DCPI(mm/blow), γ_b(kN/m^3), γ_d(kN/m^3), NMC(%), LI(%), PI(%), LL(%)and d(m), for red clay soils, with N=12 and adjusted R^2=0.860 is:

$$UCS = -5.84 - 10.87*DCPI - 717*\gamma_b + 968.4*\gamma_d + 171.6*NMC + 1701.1*LI + 91.47*PI - 63.5*LL - 16.62*d \qquad (4.4)$$

Developed equation for multiple regression of DCPI (mm/blow) with PI(%) and LL(%) for red clay soils, with N=12 and adjusted R^2=0.860 is:

$$DCPI = 14.089 + 0.345*PI - 0.124*LL \qquad (4.5)$$

4.3.4. Multiple Regression for Category-3 (Clayey Soils after incorporating data outside of Addis Ababa)

Developed equation for multiple regression of DCPI (mm/blow) with PI(%) and LL(%) for clayey soils after incorporating data outside of Addis Ababa, with N=71 and adjusted R^2=0.036 is:

$$DCPI = 33.014 + 0.732*PI - 0.452*LL \qquad (4.6)$$

5. DISCUSSION

5.1. CATEGORY-1 (CLAYEY SOILS OF ADDIS ABABA)

5.1.1. Single Regression

After carefully studying the data trend on the scatter plot and applying different models, this category revealed that unconfined compression strength is influenced by dynamic cone penetration index, liquidity index and depth by achieving coefficient of determination of 52.4%, 40.2% and 58.5%, respectively.

The dynamic cone penetration index is also influenced by depth in addition to undrained shear strength by achieving coefficient of determination of 50.6%, for depth and 52.4%, for UCS.

This category also revealed that correlation of UCS with bulk unit weight, dry unit weight, natural moisture content, plasticity index and liquid limit in this group gave fair to poor result. While correlation of DCPI with bulk unit weight, dry unit weight, natural moisture content, liquidity index, plasticity index and liquid limit gave a poor result. The summery of the correlations is presented in Tables 4-3.

5.1.2. Multiple Regression

The multiple regression of UCS with DCPI, γ_{bulk}, γ_{dry}, NMC, LI, PI, LL and depth indicated that UCS has fair correlation with the parameters by achieving adjusted coefficient of determination of 52% with 17 samples (refer to Equation 4.2). The result has shown that the multiple regression has not improved the coefficient of determination for the prediction of UCS by DCPI and depth.

The multiple regression of DCPI with PI and LL indicated that DCPI has poor correlation with the Atterberg limits by achieving adjusted coefficient of determination of 33.1% with 17 samples(refer to Equation 4.3).

5.2. CATEGORY-2 (RED CLAY SOILS OF ADDIS ABABA)

5.2.1. Single Regression

This category revealed that unconfined compression strength is significantly influenced by dynamic cone penetration index, liquidity index and depth by achieving coefficient of determination of 71.1%, 61.6% and 58.5%, respectively.

The dynamic cone penetration index is also significantly influenced by liquidity index and depth in addition to unconfined compression strength by achieving coefficient of determination of 58.8% for liquidity index; 59.6% for depth; and 71.1% for unconfined compression strength.

This category also revealed that correlation of UCS with bulk unit weight and natural moisture content in this group gave a fair result and correlation of UCS with dry unit weight, plasticity index and liquid limit gave a poor result. While correlation of DCPI with natural moisture content and bulk unit weight gave a fair result and correlation of DCPI with dry unit weight and plasticity index gave a poor result. The summery of the correlations is presented in Tables 4-4.

5.2.2. Multiple Regression

The multiple regression of UCS with DCPI, γ_{bulk}, γ_{dry}, NMC, LI, depth, PI and LL indicated that UCS has good correlation with the parameters by achieving adjusted coefficient of determination of 86.0% with 12 samples (refer to Equation 4.4).

The multiple regression of DCPI with PI and LL indicated that DCPI has poor correlation with the Atterberg limits by achieving adjusted coefficient of determination of 0% with 12 samples(refer to Equation 4.5).

5.3. CATEGORY-3 (CLAYEY SOILS AFTER INCORPORATING DATA OUTSIDE OF ADDIS ABABA)

5.3.1. Single Regression

This category revealed that natural moisture content and liquidity index have fair to poor influence on dynamic cone penetration index by achieving coefficient of determination of 29.0% and 31.1%, respectively.

This category also revealed that parameters like bulk unit weight, dry unit weight, plasticity index, liquid limit and depth have poor influence on the dynamic cone penetration index. The summery of the correlations is presented in Tables 4-5.

5.3.2. Multiple Regression

The multiple regression of DCPI with PI and LL indicated that DCPI has poor correlation with the Atterberg limits by achieving adjusted coefficient of determination of 3.6% with 71 samples(refer to Equation 4.6).

5.4. DEVELOPMENT OF EQUATION BASED ON BEARING CAPACITY THEORY

Some good correlations between undrained shear strength and the DCP penetration index for *category-1* and *category-2* are obtained (refer to Table 4-3 and 4-4, 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), as discussed in the literature review section 2.4, 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.

From Table 4-3 and Table 4-4 one can observe a correlation between UCS(kPa) and DCPI(mm/blow) gave:

$UCS = 895.8*DCPI^{-0.56}$, R^2=52.4% and N=17, for clayey soils of Addis Ababa………..(5.1)

$UCS = -197*\ln(DCPI)+735.5$, R^2=71.1% and N=12, for red clay soils of Addis Ababa…(5.2)

Converting equations 5.1 and 5.2 into cohesion, the corresponding relation will be:

$c = 447.9*DCPI^{-0.56}$, for clayey soils of Addis Ababa……………..(5.3)

$c = -98.5*\ln(DCPI)+367.75$, for red clay soils of Addis Ababa……(5.4)

After inserting equations 5.3 and 5.4 into equation 2.21, the corresponding relations of bearing capacity equations for initial loading condition will be:

$q_{ult} = 2303.1*DCPI^{-0.56}+\gamma h$, for clayey soils of Addis Ababa………..(5.5)

$q_{ult} = -506.5*\ln(DCPI)+\gamma h+1891$, for red clay soils of Addis Ababa…(5.6)

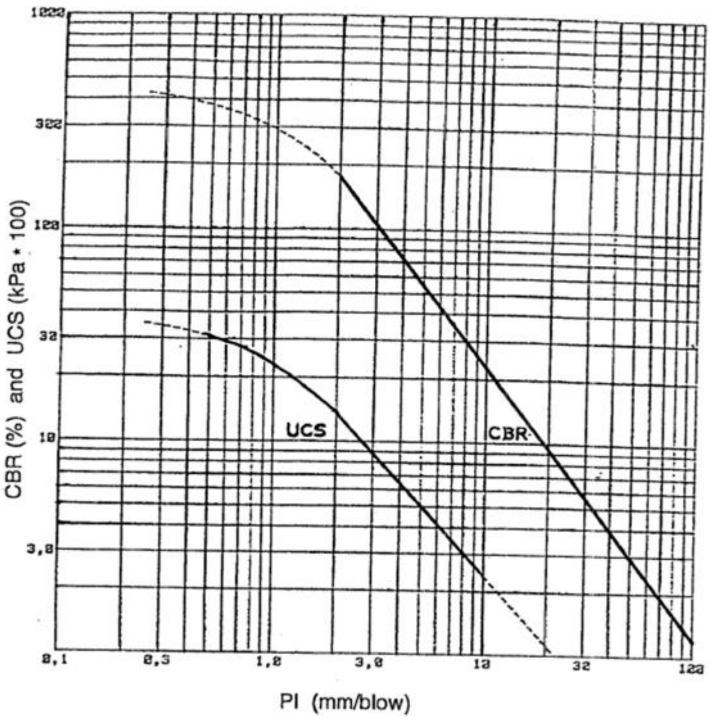
The objective of this thesis is to develop a simple method to predict the shear strength of clay soils without getting into tiresome and costly laboratory tests. It has been shown that introduction of parameters like $\gamma_{bulk,}$ Liquidity index, PI, LL, NMC and depth in to equation between UCS and 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 4.4 for red clay soils.

5.5. Comparison of Findings

Table 5-1 Comparison of findings of past studies with the current study on parametric study

✓ ***Past Studies***	✓ ***Current Study***
✓ *Klein and Savage as cited by [7]:* *Moisture content, gradation, density and plasticity were important material properties influencing the DCPI*	✓ *Unconfined compressive strength, liquidity index and depth are the important parameters influencing the DCPI* ✓ *Bulk unit weight and natural moisture content are the next important parameters* ✓ *Dry unit weight has little effect while plasticity index and liquid limit have no influence*
✓ *Hassan [8]:* *Concluded that for fine-grained soils, moisture content, soil classification and dry density affect the DCPI*	✓ *Natural moisture content has influence on the DCPI while dry unit weight has little influence* ✓ *Atterberg limits no influence (with 71 samples)*
✓ *Livneh, et al. [9]:* *There is no vertical confinement effect by upper cohesive layers on the DCP values of lower cohesive sub grade layers* ✓ *Hassan [8]:* *Stated that frictional effect is usually small in cohesive soils*	✓ *Depth has important effect on the determination of DCPI. These could be in the form of vertical confinement and/or side friction*

Table 5-2 Comparison of findings of past studies with the current study on the prediction of undrained shear strength

✓ ***Study***	✓ ***Finding***
✓ ***Current Study***	✓ *For red clay soils: UCS=-197ln(DCPI)+735.5 71.1% coefficient of determination* ✓ *For example: If DCPI is equal to 10mm/blow, UCS will be 281.90kPa.*
	✓ *For clayey soils of Addis Ababa:* $UCS=895.8*DCPI^{-0.56}$ *with 52.4% coefficient of determination* ✓ *For example: If DCPI is equal to 10mm/blow, UCS will be 246.72kPa.*
✓ *Relationship between Penetration Index (PI) and Unconfined Compressive Strength (UCS) [**13**]*	 ✓ *For example: If DCPI is equal to 10mm/blow, UCS will be around 220.0kPa.*
✓ ***McElvaney and Djatnika (1991) as cited by [7]*** *indicated that DCPI values can be correlated to the unconfined*	✓ *Log₁₀(UCS) = 3.56 - 0.807Log₁₀(DCPI)* → $Log_{10}(UCS) = 3.56 - 0.807Log_{10}(DCPI)$ *50% probability of underestimation* ✓ *For example: If DCPI is equal to 10mm/blow, UCS will be around 566.24kPa.*

✓ ***Study***	✓ ***Finding***
compressive strength (UCS) of soil-lime mixtures. They considered both individual and combined soil types in their analysis. They have concluded that the inclusion of data on mixtures from material with zero lime content has negligible effects on the correlation equations, indicating that the correlation is mainly a function of strength and not of the way in which strength is achieved. ✓ *This observation was valid only for lower range of strain values. For the combined data, three relationships, with each model permitting estimated unconfined compressive strength to a predetermined reliability level, were developed. These relationships are summarized below [7]:*	✓ $Log_{10}(UCS) = 3.29 - 0.809Log_{10}(DCPI)$ *95% confident that probability of underestimation will not exceed 15 percent* ✓ *For example: If DCPI is equal to 10mm/blow, UCS will be around 302.70kPa.* ✓ $Log_{10}(UCS) = 3.21 - 0.809Log_{10}(DCPI)$ *99% confident that probability of underestimation will not exceed 15 percent* ✓ *For example: If DCPI is equal to 10mm/blow, UCS will be around 251.77kPa.*

6. CONCLUSION

The major findings and observations are stated in this section. The objective of introducing DCP as a simple test device that 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 were also carried out on parameters affecting DCPI. The study is conducted by categorizing the locations of sample collection in to three categories and the following conclusions are drowned:

- ***Catigory-1 (Clayey Soils inside Addis Ababa)*** revealed that DCPI is influenced by UCS and depth. Parameters like γ_{bulk}, γ_{dry}, moisture content, liquidity index, PI and LL have little or no influence. UCS, for this category, can be estimated from DCPI by UCS = 895.8*DCPI$^{-0.56}$ with R^2 of 52.4%. The corresponding bearing capacity equation of q_{ult} = 2303.1*DCPI$^{-0.56}$+γh.

- ***Catigory-2 (Red Clay Soils inside Addis Ababa)*** revealed that DCPI is influenced by UCS, liquidity index and depth. Parameters like natural moisture content and γ_{bulk} have little influence while γ_{dry}, PI and LL have no influence. UCS, for this category, can be estimated from DCPI by UCS=-197ln(DCPI)+735.5 with R^2 of 71.1%. The corresponding bearing capacity equation of q_{ult}=-506.5ln(DCPI)+γh+1891.

- ***Catigory-3 (Clayey soils after incorporating data outside of Addis Ababa)*** revealed that natural moisture content and LI have little influence on DCPI. While parameters like γ_{bulk}, γ_{dry}, PI, LL and depth have no influence on the DCPI.

- The effect of PI and LL on DCPI has been studied using 12 to 71 samples in the different categories but indicated poor correlation with the Atterberg limits.

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APPENDIXES

APPENDIX – A: Field Tests Result

A – 1) DCPI Result for Sites Inside Addis Ababa

Table A - 1.1 Penetration Data Report for Imperial Hotel [I1]

Penetration Data Report for Imperial Site_01 [2.0m - 2.87m]

Site Name:	**Imperial [I1]**	Test Date: (22_23)-01-2011[DD/MM/YY]
Test Depth (m):	2.00	
Cone Angle (deg.):	60	
Soil Data:	Wet, black, high plastic, CLAY [black cotton /expansive/ soil]	

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
-	-	-	**2000**	-
1	2	2	+205	102
2	2	4	+325	60
3	2	6	+405	40
4	2	8	+485	40
5	2	10	+545	30
6	2	12	+615	35
7	2	14	+665	25
8	2	16	+745	40
9	2	18	+795	25
10	2	20	+825	15
11	**2**	**22**	**+865**	**20**

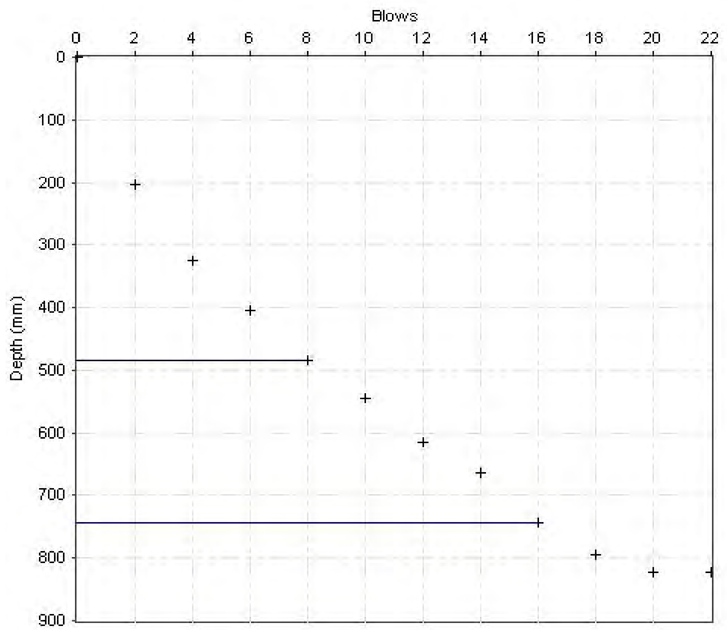

Figure A-1.1 The Dynamic Cone penetration Used for Imperial Hotel [I1] [2.0m - 2.87m]

Table A - 1.2 Penetration Data Report for Imperial Hotel [I2]

Penetration Data Report for Imperial Site_02 [2.0m - 3.005m]

Site Name:	**Imperial [I2]**	Test Date: (22_23)-01-2011[DD/MM/YY]
Test Depth (m):	2.00	
Cone Angle (deg.):	60	
Soil Data:	Wet, black, high plastic, CLAY [black cotton /expansive/ soil]	

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
-	-	-	**2000**	-

Penetration Data Report for Imperial Site_02 [2.0m - 3.005m]

Site Name: **Imperial [I_2]** Test Date: (22_23)-01-2011[DD/MM/YY]

Test Depth (m): 2.00

Cone Angle (deg.): 60

Soil Data: Wet, black, high plastic, CLAY [black cotton /expansive/ soil]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
1	2	2	+165	82
2	2	4	+255	45
3	2	6	+385	65
4	2	8	+475	45
5	2	10	+555	40
6	2	12	+615	30
7	2	14	+695	40
8	2	16	+795	50
9	2	18	+875	40
10	2	20	+945	35
11	**2**	**22**	**+1005**	**30**

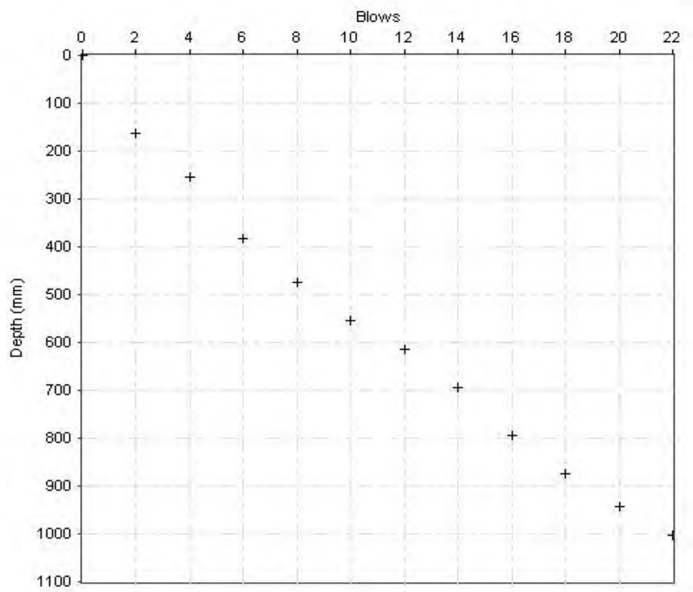

Figure A-1.2The Dynamic Cone penetration Used for Imperial Hotel [I_2] [2.0m - 3.0m]

Table A - 1.3 Penetration Data Report for Lebu [L_1] [2.20m - 3.14m]

Penetration Data Report for Lebu Site_01 [2.20m - 3.135m]

Site Name: **Lebu [L_1]** Test Date: (01_02)-02-2011[DD/MM/YY]

Test Depth (m): 2.20

Cone Angle (deg.): 60

Soil Data: Wet, black, high plastic, CLAY [black cotton /expansive/ soil]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)

Penetration Data Report for Lebu Site_01 [2.20m - 3.135m]

Site Name:	**Lebu [L1]**	Test Date:	(01_02)-02-2011[DD/MM/YY]
Test Depth (m):	2.20		
Cone Angle (deg.):	60		
Soil Data:	Wet, black, high plastic, CLAY [black cotton /expansive/ soil]		

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
-	-	-	**2200**	-
1	3	3	+200	66.33
2	2	5	+275	37.5
3	2	7	+345	35
4	2	9	+395	25
5	2	11	+455	30
6	2	13	+525	35
7	2	15	+585	30
8	2	17	+655	35
9	2	19	+735	40
10	2	21	+795	30
11	**2**	**23**	**+865**	**35**
12	2	25	+935	35
13	2	27	+1000	32.5

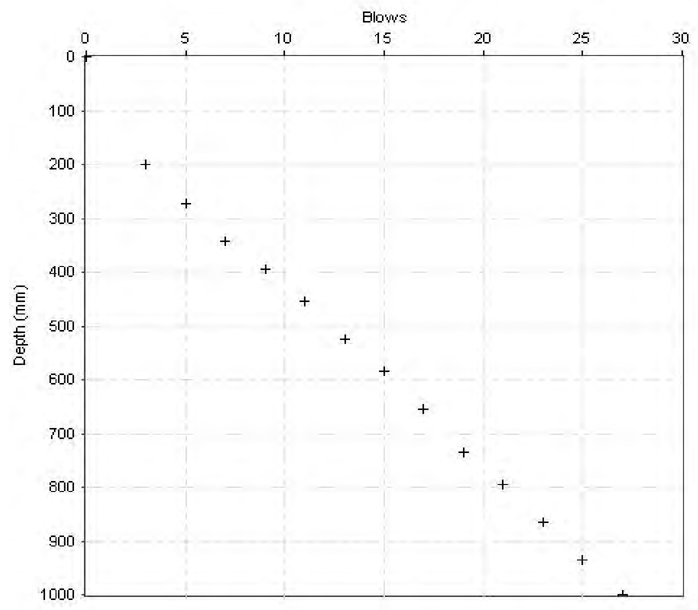

Figure A-1.3Penetration Data Report for Lebu [L1] [2.20m - 3.14m]

Table A-1.4 Penetration Data Report for Lebu [L2]

Penetration Data Report for Lebu Site _02 [2.25m - 3.36m]

Site Name:	**Lebu [L2]**	Test Date: (01_02)-02-2011[DD/MM/YY]
Test Depth (m):	2.25	
Cone Angle (deg.):	60	
Soil Data:	Wet, black, high plastic, CLAY [black cotton /expansive/ soil]	

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
-	-	-	**2250**	-
1	3	3	+210	69.67

Penetration Data Report for Lebu Site _02 [2.25m - 3.36m]

Site Name: **Lebu [L_2]** Test Date: (01_02)-02-2011[DD/MM/YY]

Test Depth (m): 2.25

Cone Angle (deg.): 60

Soil Data: Wet, black, high plastic, CLAY [black cotton /expansive/ soil]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
2	2	5	+355	72.5
3	2	7	+475	60
4	2	9	+575	50
5	2	11	+685	55
6	**2**	**13**	**+775**	**45**
7	**2**	**15**	**+875**	**50**
8	2	17	+975	50
9	2	19	+1115	70

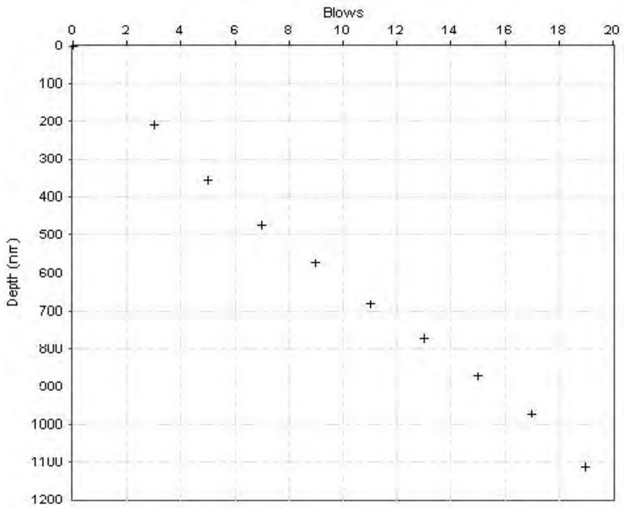

Figure A-1.4Penetration Data Report for Lebu [L_2] [2.25m - 3.36m]

Table A - 1.5 Penetration Data Report for Addisu Gebeya [AG]

Penetration Data Report for Addisu Gebeya [1.60m - 2.35m]

Site Name: **Addisu Gebeya** (09°03' 55.56", 38°43' 44.22", 2612)

Test Depth (m): 1.60 Test Date: 07/05/2011 [DD/MM/YY]

Cone Angle (deg.): 60

Soil Data: Moist, high plastic, silty CLAY [Red clay soil]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
-	-	-	**1600**	-
1	2	2	+55	27
2	2	4	+80	12.5
3	2	6	+110	15
4	2	8	+140	15
5	2	10	+180	20
6	2	12	+215	17.5
7	2	14	+260	22.5
8	2	16	+280	10
9	2	18	+305	12.5
10	2	20	+330	12.5
11	2	22	+360	15
12	**2**	**24**	**+385**	**12.5**
13	**2**	**26**	**+415**	**15**
14	**2**	**28**	**+440**	**12.5**
15	**2**	**30**	**+465**	**12.5**
16	2	32	+490	12.5
17	2	34	+520	15
18	2	36	+550	15
19	2	38	+580	15
20	2	40	+600	10
21	2	42	+625	12.5
22	2	44	+645	10
23	2	46	+670	12.5

Penetration Data Report for Addisu Gebeya [1.60m - 2.35m]

Site Name: **Addisu Gebeya** (09°03' 55.56", 38°43' 44.22", 2612)

Test Depth (m): 1.60 Test Date: 07/05/2011 [DD/MM/YY]

Cone Angle (deg.): 60

Soil Data: Moist, high plastic, silty CLAY [Red clay soil]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
24	2	48	+700	15
25	2	50	+780	40

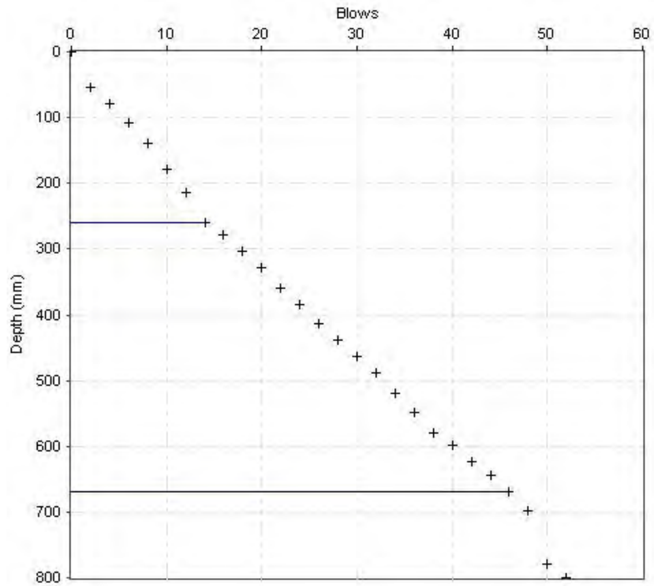

Figure A-1.5Penetration Data Report for Addisu Gebeya [AG] [1.6m – 2.35m]

Table A - 1.6 (a) Penetration Data Report for Atari [AT$_1$]

Penetration Data Report for Atari [AT$_1$] [1.00m – 2.06m]

Site Name: **Atari** (09^{0}04' 16.10", 38^{0}44' 05.1")

Test Depth (m): 1.00 Test Date: 18/05/2011 [DD/MM/YY]

Cone Angle (deg.): 60

Soil Data: Dry, high plastic, silty CLAY [Red clay soil]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
-	-	-	**1000**	-
1	2	2	+55	27
2	2	4	+110	27.5
3	2	6	+160	25
4	2	8	+215	27.5
5	2	10	+260	22.5
6	2	12	+315	27.5
7	2	14	+365	25
8	2	16	+415	25
9	2	18	+465	25
10	**2**	**20**	**+515**	**25**
11	**2**	**22**	**+555**	**20**
12	**2**	**24**	**+595**	**20**
13	**2**	**26**	**+640**	**22.5**
14	2	28	+680	20
15	2	30	+730	25
16	2	32	+780	25
17	2	34	+830	25
18	2	36	+875	22.5
19	2	38	+915	20

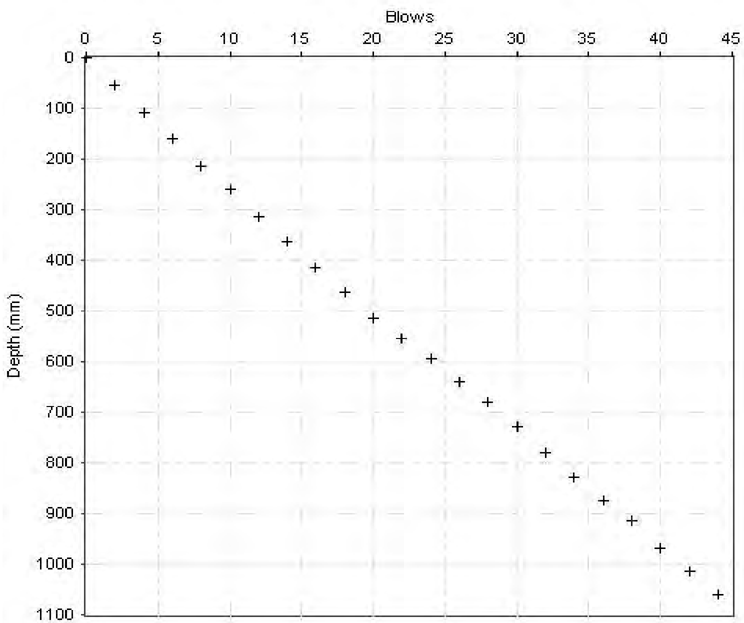

Figure A-1.6 (a) Penetration Data Report for Atari [AT1] [1.0m – 2.06m]

Table A - 1.6 (b) Penetration Data Report for Atari [AT2]

Penetration Data Report for Atari [AT2] [2.00m – 3.06 m]

Site Name:	**Atari** ($09^{0}04'$ 16.10", 38044' 05.1")	Test Date:	18/05/2011 [DD/MM/YY]
Test Depth (m):	2.00		
Cone Angle (deg.):	60		
Soil Data:	Moist, high plastic, silty CLAY [Red clay soil]		

Penetration Data Report for Atari [AT2] [2.00m – 3.06 m]

Site Name: **Atari** ($09^{0}04'$ 16.10", 38044' 05.1") Test Date: 18/05/2011 [DD/MM/YY]
Test Depth (m): 2.00
Cone Angle (deg.): 60
Soil Data: Moist, high plastic, silty CLAY [Red clay soil]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
-	-	-	**2000**	-
2	2	2	+50	24.5
3	2	4	+90	20
4	2	6	+130	20
5	2	8	+170	20
6	2	10	+210	20
7	2	12	+258	24
8	2	14	+300	21
9	2	16	+345	22.5
10	2	18	+395	25
11	2	20	+450	27.5
12	2	22	+505	27.5
13	2	24	+550	22.5
14	2	26	+590	20
15	2	28	+630	20
16	2	30	+670	20
17	2	32	+705	17.5
18	2	34	+750	22.5
19	2	36	+790	20
20	2	38	+830	20
21	2	40	+900	35
22	2	42	+945	22.5
23	2	44	+985	20
24	**2**	**46**	**+1025**	**20**
25	**2**	**48**	**+1060**	**17.5**

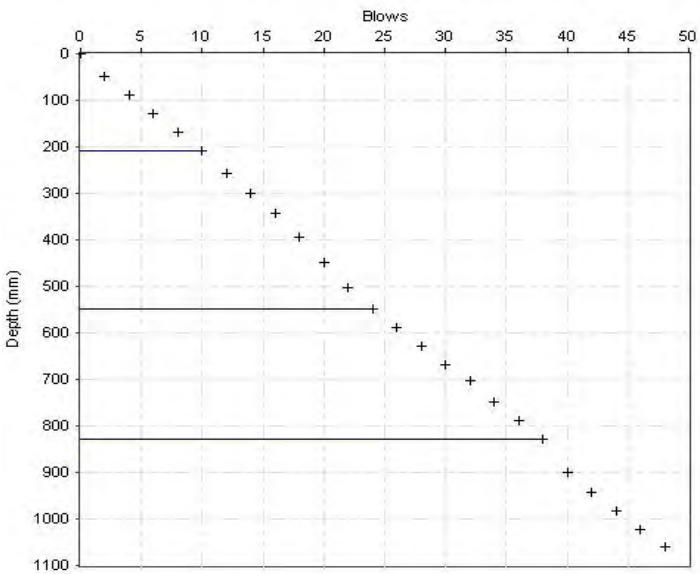

Figure A-1.6 (b) Penetration Data Report for Atari [AT_2] [2.0m – 3.06m]

Table A - 1.7 (a) Penetration Data Report for Aweliya [AW_1]

Penetration Data Report for Aweliya [AW_1] [0.90m - 1.91m]

Site Name:	**Aweliya** (09^{0}04'16.1", 38^{0}44' 05.1")	Test Date:	25/05/2011 [DD/MM/YY]
Test Depth (m):	0.90		
Cone Angle (deg.):	60		
Soil Data:	Moist, high plastic, silty CLAY [Red clay soil]		

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
-	-	-	**900**	-
1	2	2	+70	34.5
2	2	4	+120	25

Penetration Data Report for Aweliya [AW_1] [0.90m - 1.91m]

Site Name:	**Aweliya** (09^{0}04'16.1", 38^{0}44' 05.1")	Test Date:	25/05/2011 [DD/MM/YY]
Test Depth (m):	0.90		
Cone Angle (deg.):	60		
Soil Data:	Moist, high plastic, silty CLAY [Red clay soil]		

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
3	2	6	+170	25
4	2	8	+210	20
5	2	10	+245	17.5
6	2	12	+280	17.5
7	2	14	+325	22.5
8	2	16	+360	17.5
9	2	18	+393	16.5
10	2	20	+420	13.5
11	2	22	+445	12.5
12	2	24	+475	15
13	2	26	+500	12.5
14	2	28	+524	12
15	2	30	+545	10.5
16	2	32	+574	14.5
17	**2**	**34**	**+600**	**13**
18	2	36	+625	12.5
19	2	38	+650	12.5
20	2	40	+675	12.5
21	2	42	+700	12.5
22	2	44	+725	12.5
23	2	46	+755	15
24	2	48	+775	10
25	2	50	+800	12.5
26	2	52	+822	11
27	2	54	+845	11.5
28	2	56	+872	13.5

Penetration Data Report for Aweliya [AW1] [0.90m - 1.91m]

Site Name:	**Aweliya** (09^{0}04'16.1", 38^{0}44' 05.1")	Test Date:	25/05/2011 [DD/MM/YY]
Test Depth (m):	0.90		
Cone Angle (deg.):	60		
Soil Data:	Moist, high plastic, silty CLAY [Red clay soil]		

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
29	2	58	+900	14
30	2	60	+925	12.5
31	2	62	+955	15
32	2	64	+982	13.5
33	2	66	+1010	14

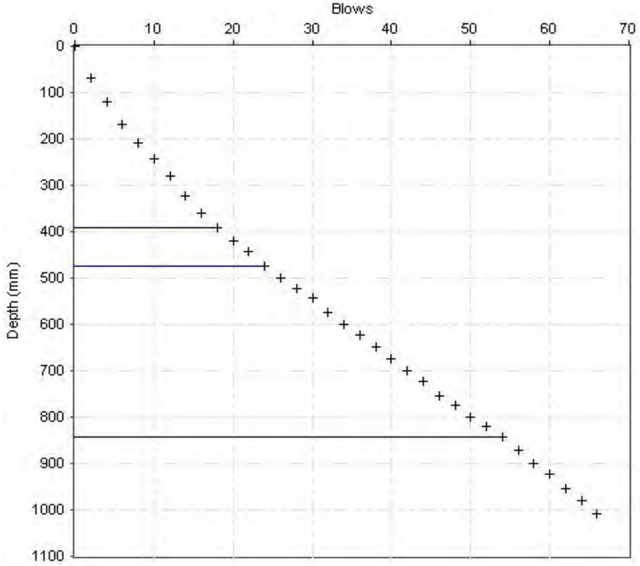

Figure A - 1.7 (a) Penetration Data Report for Aweliya [AW1] [0.9m – 1.91m]

Table A - 1.7 (b) Penetration Data Report for Aweliya [AW2-4]

Penetration Data Report for Aweliya [AW2-4] [1.80m - 2.87m]

Site Name:	**Aweliya** (09^{0}04'16.1", 38^{0}44' 05.1")	Test Date:	25/05/2011 [DD/MM/YY]
Test Depth (m):	1.80		
Cone Angle (deg.):	60		
Soil Data:	Moist, high plastic, silty CLAY [Red clay soil]		

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
-	-	-	**1800**	-
1	2	2	+25	12
2	2	4	+70	22.5
3	2	6	+100	15
4	2	8	+130	15
5	2	10	+160	15
6	2	12	+180	10
7	2	14	+200	10
8	2	16	+220	10
9	2	18	+240	10
10	2	20	+275	17.5
11	2	22	+295	10
12	2	24	+320	12.5
13	2	26	+335	7.5
14	2	28	+360	12.5
15	2	30	+385	12.5
16	2	32	+400	7.5
17	2	34	+420	10
18	2	36	+450	15
19	6	42	+590	23.33
20	2	44	+615	12.5
21	**2**	**46**	**+640**	**12.5**

Penetration Data Report for Aweliya [AW2-4] [1.80m - 2.87m]

Site Name:	**Aweliya** (09⁰04'16.1", 38⁰44' 05.1")	Test Date:	25/05/2011 [DD/MM/YY]
Test Depth (m):	1.80		
Cone Angle (deg.):	60		
Soil Data:	Moist, high plastic, silty CLAY [Red clay soil]		

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
22	**2**	**48**	**+670**	**15**
23	**2**	**50**	**+690**	**10**
24	**2**	**52**	**+715**	**12.5**
25	2	54	+740	12.5
26	2	56	+770	15
27	2	58	+795	12.5
28	2	60	+830	17.5
29	2	62	+855	12.5
30	2	64	+890	17.5
31	2	66	+920	15
32	2	68	+960	20
33	2	70	+990	15
34	2	72	+1030	20
35	2	74	+1070	20

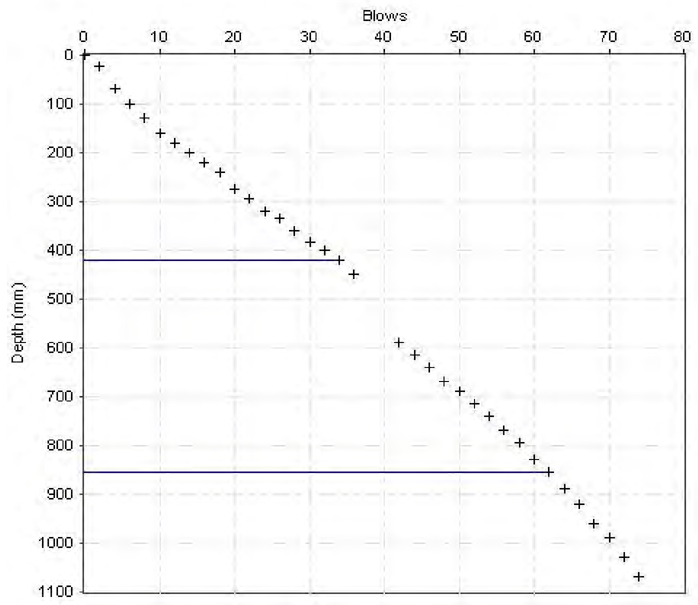

Figure A-1.7 (b) Penetration Data Report for Aweliya [AW2-4] [1.80m - 2.87m]

Table A - 1.8 (a) Penetration Data Report for Addis Ketema [AK1-4]

Penetration Data Report for Addis Ketema [AK1-4] [1.00m - 2.08m]

Site Name:	**Addis Ketema**	Test Date:	26/06/2011 [DD/MM/YY]
Test Depth (m):	1.00		
Cone Angle (deg.):	60		
Soil Data:	Wet, high plastic, CLAY [black cotton /expansive/ soil]		

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
-	-	-	**1000**	-

Penetration Data Report for Addis Ketema [AK1-4] [1.00m - 2.08m]

Site Name: **Addis Ketema** Test Date: 26/06/2011 [DD/MM/YY]

Test Depth (m): 1.00

Cone Angle (deg.): 60

Soil Data: Wet, high plastic, CLAY [black cotton /expansive/ soil]

1	**2**	**2**	**+20**	**9.5**
2	2	4	+65	22.5
3	2	6	+85	10
4	2	8	+110	12.5
5	2	10	+145	17.5
6	2	12	+170	12.5
7	2	14	+200	15
8	2	16	+225	12.5
9	2	18	+250	12.5
10	2	20	+280	15
11	2	22	+310	15
12	2	24	+345	17.5
13	2	26	+380	17.5
14	2	28	+415	17.5
15	2	30	+445	15
16	2	32	+475	15
17	2	34	+500	12.5
18	2	36	+525	12.5
19	**2**	**38**	**+550**	**12.5**
20	2	40	+580	15
21	2	42	+600	10
22	2	44	+625	12.5
23	**2**	**46**	**+655**	**15**
24	2	48	+680	12.5
25	2	50	+710	15
26	2	52	+735	12.5
27	2	54	+760	12.5
28	2	56	+785	12.5
29	2	58	+815	15
30	2	60	+845	15

Penetration Data Report for Addis Ketema [AK1-4] [1.00m - 2.08m]

Site Name: **Addis Ketema** Test Date: 26/06/2011 [DD/MM/YY]

Test Depth (m): 1.00

Cone Angle (deg.): 60

Soil Data: Wet, high plastic, CLAY [black cotton /expansive/ soil]

31	2	62	+870	12.5
32	2	64	+905	17.5
33	2	66	+930	12.5
34	2	68	+955	12.5
35	2	70	+980	12.5
36	2	72	+1010	15
37	2	74	+1080	35

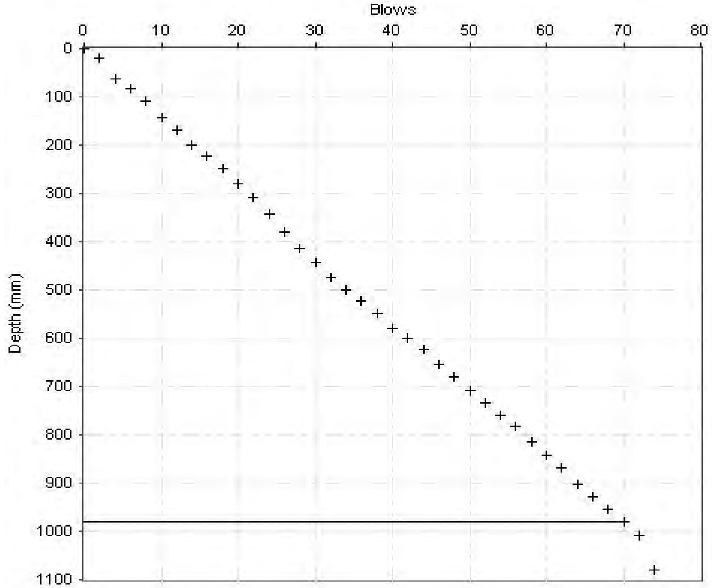

Figure A-1.8 (a) Penetration Data Report for Addis Ketema [AK1-4] [1.00m - 2.08m]

Table A - 1.8 (b) Penetration Data Report for Addis Ketema[AK5]

Penetration Data Report for Addis Ketema [AK5] [2.00m - 3.08m]

Site Name:	**Addis Ketema**	Test Date:	26/06/2011 [DD/MM/YY]
Test Depth (m):	2.00		
Cone Angle (deg.):	60		
Soil Data:	Wet, high plastic, CLAY [black cotton /expansive/ soil]		

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
-	-	-	**2000**	-
1	2	2	+45	22
2	2	4	+80	17.5
3	2	6	+110	15
4	2	8	+160	25
5	2	10	+195	17.5
6	2	12	+230	17.5
7	2	14	+260	15
8	2	16	+295	17.5
9	2	18	+330	17.5
10	2	20	+360	15
11	2	22	+390	15
12	2	24	+430	20
13	2	26	+470	20
14	2	28	+505	17.5
15	2	30	+550	22.5
16	**2**	**32**	**+590**	**20**
17	2	34	+630	20
18	2	36	+660	15
19	2	38	+690	15
20	2	40	+725	17.5
21	2	42	+770	22.5
22	2	44	+815	22.5
23	2	46	+865	25
24	2	48	+915	25
25	2	50	+970	27.5
26	2	52	+1030	30
27	2	54	+1075	22.5

Penetration Data Report for Addis Ketema [AK5] [2.00m - 3.08m]

Site Name: **Addis Ketema** Test Date: 26/06/2011 [DD/MM/YY]

Test Depth (m): 2.00

Cone Angle (deg.): 60

Soil Data: Wet, high plastic, CLAY [black cotton /expansive/ soil]

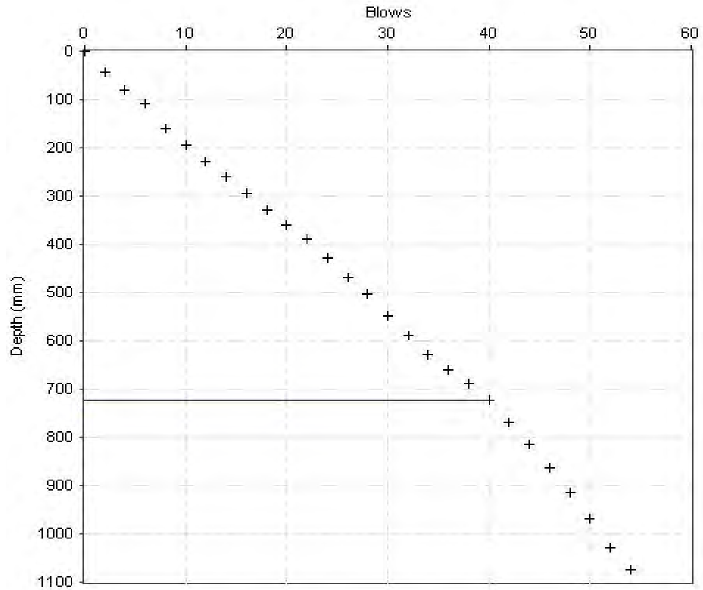

Figure A-1.8 (a) Penetration Data Report for Addis Ketema [AK5] [2.00m - 3.08m]

A – 2) DCPI Result of Some of the sites Outside Addis Ababa

Table A - 2.1 Penetration Data Report for Sacha[SA_1]

Penetration Data Report for Sacha [0.0m - 1.5m]

Project Name: **Sacha**

Test Depth (m): 0.00 Test Date: 27-09-2011 [DD/MM/YY]

Cone Angle (deg.) 60

Soil Data: Moist, high plastic, silty CLAY [Red clay soil]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
-	-	-	**0**	-
2	2	2	+166	82.5
3	2	4	+260	47
4	2	6	+340	40
5	2	8	+420	40
6	2	10	+490	35
7	2	12	+560	35
8	2	14	+620	30
9	2	16	+675	27.5
10	3	19	+760	28.33
11	3	22	+840	26.67
12	3	25	+915	25
13	3	28	+995	26.67
14	3	31	+1095	33.33
15	3	34	+1160	21.67
16	3	37	+1230	23.33
17	3	40	+1295	21.67
18	3	43	+1360	21.67
19	3	46	+1435	25
20	3	49	+1500	21.67

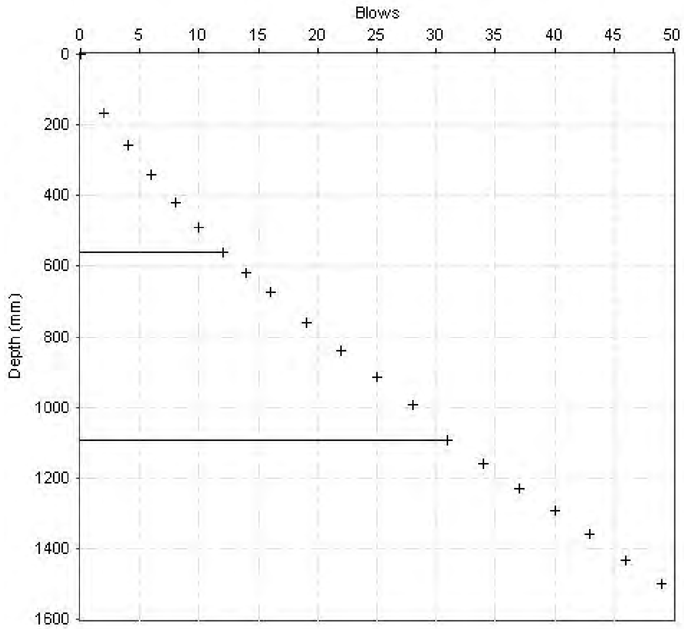

Figure A-2.1 (a) Penetration Data Report for Sacha [SA_1]

Table A - 2.1 (b) Penetration Data Report for Sacha[SA_2]

Penetration Data Report for Sacha [1.5m - 2.95m]

Project Name: **Sacha**

Test Depth (m): 1.50 Test Date: 27-09-2011 [DD/MM/YY]

Cone Angle (deg.) 60

Soil Data: Moist, high plastic, silty CLAY [Red clay soil]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
-	-	-	**1500**	-
2	4	4	+85	21
3	4	8	+155	17.5
4	4	12	+225	17.5

Penetration Data Report for Sacha [1.5m - 2.95m]				
Project Name:	**Sacha**			
Test Depth (m):	1.50	Test Date:	27-09-2011 [DD/MM/YY]	
Cone Angle (deg.)	60			
Soil Data:	Moist, high plastic, silty CLAY [Red clay soil]			

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
5	4	16	+285	15
6	4	20	+360	18.75
7	4	24	+425	16.25
8	4	28	+485	15
9	4	32	+555	17.5
10	4	36	+610	13.75
11	4	40	+680	17.5
12	4	44	+755	18.75
13	6	50	+815	10
14	6	56	+905	15
15	6	62	+990	14.17
16	6	68	+1080	15
17	6	74	+1160	13.33
18	6	80	+1215	9.17
19	6	86	+1315	16.67
20	6	92	+1395	13.33
21	6	98	+1495	16.67

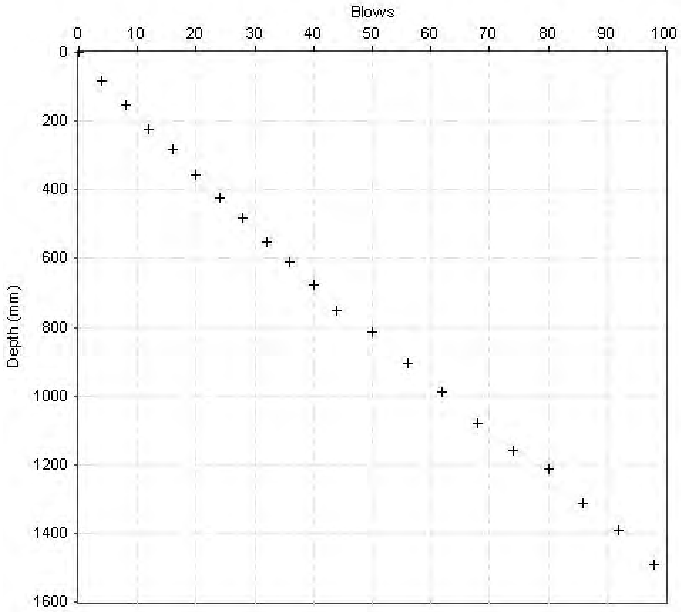

Figure A-2.1 (b) Penetration Data Report for Sacha [SA_2] [1.5m – 2.95m]

Table A - 2.2 (a) Penetration Data Report for Wodito [WO_1]

Penetration Data Report for Wodito [0.0m - 1.5m]				
Project Name:	**Wodito**			
Test Depth (m):	0.00	Test Date:	30-09-2011 [DD/MM/YY]	
Cone Angle (deg.)	60			
Soil Data:	Wet, black, high plastic, CLAY [black cotton /expansive/ soil] [The top 0.98m]			

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)

Penetration Data Report for Wodito [0.0m - 1.5m]

Project Name: **Wodito**
Test Depth (m): 0.00 Test Date: 30-09-2011 [DD/MM/YY]
Cone Angle (deg.) 60
Soil Data: Wet, black, high plastic, CLAY [black cotton /expansive/ soil] [The top 0.98m]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
-	-	-	**0**	-
2	2	2	+575	287
3	2	4	+750	87.5
4	1	5	+860	110
5	2	7	+925	32.5
6	2	9	+980	27.5
7	2	11	+1005	12.5
8	2	13	+1045	20
9	2	15	+1065	10
10	2	17	+1090	12.5
11	2	19	+1110	10
12	2	21	+1135	12.5
13	2	23	+1165	15
14	2	25	+1195	15
15	2	27	+1225	15
16	2	29	+1255	15
17	2	31	+1295	20
18	2	33	+1320	12.5
19	2	35	+1350	15
20	2	37	+1390	20
21	2	39	+1435	22.5
22	2	41	+1475	20

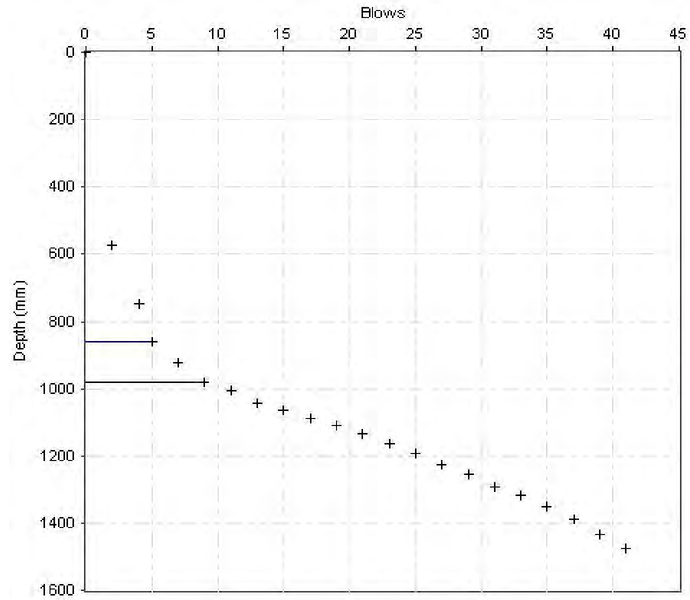

Figure A-2.2 (a) Penetration Data Report for Wodito [WO_1] [0.0m – 1.5m]

Table A - 2.2 (b) Penetration Data Report for Wodito [WO_2]

Penetration Data Report for Wodito [1.5m - 3.0m]

Project Name: **Wodito**
Test Depth (m): 1.50 Test Date: 30-09-2011 [DD/MM/YY]
Cone Angle (deg.) 60
Soil Data: Wet, black, high plastic, CLAY [black cotton /expansive/ soil]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
-	-	-	**1500**	-
2	4	4	+55	13.5
3	4	8	+145	22.5
4	4	12	+210	16.25

Penetration Data Report for Wodito [1.5m - 3.0m]

Project Name: **Wodito**

Test Depth (m): 1.50 Test Date: 30-09-2011 [DD/MM/YY]

Cone Angle (deg.) 60

Soil Data: Wet, black, high plastic, CLAY [black cotton /expansive/ soil]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
5	4	16	+260	12.5
6	4	20	+345	21.25
7	4	24	+440	23.75
8	4	28	+475	8.75
9	4	32	+515	10
10	4	36	+555	10
11	4	40	+615	15
12	4	44	+725	27.5
13	4	48	+835	27.5
14	6	54	+905	11.67
15	6	60	+990	14.17
16	6	66	+1080	15
17	6	72	+1155	12.5
18	6	78	+1220	10.83
19	6	84	+1285	10.83
20	4	88	+1355	17.5
21	4	92	+1385	7.5
22	4	96	+1415	7.5
23	4	100	+1445	7.5
24	4	104	+1475	7.5

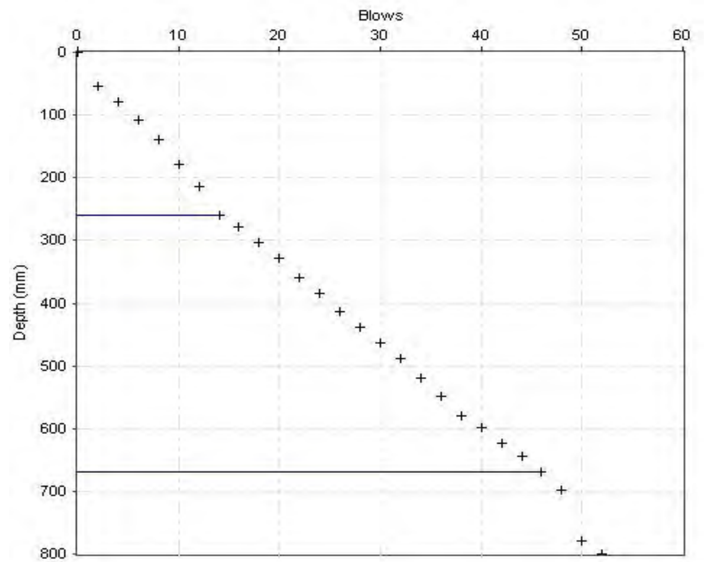

Figure A-2.2 (b) Penetration Data Report for Wodito [WO_2] [1.5m – 3.0m]

Table A - 2.3 (a) Penetration Data Report for Gerbicho [GR1]

Penetration Data Report for Gerbicho [0.0m - 1.5m]

Project Name: **Gerbicho** Test Date: 19-09-2011 [DD/MM/YY]

Test Depth (m): 0

Cone Angle (deg.) 60

Soil Data: Moist, high plastic, silty CLAY [Red clay soil]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
-	-	-	**0**	-
2	2	2	+19	9
3	2	4	+305	143
4	2	6	+380	37.5
5	2	8	+460	40

Penetration Data Report for Gerbicho [0.0m - 1.5m]

Project Name: **Gerbicho** Test Date: 19-09-2011 [DD/MM/YY]

Test Depth (m): 0

Cone Angle (deg.) 60

Soil Data: Moist, high plastic, silty CLAY [Red clay soil]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
6	2	10	+550	45
7	2	12	+640	45
8	2	14	+725	42.5
9	2	16	+810	42.5
10	2	18	+880	35
11	1	19	+945	65
12	2	21	+1010	32.5
13	2	23	+1070	30
14	2	25	+1120	25
15	2	27	+1180	30
16	2	29	+1225	22.5
17	2	31	+1275	25
18	2	33	+1325	25
19	2	35	+1380	27.5
20	2	37	+1430	25
21	2	39	+1475	22.5

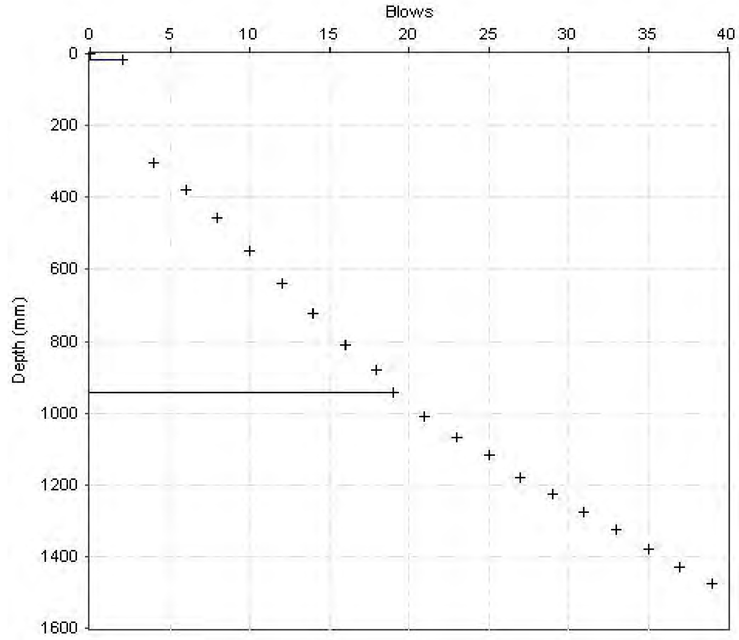

Figure A-2.3 (a) Penetration Data Report for Gerbicho [GR_1] [0.0m – 1.5m]

Table A - 2.3 (b) Penetration Data Report for Gerbicho[GR_2]

Penetration Data Report for Gerbicho [1.5m - 2.35m]

Project Name: **Gerbicho** Test Date: 19-09-2011 [DD/MM/YY]

Test Depth (m): 1.50

Cone Angle (deg.) 60

Soil Data: Moist, high plastic, silty CLAY [Red clay soil]

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
-	-	-	**1500**	-
2	2	2	+50	24.5
3	4	6	+115	16.25

Penetration Data Report for Gerbicho [1.5m - 2.35m]

Project Name:	**Gerbicho**	Test Date:	19-09-2011 [DD/MM/YY]
Test Depth (m):	1.50		
Cone Angle (deg.)	60		
Soil Data:	Moist, high plastic, silty CLAY [Red clay soil]		

S.N.	Blows	Cumulative Blows	Penetration Depth (mm)	Penetration Rate (mm/blow)
4	4	10	+245	32.5
5	4	14	+305	15
6	4	18	+365	15
7	4	22	+415	12.5
8	4	26	+470	13.75
9	4	30	+540	17.5
10	4	34	+595	13.75
11	4	38	+650	13.75
12	4	42	+695	11.25
13	4	46	+740	11.25
14	4	50	+795	13.75
15	4	54	+850	13.75

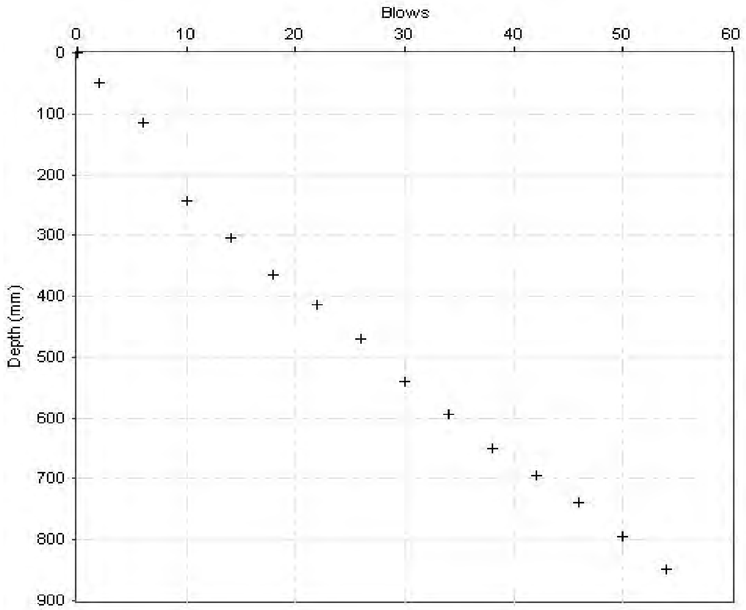

Figure A-2.3 (b) Penetration Data Report for Gerbicho [GR_2] [1.5m – 2.25m]

APPENDIX – B: Laboratory Test Results

B – 1) Moisture Content Determination

Table B - 1.1 Moisture Content for Addisu Gebeya [AG]

Determination no-	1	2
Container no-	P8	A1
Mass of container, g	15.662	15.594
Mass of container +wet soil, g	60.712	61.92
Mass of container +dry soil, g	51.412	54.64
Mass of water, g	9.3	7.28
Mass of dry soil, g	35.75	39.046
Water content, %	26.01	18.64
Average water content, %	**22.33**	

Table B - 1.2Moisture Content for Atari [AT$_1$] at 1.5m

Determination no-	1	2
Container no-	16	C18
Mass of container, g	14.41	13.54
Mass of container +wet soil, g	49.212	47.2
Mass of container +dry soil, g	41.1	39.433
Mass of water, g	8.112	7.767
Mass of dry soil, g	26.69	25.893
Water content, %	30.39	30.00
Average water content, %	**30.19**	

Table B - 1.3 Moisture Content for Atari [AT$_2$] at 3.0m

Determination no-	1	2
Container no-	74	12
Mass of container, g	15.8815	15.868
Mass of container +wet soil, g	49.42	46.46
Mass of container +dry soil, g	41.895	39.5
Mass of water, g	7.525	6.96
Mass of dry soil, g	26.0135	23.632
Water content, %	28.93	29.45
Average water content, %	**29.19**	

Table B - 1.4 Moisture Content for Aweliya [AW$_1$] at 1.5m

Determination no-	1	2
Container no-	7	H3
Mass of container, g	15.574	14.055
Mass of container +wet soil, g	45.3	47.54
Mass of container +dry soil, g	38.974	40.43
Mass of water, g	6.326	7.11
Mass of dry soil, g	23.4	26.375
Water content, %	27.03419	26.957
Average water content, %	**27.00**	

Table B - 1.5 Moisture Content for Aweliya [AW$_{2\&3}$] at 2.4m

Determination no-	1	2
Container no-	10b	70
Mass of container, g	15.582	15.721
Mass of container +wet soil, g	56.4	47.36
Mass of container +dry soil, g	45.78	39.765
Mass of water, g	10.62	7.595
Mass of dry soil, g	30.198	24.044
Water content, %	35.16789	31.5879
Average water content, %	**33.38**	

Table B - 1.6 Moisture Content for Aweliya [AW$_4$] at 3.0m

Determination no-	1	2
Container no-	A34	67
Mass of container, g	15.8815	15.5

Determination no-	1	2
Mass of container +wet soil, g	49.42	67.8
Mass of container +dry soil, g	41.895	54.9
Mass of water, g	7.525	12.9
Mass of dry soil, g	26.0135	39.4
Water content, %	28.92729	32.741
Average water content, %	**30.83**	

Table B - 1.7 Moisture Content for Addis Ketema[$AK_{3\&4}$] at 1.5m

Determination no-	1	2
Container no-	7	H3
Mass of container, g	15.2928	15.666
Mass of container +wet soil, g	36.879	32.789
Mass of container +dry soil, g	31.711	28.484
Mass of water, g	5.168	4.305
Mass of dry soil, g	16.4182	12.818
Water content, %	31.47726	33.58558
Average water content, %	**32.53**	

Table B - 1.8 Moisture Content for Addis Ketema[AK_5] at 3.0m

Determination no-	1	2
Container no-	A34	67
Mass of container, g	13.8957	15.6901
Mass of container +wet soil, g	31.575	33.536
Mass of container +dry soil, g	26.56	28.45
Mass of water, g	5.015	5.086
Mass of dry soil, g	12.6643	12.7599
Water content, %	39.5995	39.85925
Average water content, %	**39.73**	

Table B - 1.9 Moisture Content for Meskel Flower [MF_1] at 3.0m

Determination no-	1	2	3
Container no-	66	43	78
Mass of container, g	5.33	5.17	5.23
Mass of container +wet soil, g	63.11	62.32	53.78
Mass of container +dry soil, g	47.25	46.6	41.16
Mass of water, g	15.86	15.72	12.62
Mass of dry soil, g	41.92	41.43	35.93
Water content, %	37.83397	37.94352	35.12385
Average water content, %	**36.97**		

Table B - 1.10 Moisture Content for Meskel Flower [MF_2] at 3.0m

Determination no-	1	2	3
Container no-	D1	13	4
Mass of container, g	5.35	5.22	5.18
Mass of container +wet soil, g	54.2	52.6	55.18
Mass of container +dry soil, g	42.5	41.2	43.2
Mass of water, g	11.7	11.4	11.98
Mass of dry soil, g	37.15	35.98	38.02
Water content, %	31.49394	31.68427	31.50973
Average water content, %	**31.56**		

B – 2) Bulk Density Determination

Table B - 2.1 Bulk Density for Addisu Gebeya [AG]

Area of Specimen, m^2	0.0011341
Dia. Of specimen, mm	38
Height of Specimen, mm	76
Moisture content	18.645
Mass, gm	161
Wet Unit Weight Computation	18.324166
Dry Unit Weight Computation	15.444533
Dry Density, gm/cm^3	1.5743662

Table B - 2.2 (a) Bulk Density for Atari [AT_1] at 1.5m

Area of Specimen,m^2	0.0011341
Dia. Of specimen, mm	38.00
Height of Specimen, mm	76.00
Moisture content	30.19
Mass, gm	167.5
Wet Unit Weight Computation	19.06
Dry Unit Weight Computation	14.64
Dry Density, gm/cm^3	1.49

Table B - 2.2 (b) Bulk Density for Atari [AT_2] at 3.0m

Area of Specimen, m^2	0.0011341
Dia. Of specimen, mm	38.00
Height of Specimen, mm	76.00
Moisture content	29.19
Mass, gm	165.23
Wet Unit Weight Computation	18.81
Dry Unit Weight Computation	14.56
Dry Density, gm/cm^3	1.48

Table B - 2.3 (a) Bulk Density for Aweliya [AW_1] at 1.5m

Area of Specimen, m^2	0.001134
Dia. Of specimen, mm	38
Height of Specimen, mm	76
Moisture content	27.00
Mass, gm	154.91
Wet Unit Weight Computation	17.63
Dry Unit Weight Computation	13.88
Dry Density, gm/cm^3	1.42

Table B - 2.3 (b) Bulk Density for Aweliya [AW_2] at 2.4m

Area of Specimen, m^2	0.001134
Dia. Of specimen, mm	38
Height of Specimen, mm	76
Moisture content	31.59
Mass, gm	157.9
Wet Unit Weight Computation	17.97
Dry Unit Weight Computation	13.66
Dry Density, gm/cm^3	1.39

Table B - 2.3 (c) Bulk Density for Aweliya [AW_3] at 2.5m

Area of Specimen, m^2	0.001134
Dia. Of specimen, mm	38
Height of Specimen, mm	76
Moisture content	35.16789
Mass, gm	157.17
Wet Unit Weight Computation	17.89
Dry Unit Weight Computation	13.23
Dry Density, gm/cm^3	1.35

Table B - 2.3 (d) Bulk Density for Aweliya [AW_4] at 3.0m

Area of Specimen, m^2	0.001134
Dia. Of specimen, mm	38
Height of Specimen, mm	76
Moisture content	41.08
Mass, gm	149.23
Wet Unit Weight Computation	16.98
Dry Unit Weight Computation	12.04
Dry Density, gm/cm^3	1.23

Table B - 2.4 (a) Bulk Density for Addis Ketema [AK_1] at 0.25m

Area of Specimen, m^2	0.001134
Dia. Of specimen, mm	38
Height of Specimen, mm	76
Moisture content	20.96
Mass, gm	132.6
Wet Unit Weight Computation	15.09
Dry Unit Weight Computation	12.48
Dry Density, gm/cm^3	1.27

Table B - 2.4 (b) Bulk Density for Addis Ketema [AK_2] at 0.35m

Area of Specimen, m^2	0.001134
Dia. Of specimen, mm	38
Height of Specimen, mm	76
Moisture content	21.80
Mass, gm	132.745
Wet Unit Weight Computation	15.11
Dry Unit Weight Computation	12.40
Dry Density, gm/cm^3	1.26

Table B - 2.4 (c) Bulk Density for Addis Ketema [AK_3] at 1.55m

Area of Specimen, m^2	0.001134
Dia. Of specimen, mm	38
Height of Specimen, mm	76
Moisture content	32.53
Mass, gm	164.745
Wet Unit Weight Computation	18.75
Dry Unit Weight Computation	14.15
Dry Density, gm/cm^3	1.44

Table B - 2.4 (d) Bulk Density for Addis Ketema [AK_4] at 1.65m

Area of Specimen, m^2	0.001134
Dia. Of specimen, mm	38
Height of Specimen, mm	76
Moisture content	32.83
Mass, gm	160.329
Wet Unit Weight Computation	18.25
Dry Unit Weight Computation	13.74
Dry Density, gm/cm^3	1.40

Table B - 2.4 (e) Bulk Density for Addis Ketema [AK_5] at 2.60m

Area of Specimen, m^2	0.001134
Dia. Of specimen, mm	38
Height of Specimen, mm	76
Moisture content	39.73
Mass, gm	153.48
Wet Unit Weight Computation	17.47
Dry Unit Weight Computation	12.50
Dry Density, gm/cm^3	1.27

B – 3) Specific Gravity Determination

Table B-3.1 Specific Gravity for Addisu Gebeya [AG]

Determination No.	1	2
Pycnometer No.	P1	20
Weight of pycnometer + soil + water, W_{pws} (g)	157.31	157.47
Temperature, T_x(°c)	25.6	25.6
Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g)	144.71	144.68
Weight of dry soil , w_s (gm)	20	20
Conversion factor , K	0.9989	0.9989
Specific gravity of soil at 20°c.	2.61	2.66
Average specific gravity of soil.	**2.64**	

Table B-3.2 (a) Specific Gravity for Atari [AT_1] at 1.5m

Determination No.	1	2
Pycnometer No.	P10	P1
Weight of pycnometer + soil + water, W_{pws} (g)	162.1	162.21
Temperature, T_x(°c)	24.6	24.6
Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g)	145.28	144.76
Weight of dry soil , w_s (gm)	26	27
Conversion factor , K	0.9989	0.9989
Specific gravity of soil at 20°c.	2.76	2.81
Average specific gravity of soil.	**2.78**	

Table B-3.2 (b) Specific Gravity for Atari [AT_2] at 3.0m

Determination No.	1	2
Pycnometer No.	Pp	20
Weight of pycnometer + soil + water, W_{pws} (g)	164.70 2	161.53 1
Temperature, T_x(°c)	24.6	24.6
Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g)	148.11	144.82
Weight of dry soil , w_s (gm)	26	26
Conversion factor , K	0.9989	0.9989
Specific gravity of soil at 20°c.	2.73	2.85
Average specific gravity of soil.	**2.79**	

Table B-3.3 (a) Specific Gravity for Aweliya [AW_1] at 1.5m

Determination No.	1	2
Pycnometer No.	P1	P3
Weight of pycnometer + soil + water, W_{pws} (g)	165.3646	168.079
Temperature, T_x(°c)	23.2	23.6
Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g)	144.78	149.03
Weight of dry soil , w_s (gm)	32	29
Conversion factor , K	0.9989	0.9989
Specific gravity of soil at 20°c.	2.83	2.88
Average specific gravity of soil.	**2.85**	

Table B-3.3 (b) Specific Gravity for Aweliya [$AW_{2\&3}$] at 2.4m

Determination No.	1	2
Pycnometer No.	P10	Pp
Weight of pycnometer + soil + water, W_{pws} (g)	167.73	168.0542
Temperature, T_x(°c)	22.2	22.6
Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g)	145.29	148.01
Weight of dry soil , w_s (gm)	34	31
Conversion factor , K	0.9989	0.9989
Specific gravity of soil at 20°c.	2.89	2.90
Average specific gravity of soil.	**2.90**	

Table B-3.3 (c) Specific Gravity for Aweliya [AW_4] at 3.1m

Determination No.	1	2
Pycnometer No.	P9	20
Weight of pycnometer + soil + water, W_{pws} (g)	173.9373	169.2695
Temperature, T_x(°c)	21.9	21.9
Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g)	149.03	144.84
Weight of dry soil , w_s (gm)	38	37
Conversion factor , K	0.9989	0.9989
Specific gravity of soil at 20°c.	2.92	2.91
Average specific gravity of soil.	**2.92**	

Table B-3.4 (a) Specific Gravity for Addis Ketema [$AK_{3\&4}$] at 1.5m

Determination No.	1	2
Pycnometer No.	P20	P18
Weight of pycnometer + soil + water, W_{pws} (g)	161.19	161.83
Temperature, T_x(°c)	21	21
Weight of pycnometer + water at T_x , W_{pw}(atT_x) (g)	145.02	145.69
Weight of dry soil , w_s (gm)	25	25
Conversion factor , K	0.9998	0.9998
Specific gravity of soil at 20°c.	2.83	2.82
Average specific gravity of soil.	**2.82**	

Table B-3.4 (b) Specific Gravity for Addis Ketema [AK_5] at 3.0m

Determination No.	1	2
Pycnometer No.	p1	2
Weight of pycnometer + soil + water, W_{pws} (g)	165.423	165.733
Temperature, T_x(°c)	21	21
Weight of pycnometer + water at T_x , W_{pw}(atT_x) (g)	149.45	149.31
Weight of dry soil , w_s (gm)	25	25
Conversion factor , K	0.9998	0.9998
Specific gravity of soil at 20°c.	2.77	2.91
Average specific gravity of soil.	**2.84**	

Table B-3.5 Specific Gravity for Meskel Flower [MF_1] at 3.0m

Determination No.	1	2	3
Pycnometer No.	P7	P41	P11
Weight of pycnometer + soil + water, W_{pws} (g)	160.379	165.03 4	160.59
Temperature, T_x(°c)	23.8	23.4	24
Weight of pycnometer + water at T_x , W_{pw}(atT_x) (g)	145.26	149.39	145.38
Weight of dry soil , w_s (gm)	25	26	25
Conversion factor , K	0.9991	0.9992	0.9991
Specific gravity of soil at 20°c.	2.56	2.57	2.56
Average specific gravity of soil .	**2.56**		

B – 4) Atterberg Limits Determination

Table B - 4.1Atterberg Limits for Addisu Gebeya [AG]

Location: Addisu Gebeya
Date: 09/05/2011
Depth, m: 2.00

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	1	2	3	4	5	6
Mass of container, g	6.42	6.33	6.52	6.49	6.33	6.32
Mass of container + Wet soil, g	23.35	29.14	32.55	27.42	9.39	9.71
Mass of container + Dry soil, g	16.30	19.71	21.70	18.75	8.56	8.83
Mass of water, g	7.05	9.43	10.85	8.68	0.83	0.88
Mass of dry soil, g	9.89	13.38	15.18	12.26	2.23	2.52
Water content, %	71.28	70.51	71.46	70.79	37.23	34.82
No of blows	15	29	19	25	------	------

Liquid Limit, % = 70.8 **Plastic Limit, % = 36.0** **PI, %= 35**

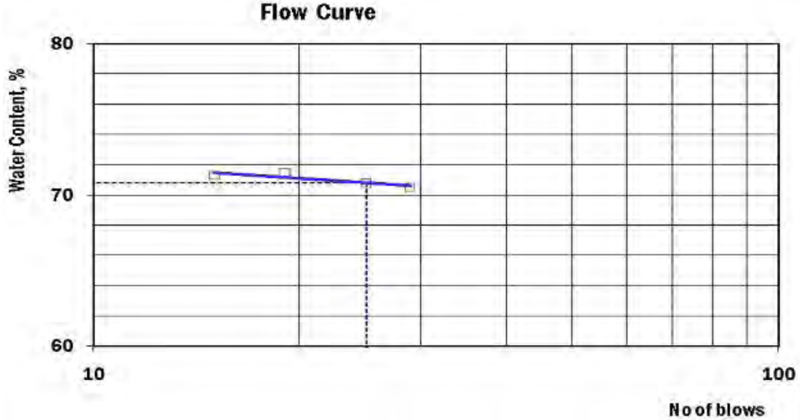

Figure B -4.1Atterberg Limits for Addisu Gebeya [AG]

Table B - 4.2 (a) Atterberg Limits for Atari [AT1] at 1.5m

Location: Atari
Date: 18/05/2011
Depth, m: 1.5

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	36	30	24	4	67	68
Mass of container, g	13.89	15.67	15.65	15.67	15.50	15.50
Mass of container + Wet soil, g	43.42	42.56	49.48	30.54	25.00	25.00
Mass of container + Dry soil, g	31.40	31.83	35.97	24.77	22.45	22.45
Mass of water, g	12.02	10.73	13.52	5.77	2.55	2.55
Mass of dry soil, g	17.51	16.16	20.32	9.10	6.95	6.95
Water content, %	68.64	66.40	66.54	63.40	36.69	36.69
No of blows	15	23	20	35	------	------

Liquid Limit, % =65.5 **Plastic Limit, % = 36.7** **PI, %= 29**

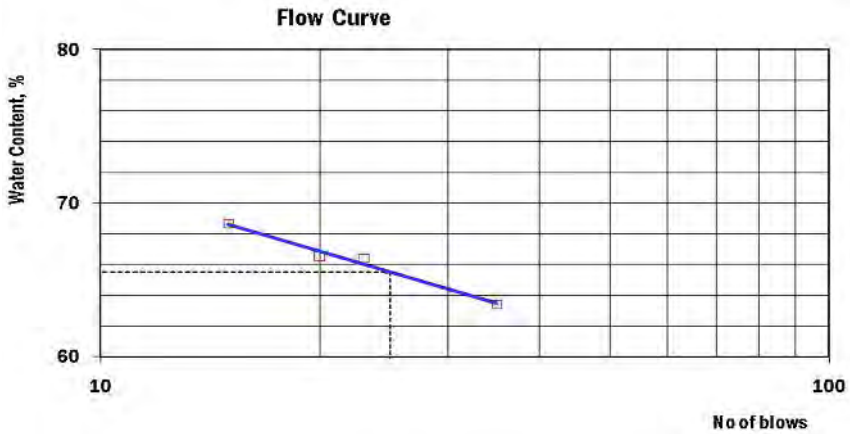

Figure B -4.2 (a) Atterberg Limits for Atari [AT1] at 1.5m

Table B - 4.2 (b) Atterberg Limits for Atari [AT_2] at 3.0m

Location: Atari
Date: 18-05-2011
Depth, m: 3.00

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	A1	70	D-5	D-33	7	A34
Mass of container, g	15.18	15.72	15.70	15.51	15.56	15.67
Mass of container + Wet soil, g	41.09	36.05	45.37	40.00	21.80	22.46
Mass of container + Dry soil, g	30.50	27.64	33.23	29.90	19.93	20.44
Mass of water, g	10.60	8.41	12.14	10.10	1.87	2.02
Mass of dry soil, g	15.31	11.92	17.53	14.39	4.37	4.77
Water content, %	69.21	70.55	69.25	70.19	42.79	42.35
No of blows		23	30	27	------	------

Liquid Limit, % = 70.3 **Plastic Limit, % =42.6** **PI, %=28**

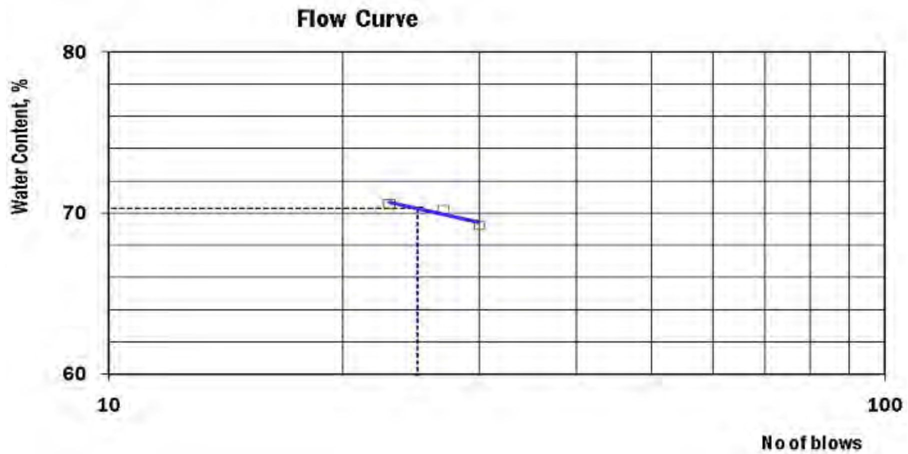

Figure B -4.2 (b) Atterberg Limits for Atari [AT_2] at 3.0m

Table B - 4.3 (a) Atterberg Limits for Aweliya [AW_1] at 1.5m

Location: Aweliya
Date: 25-05-2011
Depth, m: 1.5

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	3.1	47	D15	D22	108	13
Mass of container, g	15.47	15.68	15.62	15.76	15.65	13.89
Mass of container + Wet soil, g	29.11	30.28	30.95	30.86	19.10	17.88
Mass of container + Dry soil, g	24.38	25.00	25.38	25.19	18.26	16.91
Mass of water, g	4.73	5.29	5.57	5.68	0.84	0.97
Mass of dry soil, g	8.92	9.32	9.76	9.43	2.61	3.02
Water content, %	53.04	56.77	57.13	60.21	32.21	32.11
No of blows	35	25	22	16	------	------

Liquid Limit, % = 56.3 **Plastic Limit, % = 32.2** **PI, %= 24**

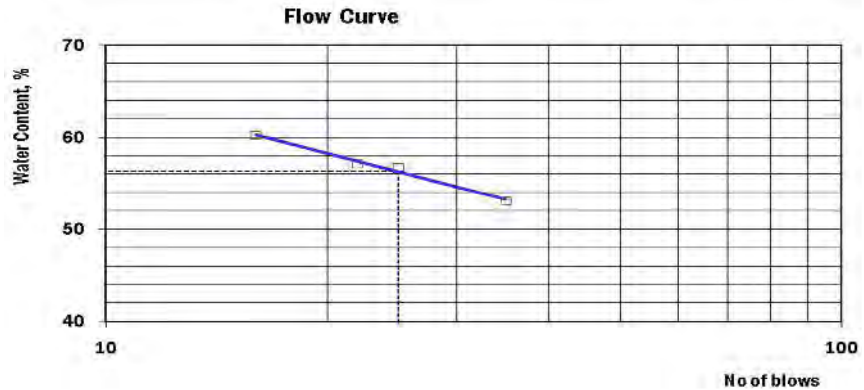

Figure B -4.3 (b) Atterberg Limits for Aweliya [AW_1] at 1.5m

Table B - 4.3 (c) Atterberg Limits for Aweliya [$AW_{2\&3}$] at 2.4m

Location: Aweliya
Date: 25-05-2011
Depth, m: 2.4

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	C18	63	103	A1	A31	24
Mass of container, g	13.55	15.60	15.61	15.60	15.83	15.66
Mass of container + Wet soil, g	28.17	31.38	31.05	31.60	19.13	18.36
Mass of container + Dry soil, g	22.31	24.89	24.83	25.04	18.19	17.59
Mass of water, g	5.86	6.49	6.22	6.56	0.94	0.77
Mass of dry soil, g	8.76	9.29	9.23	9.44	2.36	1.93
Water content, %	66.91	69.90	67.43	69.46	39.59	39.81
No of blows	32	18	30	22	------	------

Liquid Limit, % = 68.4 Plastic Limit, % =39.7 PI, %=29

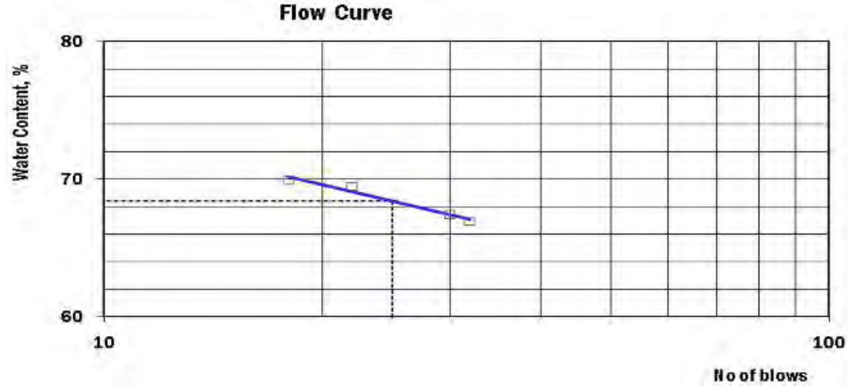

Figure B -4.3 (c) Atterberg Limits for Aweliya [$AW_{2\&3}$] at 2.4m

Table B - 4.3 (d) Atterberg Limits for Aweliya [AW_4] at 3.0m

Location: Aweliya
Date: 25-05-2011
Depth, m: 3.10

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	15	40	12	36	C61	H2
Mass of container, g	15.51	15.34	15.87	13.88	14.05	15.71
Mass of container + Wet soil, g	31.89	33.45	28.98	27.03	20.94	22.37
Mass of container + Dry soil, g	25.78	26.86	24.13	22.19	19.12	20.61
Mass of water, g	6.11	6.59	4.85	4.84	1.82	1.76
Mass of dry soil, g	10.27	11.52	8.26	8.31	5.07	4.91
Water content, %	59.48	57.21	58.72	58.26	35.93	35.88
No of blows	16	35	20	26	------	------

Liquid Limit, % = 58.3; Plastic Limit, % =35.9; PI, %=22

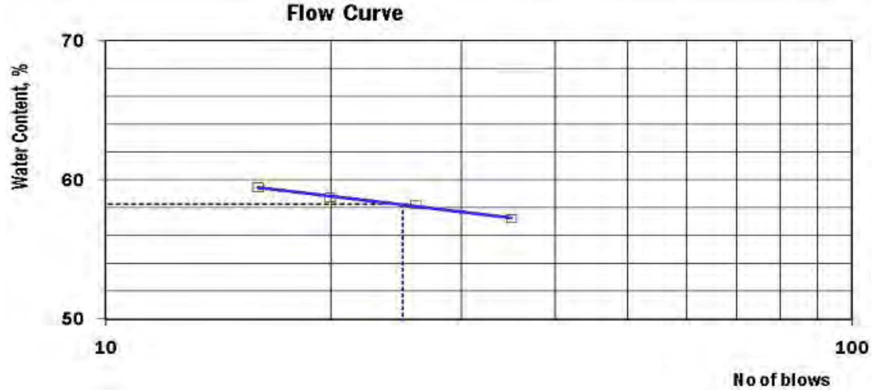

Figure B -4.3 (d) Atterberg Limits for Aweliya [AW_4] at 3.0m

Table B - 4.4 (a) Atterberg Limits for Addis Ketema (Worede - 10) at 1.5m

Location: Addis Ketema (Woreda_10)

Date: 26-06-2011

Depth, m: 1.5

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	E2	5.2	2.1	D20	D4	B1
Mass of container, g	15.67	15.57	15.6 3	15.5 0	15.9 8	15.9 6
Mass of container + Wet soil, g	47.11	42.18	40.6 5	33.4 6	20.2 5	22.3 6
Mass of container + Dry soil, g	34.76	31.96	31.5 8	26.9 1	19.0 0	20.5 0
Mass of water, g	12.35	10.22	9.07	6.55	1.25	1.86
Mass of dry soil,g	19.09	16.39	15.9 5	11.4 1	3.02	4.54
Water content, %	64.69	62.33	56.8 7	57.4 6	41.3 7	40.9 5
No of blows	15	24	34	26	------	------

Liquid Limit, % = 60.0; Plastic Limit, % =41.2; PI, %=19

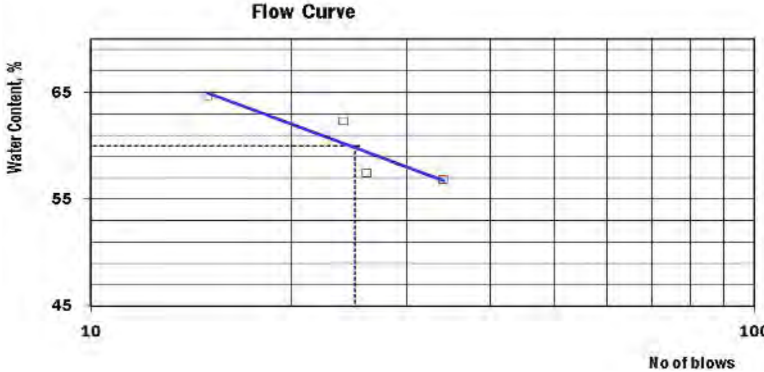

Figure B -4.4 (a) Atterberg Limits for Addis Ketema at 1.5m

Table B - 4.4 (b) Atterberg Limits for Addis Ketema (Worede - 10) at 2.6m

Location: Addis Ketema(Woreda_10)

Date: 26-06-2011

Depth, m: 2.60

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	E1	E3	22	54	C18	D5
Mass of container, g	15.63	15.83	15.55	15.88	13.62	15.86
Mass of container + Wet soil, g	44.31	41.11	46.45	46.79	17.58	20.34
Mass of container + Dry soil, g	33.27	31.21	33.96	34.36	16.38	18.98
Mass of water, g	11.04	9.90	12.49	12.43	1.20	1.36
Mass of dry soil, g	17.64	15.39	18.41	18.48	2.76	3.12
Water content, %	62.58	64.34	67.87	67.25	43.32	43.59
No of blows	28	23	18	16	------	------

Liquid Limit, % = 63.8; Plastic Limit, % = 43.5; PI, %= 20

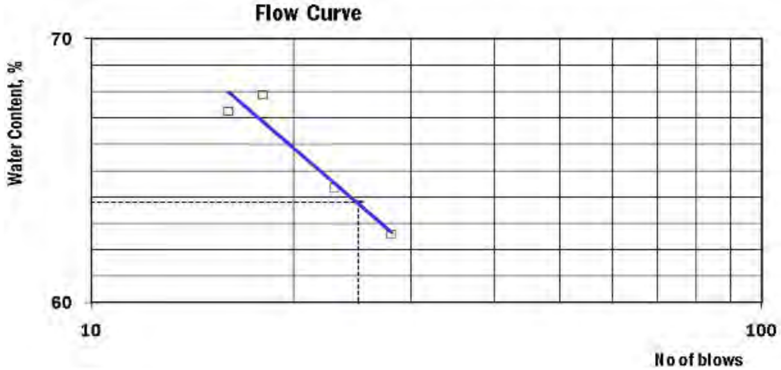

Figure B -4.4 (b) Atterberg Limits for Addis Ketema at 2.6m

Table B - 4.5 Atterberg Limits for Meskel Flower [MF1] at 2.60m

Location: Kirkos S.city _Wor. 3
Date: 02-07-2011
Depth, m: 2.6

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	D15	D31	1A	A31	66	45
Mass of container, g	15.62	15.47	15.29	15.80	15.64	15.67
Mass of container + Wet soil, g	29.14	27.22	26.40	31.85	25.43	25.70
Mass of container + Dry soil, g	22.12	21.06	20.68	23.27	22.59	22.62
Mass of water, g	7.02	6.15	5.72	8.58	2.84	3.08
Mass of dry soil, g	6.50	5.59	5.39	7.47	6.95	6.95
Water content, %	108.06	109.99	106.12	114.87	40.94	44.23
No of blows	30	26	32	16	------	------

Liquid Limit, % = 109.8; Plastic Limit, % = 42.6; PI, %= 67

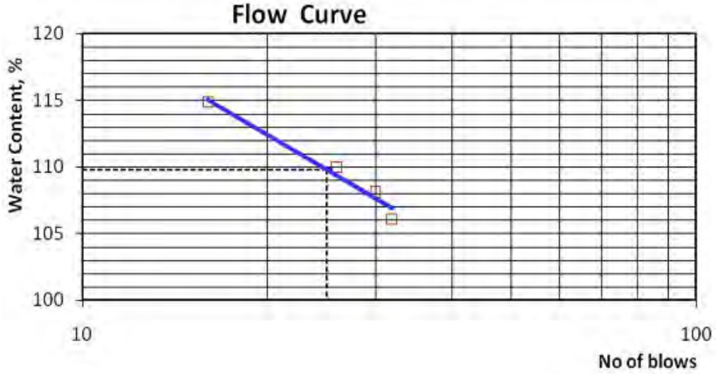

Figure B -4.5Atterberg Limits for Meskel Flower [MF1] at 2.60m

B – 5) Free Swell Determination

Table B - 5.3 Free Swell for Meskel Flower [MF_1]

Location: KirkosS.city _Wor. 3

Date: 02-07-2011

Depth, m: 2.6

Initial Volume (cc)	Final Volume One Sample (cc)	Average Final Volume (cc)	Free Swell Index (%)
10.0	35.0	35.0	250

B – 6) Particle Size Determination

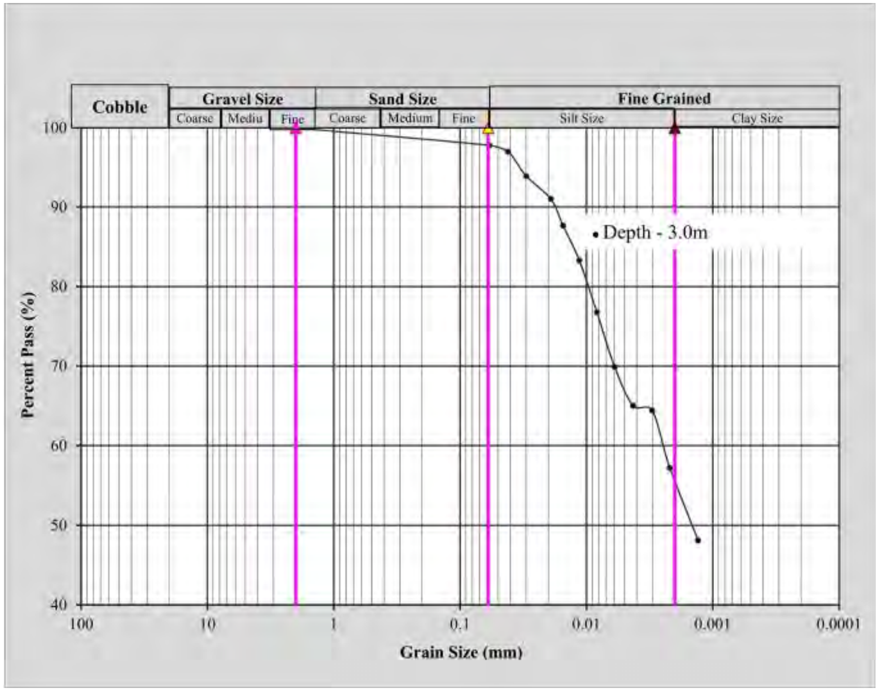

Figure B -6.1Particle Size for Imperial Hotel [I1]

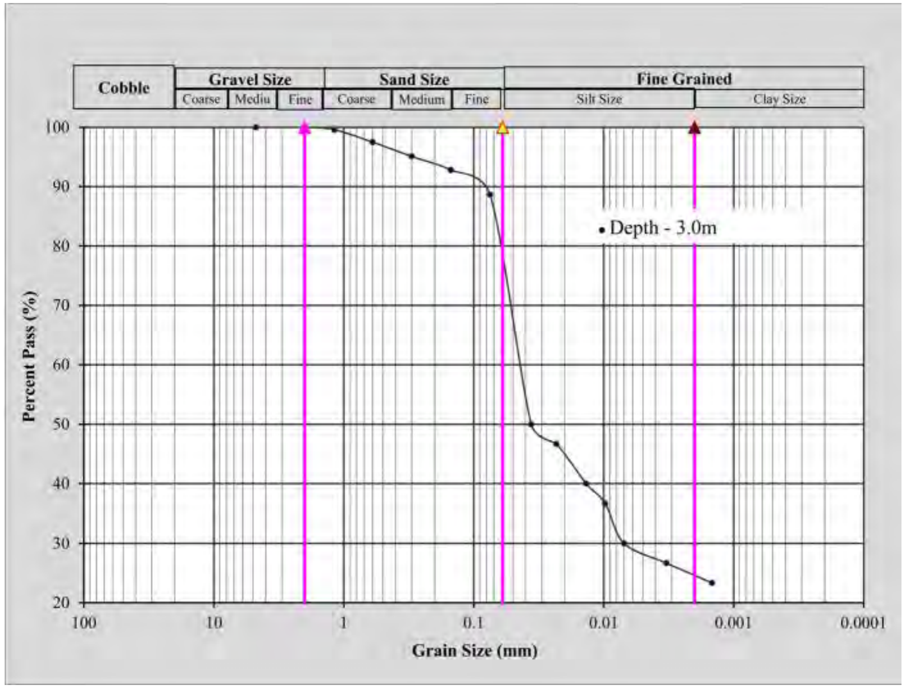

Figure B -6.2 Particle Size for Lebu [L1]

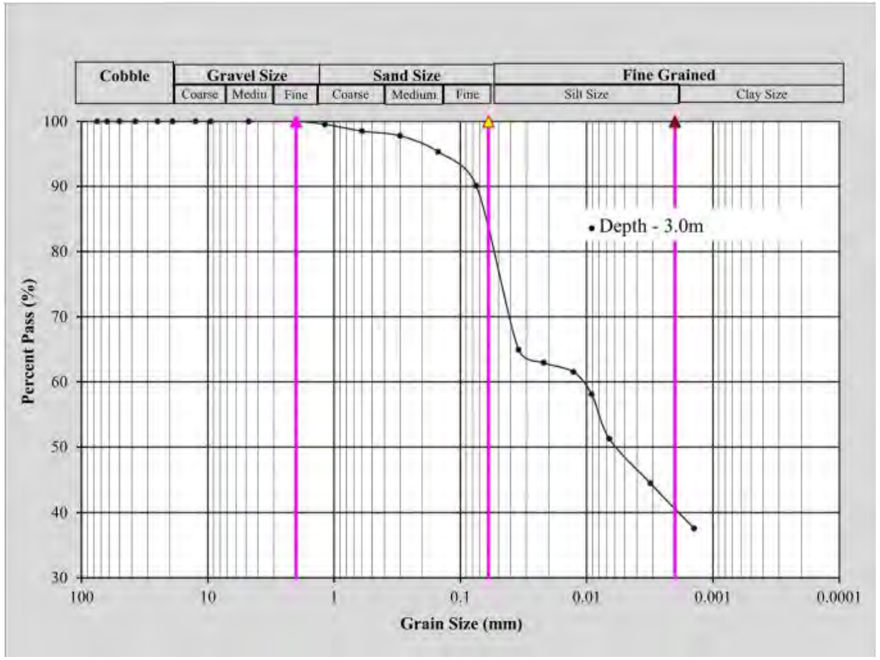

Figure B -6.3 Particle Size for Lebu [L_2]

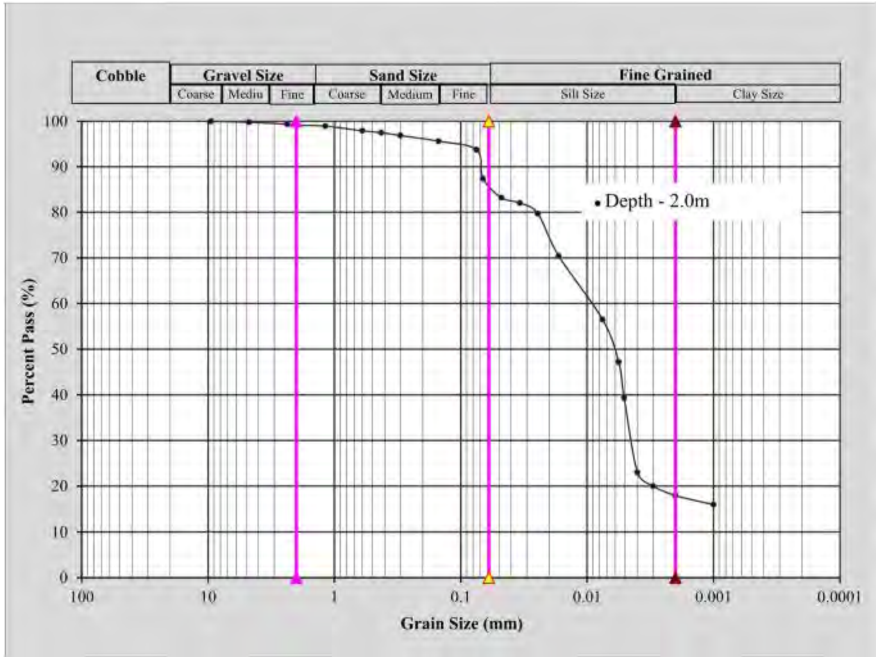

Figure B -6.4 Particle Size for Addisu Gebeya [AG]

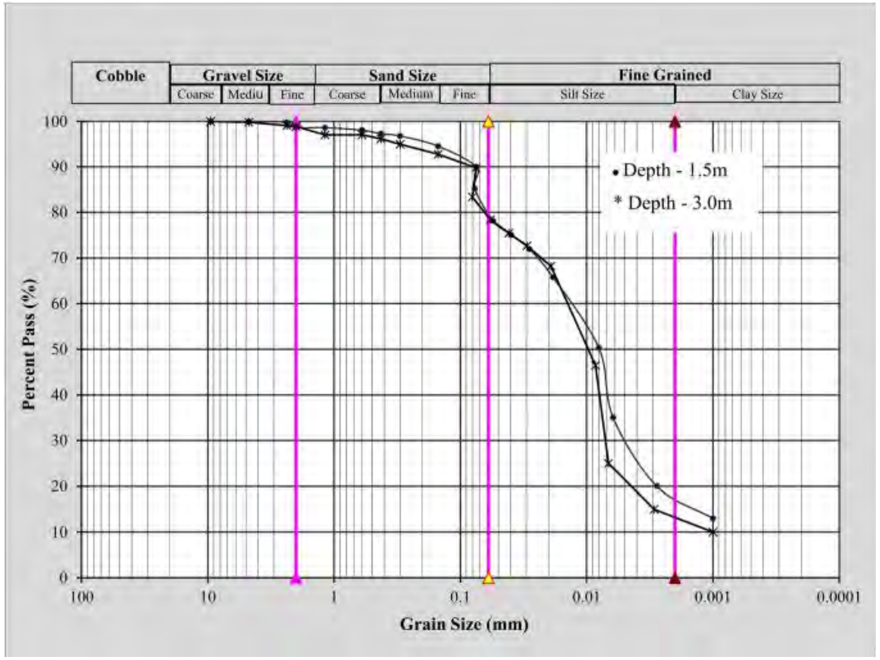

Figure B -6.5 Particle Size for Atari [AT]

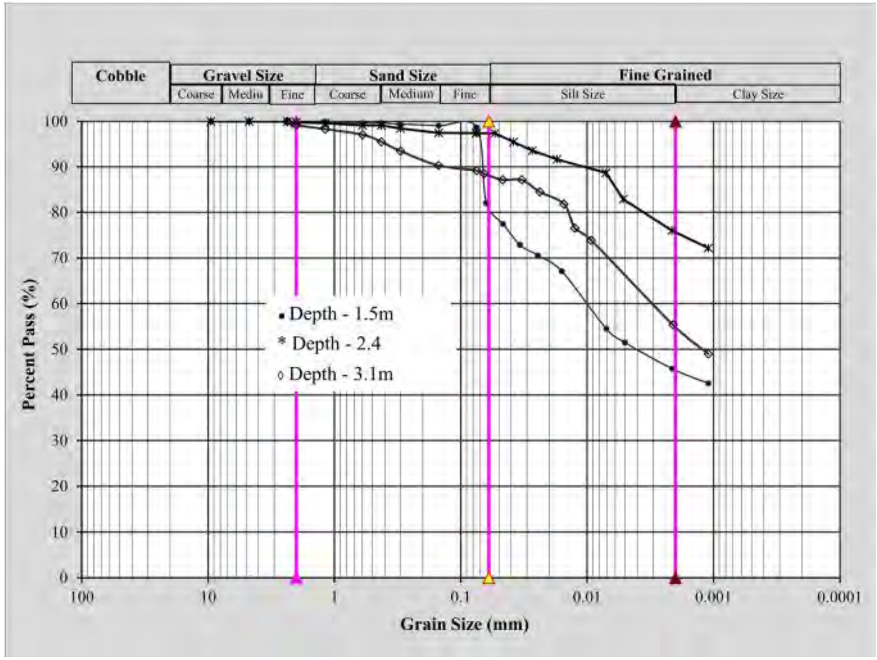

Figure B -6.6 Particle Size for Aweliya [AW]

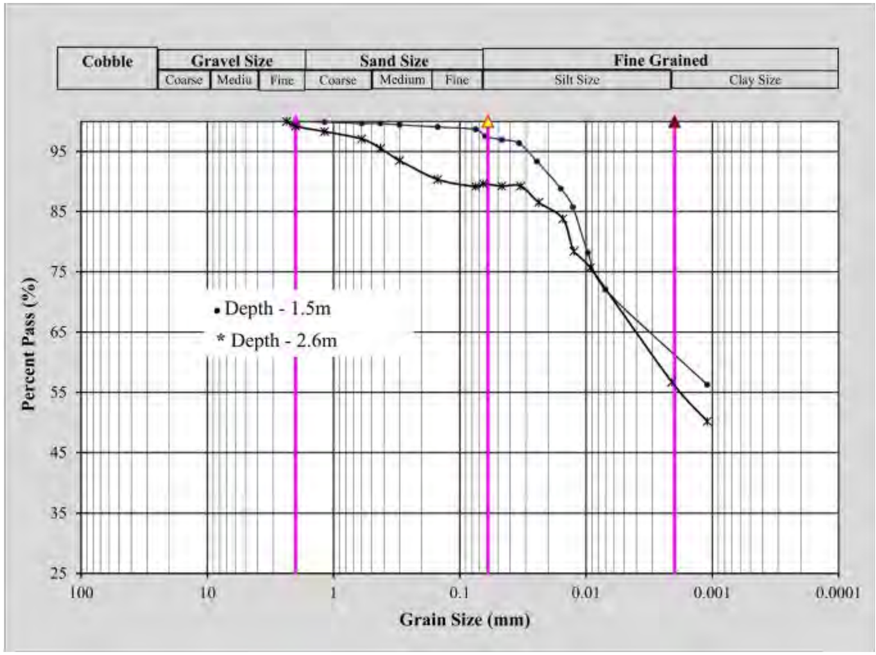

Figure B -6.7 Particle Size for Addis Ketema [AK]

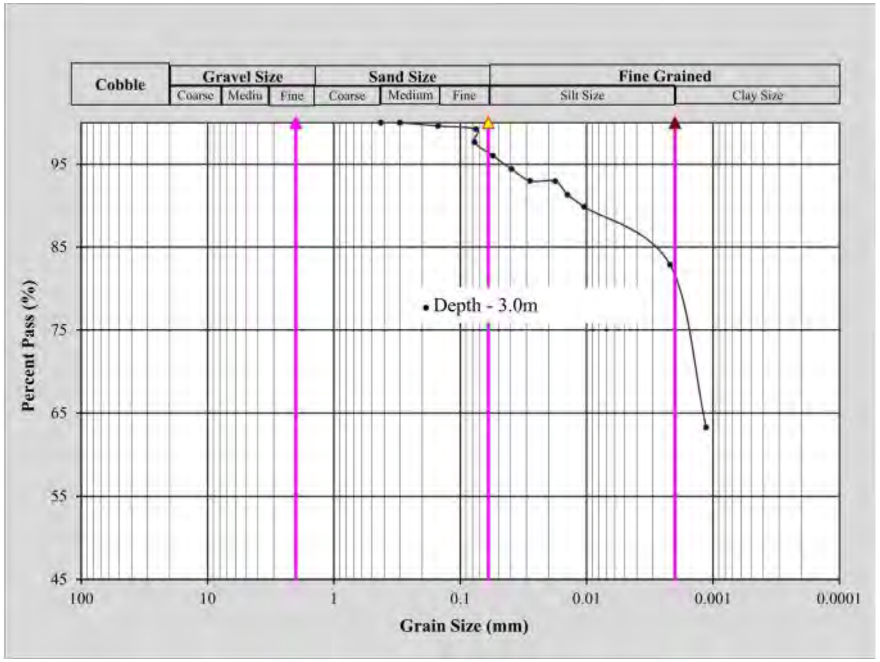

Figure B -6.8 Particle Size for Meskel Flower [MF]

B – 7) Unconfined Compression Strength Determination

Table B - 7.1 Unconfined Compression Strength for Addisu Gebeya [AG]

Test Pit No:	Addisu Gebeya	Cross- Sectional Area , m^2	0.001134
Depth, m:	2.00	Ring Calibration Factor, kN/div	0.00138
Sample:	Undisturbed	Moisture content, %	26.20
Diameter of sample , mm	38	Wet unit weight, kN/m^3	18.32
Length of sample , mm	76	Dry Unit Weight, kN/m^3	15.44
		Rate of Strain, mm/min	1.70

Axial Deformation [mm]	Axial Strain [%]	Proving Ring Reading [div]	Axial Load [kN]	Corrected Area [m^2]	Axial Stress [kPa]
0	0.00	0	0.0000	0.001134	0
0.2	0.26	8	0.0110	0.001137	9.71
0.4	0.53	16.5	0.0228	0.001140	19.97
0.6	0.79	26	0.0359	0.001143	31.39
0.8	1.05	38.5	0.0531	0.001146	46.35
1	1.32	48.5	0.0669	0.001149	58.24
1.2	1.58	70.5	0.0973	0.001152	84.43
1.4	1.84	90.5	0.1249	0.001155	108.09
1.6	2.11	111.75	0.1542	0.001159	133.12
1.8	2.37	136	0.1877	0.001162	161.57
2	2.63	160.5	0.2215	0.001165	190.16
2.2	2.89	187.5	0.2588	0.001168	221.55
2.4	3.16	209.4	0.2890	0.001171	246.75
2.6	3.42	207.5	0.2864	0.001174	243.85
2.8	3.68	207	0.2857	0.001177	242.60
3	3.95	201.5	0.2781	0.001181	235.51

Unconfined Compressive Strength , kPa = 248.18

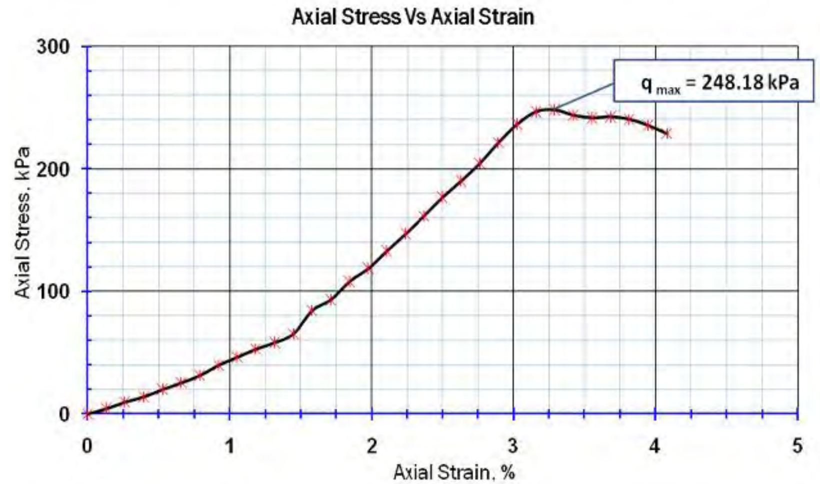

Figure B - 7.1 Unconfined Compression Strength for Addisu Gebeya [AG]

Table B - 7.2 (a) Unconfined Compression Strength for Atari [AT_1] at 1.5m

Test Pit No:	Atari	Cross- Sectional Area , m^2	0.001134
Depth, m:	1.50	Ring Calibration Factor, kN/div	0.00138
Sample:	Undisturbed	Moisture content, %	30.19
Diameter of sample , mm	38	Wet unit weight, kN/m^3	19.06
Length of sample , mm	76	Dry Unit Weight, kN/m^3	14.64
		Rate of Strain, mm/min	1.70

Axial Deformation [mm]	Axial Strain [%]	Proving Ring Reading [div]	Axial Load [kN]	Corrected Area [m^2]	Axial Stress [kPa]
0	0.00	0	0.0000	0.001134	0
0.2	0.26	12	0.0166	0.001137	14.56
0.4	0.53	23	0.0317	0.001140	27.84
0.6	0.79	34.5	0.0476	0.001143	41.65
0.8	1.05	54.5	0.0752	0.001146	65.62
1	1.32	76.5	0.1056	0.001149	91.86
1.2	1.58	93	0.1283	0.001152	111.38
1.4	1.84	121	0.1670	0.001155	144.52
1.6	2.11	140	0.1932	0.001159	166.77
1.8	2.37	148.5	0.2049	0.001162	176.42
2	2.63	153	0.2111	0.001165	181.27
2.2	2.89	145	0.2001	0.001168	171.33
2.4	3.16	137.5	0.1898	0.001171	162.03
2.6	3.42	140	0.1932	0.001174	164.53
2.8	3.68	145	0.2001	0.001177	169.94
3	3.95	147	0.2029	0.001181	171.81

Unconfined Compressive Strength , kPa = 181.27

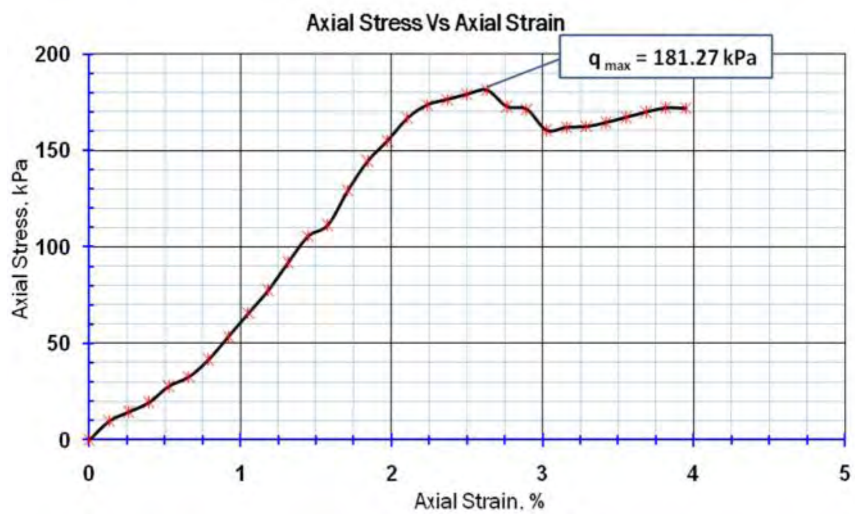

Figure B - 7.2 (a) Unconfined Compression Strength for Atari [AT_1] at 1.5m

Table B - 7.2 (b) Unconfined Compression Strength for Atari [AT_2] at 3.0m

Test Pit No:	Atari	Cross- Sectional Area , m^2	0.001134
Depth, m:	3.00	Ring Calibration Factor, kN/div	0.00138
Sampling	Undisturbed	Moisture content, %	29.19
Diameter of sample , mm	38	Wet unit weight, kN/m^3	18.81
Length of sample , mm	76	Dry Unit Weight, kN/m^3	14.56
		Rate of Strain, mm/min	1.70

Axial Deformation [mm]	Axial Strain [%]	Proving Ring Reading [div]	Axial Load [kN]	Corrected Area [m^2]	Axial Stress [kPa]
0	0.00	0	0.0000	0.001134	0
0.2	0.26	11.5	0.0159	0.001137	13.96
0.4	0.53	20.5	0.0283	0.001140	24.81
0.6	0.79	30	0.0414	0.001143	36.22
0.8	1.05	38	0.0524	0.001146	45.75
1	1.32	54	0.0745	0.001149	64.84
1.2	1.58	64	0.0883	0.001152	76.65
1.4	1.84	70	0.0966	0.001155	83.61
1.6	2.11	72	0.0994	0.001159	85.77
1.8	2.37	78.5	0.1083	0.001162	93.26
2	2.63	85	0.1173	0.001165	100.71
2.2	2.89	90	0.1242	0.001168	106.34
2.4	3.16	90.6	0.1250	0.001171	106.76
2.6	3.42	89	0.1228	0.001174	104.59
2.8	3.68	75	0.1035	0.001177	87.90
3	3.95	65	0.0897	0.001181	75.97
3.2	4.21	58	0.0800	0.001184	67.60
3.4	4.47	51	0.0704	0.001187	59.28

Unconfined Compressive Strength , kPa = 108.26

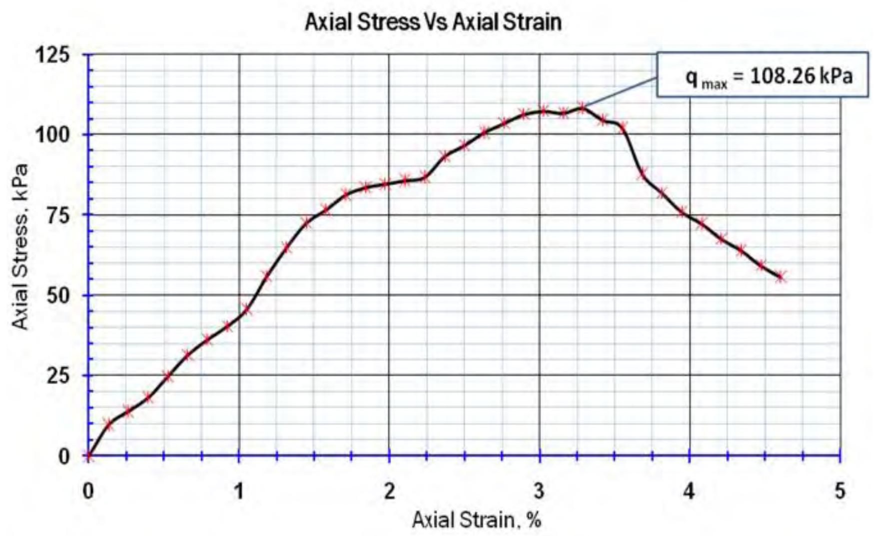

Figure B - 7.2 (b) Unconfined Compression Strength for Atari [AT_2] at 3.0m

Table B - 7.3 (a) Unconfined Compression Strength for Aweliya [AW_1] at 1.5m

Test Pit No:	Aweliya	Cross- Sectional Area , m^2	0.001134
Depth, m:	1.50	Ring Calibration Factor, kN/div	0.00138
Sample:	Undisturbed	Moisture content, %	27.00
Diameter of sample , mm	38	Wet unit weight, kN/m^3	17.63
Length of sample , mm	76	Dry Unit Weight, kN/m^3	13.88
		Rate of Strain, mm/min	1.70

Axial Deformation [mm]	Axial Strain [%]	Proving Ring Reading [div]	Axial Load [kN]	Corrected Area [m^2]	Axial Stress [kPa]
0	0.00	1	0.0014	0.001134	1.21680787
0.2	0.26	3	0.0041	0.001137	3.64
0.4	0.53	5.5	0.0076	0.001140	6.66
0.6	0.79	8	0.0110	0.001143	9.66
0.8	1.05	12	0.0166	0.001146	14.45
1	1.32	18	0.0248	0.001149	21.61
1.2	1.58	25	0.0345	0.001152	29.94
1.4	1.84	36	0.0497	0.001155	43.00
1.6	2.11	48	0.0662	0.001159	57.18
1.8	2.37	62	0.0856	0.001162	73.66
2	2.63	78	0.1076	0.001165	92.41
2.2	2.89	83	0.1145	0.001168	98.07
2.4	3.16	85	0.1173	0.001171	100.16
2.6	3.42	95	0.1311	0.001174	111.64
2.8	3.68	105	0.1449	0.001177	123.06
3	3.95	110	0.1518	0.001181	128.57
3.2	4.21	108	0.1490	0.001184	125.88
3.4	4.47	105	0.1449	0.001187	122.05
3.6	4.74	98	0.1352	0.001191	113.60
3.8	5.00	90	0.1242	0.001194	104.04
4	5.26	85	0.1173	0.001197	97.99
4.2	5.53	80	0.1104	0.001200	91.97
4.4	5.79	73	0.1007	0.001204	83.68
4.6	6.05	70	0.0966	0.001207	80.02
4.8	6.32	66	0.0911	0.001211	75.24
5	6.58	62	0.0856	0.001214	70.48
5.2	6.84	59	0.0814	0.001217	66.88
5.4	7.11	55	0.0759	0.001221	62.17
5.6	7.37	53	0.0731	0.001224	59.74
5.8	7.63	51	0.0704	0.001228	57.32

Unconfined Compressive Strength , kPa = 128.74

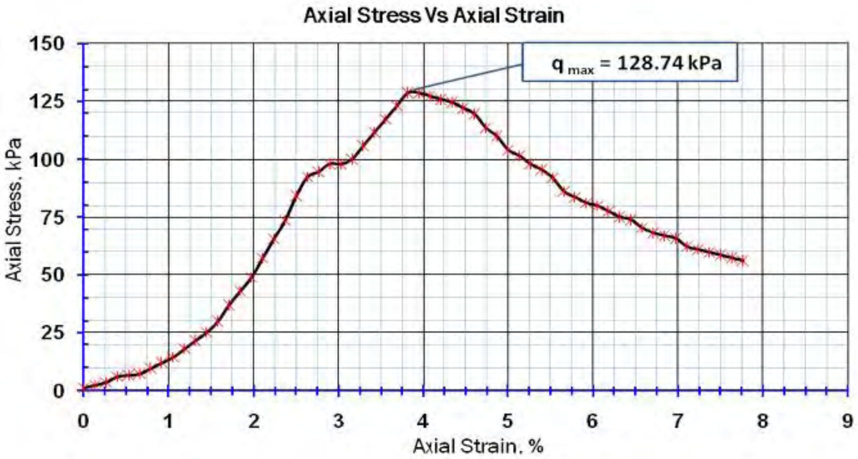

Figure B - 7.3 (a) Unconfined Compression Strength for Aweliya [AW_1] at 1.5m

Table B - 7.3 (b) Unconfined Compression Strength for Aweliya [AW_2] at 2.4m

Test Pit No:	Aweliya	Cross- Sectional Area , m^2	0.001134
Depth, m:	2.40	Ring Calibration Factor, kN/div	0.00138
Sample:	Undisturbed	Moisture content, %	31.59
Diameter of sample, mm	38	Wet unit weight, kN/m^3	17.97
Length of sample , mm	76	Dry Unit Weight, kN/m^3	13.66
		Rate of Strain, mm/min	1.70

Axial Deformation [mm]	Axial Strain [%]	Proving Ring Reading [div]	Axial Load [kN]	Corrected Area [m^2]	Axial Stress [kPa]
0	0.00	1	0.0014	0.001134	1.21681
0.2	0.26	5	0.0069	0.001137	6.07
0.4	0.53	20	0.0276	0.001140	24.21
0.6	0.79	40	0.0552	0.001143	48.29
0.8	1.05	60	0.0828	0.001146	72.24
1	1.32	85	0.1173	0.001149	102.07
1.2	1.58	110	0.1518	0.001152	131.74
1.4	1.84	133	0.1835	0.001155	158.85
1.6	2.11	155	0.2139	0.001159	184.63
1.8	2.37	172	0.2374	0.001162	204.33
2	2.63	185	0.2553	0.001165	219.19
2.2	2.89	192	0.2650	0.001168	226.86
2.4	3.16	180	0.2484	0.001171	212.11
2.6	3.42	170	0.2346	0.001174	199.78
2.8	3.68	150	0.2070	0.001177	175.80
3	3.95	142	0.1960	0.001181	165.97
3.2	4.21	132	0.1822	0.001184	153.86
3.4	4.47	118	0.1628	0.001187	137.16
3.6	4.74	110	0.1518	0.001191	127.51
3.8	5.00	80	0.1104	0.001194	92.48

Unconfined Compressive Strength , kPa = 226.86

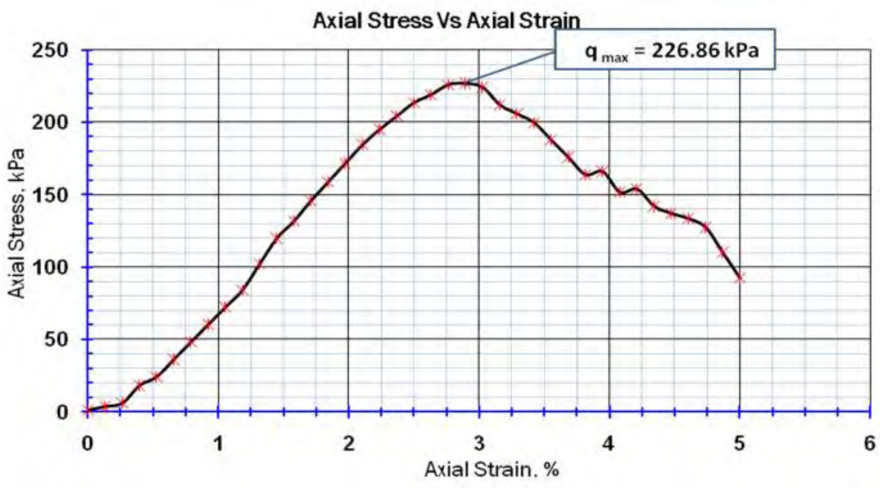

Figure B - 7.3 (b) Unconfined Compression Strength for Aweliya [AW_2] at 2.4m

Table B - 7.3 (c) Unconfined Compression Strength for Aweliya [AW3] at 2.5m

Test Pit No:	Aweliya	Cross- Sectional Area , m^2	0.001134
Depth, m:	2.50	Ring Calibration Factor, kN/div	0.00138
Sampling	Undisturbed	Moisture content, %	35.17
Diameter of sample, mm	38	Wet unit weight, kN/m^3	17.89
Length of sample, mm	76	Dry Unit Weight, kN/m^3	13.23
		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	0.00	5	0.0069	0.001134	6.0840393
0.2	0.26	18	0.0248	0.001137	21.84
0.4	0.53	35	0.0483	0.001140	42.36
0.6	0.79	52	0.0718	0.001143	62.77
0.8	1.05	75	0.1035	0.001146	90.30
1	1.32	91	0.1256	0.001149	109.27
1.2	1.58	120	0.1656	0.001152	143.71
1.4	1.84	145	0.2001	0.001155	173.19
1.6	2.11	165	0.2277	0.001159	196.55
1.8	2.37	188	0.2594	0.001162	223.34
2	2.63	202	0.2788	0.001165	239.33
2.2	2.89	213	0.2939	0.001168	251.68
2.4	3.16	218	0.3008	0.001171	256.89
2.6	3.42	205	0.2829	0.001174	240.91
2.8	3.68	170	0.2346	0.001177	199.24
3	3.95	150	0.2070	0.001181	175.32
3.2	4.21	120	0.1656	0.001184	139.87
3.4	4.47	100	0.1380	0.001187	116.24
3.6	4.74	85	0.1173	0.001191	98.53
3.8	5.00	70	0.0966	0.001194	80.92
4	5.26	50	0.0690	0.001197	57.64
4.2	5.53	30	0.0414	0.001200	34.49

Unconfined Compressive Strength , kPa = 257.24

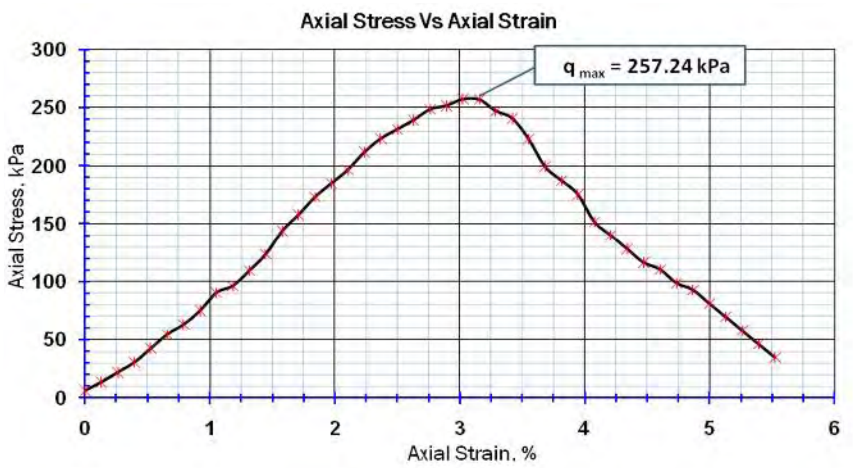

Figure B-7.3 (c) Unconfined Compression Strength for Aweliya [AW3] at 2.5m

Table B - 7.3 (d) Unconfined Compression Strength for Aweliya [AW_4] at 3.0m

Test Pit No:	Aweliya	Cross- Sectional Area , m^2	0.001134
Depth, m:	3.10	Ring Calibration Factor, kN/div	0.00138
Sample:	Undisturbed	Moisture content, %	41.08
Diameter of sample, mm	38	Wet unit weight, kN/m^3	16.98
Length of sample, mm	76	Dry Unit Weight, kN/m^3	12.04
		Rate of Strain, mm/min	1.70

Axial Deformation [mm]	Axial Strain [%]	Proving Ring Reading [div]	Axial Load [kN]	Corrected Area [m^2]	Axial Stress [kPa]
0	0.00	0.7	0.0010	0.001134	0.85176
0.2	0.26	3	0.0041	0.001137	3.64
0.4	0.53	5.5	0.0076	0.001140	6.66
0.6	0.79	9	0.0124	0.001143	10.86
0.8	1.05	14	0.0193	0.001146	16.86
1	1.32	21	0.0290	0.001149	25.22
1.2	1.58	30	0.0414	0.001152	35.93
1.4	1.84	40	0.0552	0.001155	47.78
1.6	2.11	54	0.0745	0.001159	64.32
1.8	2.37	68	0.0938	0.001162	80.78
2	2.63	81	0.1118	0.001165	95.97
2.2	2.89	92	0.1270	0.001168	108.71
2.4	3.16	102	0.1408	0.001171	120.20
2.6	3.42	110	0.1518	0.001174	129.27
2.8	3.68	115	0.1587	0.001177	134.78
3	3.95	120	0.1656	0.001181	140.25
3.2	4.21	122	0.1684	0.001184	142.20
3.4	4.47	125.5	0.1732	0.001187	145.88
3.6	4.74	126	0.1739	0.001191	146.06
3.8	5.00	125	0.1725	0.001194	144.50
4	5.26	118	0.1628	0.001197	136.03
4.2	5.53	110	0.1518	0.001200	126.45
4.4	5.79	95	0.1311	0.001204	108.90

Unconfined Compressive Strength , kPa = 146.06

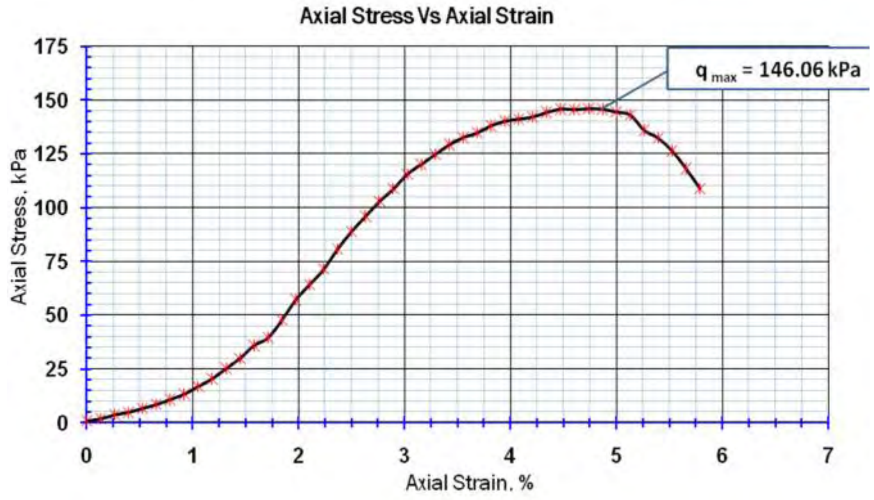

Figure B - 7.3 (d) Unconfined Compression Strength for Aweliya [AW_4] at 3.0m

Table B - 7.4 (a) Unconfined Compression Strength for Addis Ketema [AK1] at 0.25m

Test Pit No:	Addis Ketema	Cross- Sectional Area, m^2	0.001134
Depth, m:	1.65	Ring Calibration Factor, kN/div	0.00138
Sampling	Undisturbed	Moisture content, %	32.83
Diameter of sample, mm	38	Wet unit weight, kN/m^3	18.25
Length of sample , mm	76	Dry Unit Weight, kN/m^3	13.74
		Rate of Strain, mm/min	1.70

Axial Deformation [mm]	Axial Strain [%]	Proving Ring Reading [div]	Axial Load [kN]	Corrected Area [m^2]	Axial Stress [kPa]
0	0.00	0	0.0000	0.001134	0
0.2	0.26	19	0.0262	0.001137	23.06
0.4	0.53	49	0.0676	0.001140	59.31
0.6	0.79	92	0.1270	0.001143	111.06
0.8	1.05	152	0.2098	0.001146	183.01
1	1.32	211	0.2912	0.001149	253.37
1.2	1.58	259	0.3574	0.001152	310.18
1.4	1.84	284	0.3919	0.001155	339.21
1.6	2.11	286	0.3947	0.001159	340.68
1.8	2.37	262	0.3616	0.001162	311.25
2	2.63	259	0.3574	0.001165	306.86
2.2	2.89	259.5	0.3581	0.001168	306.62
2.4	3.16	261.5	0.3609	0.001171	308.15
2.6	3.42	260.5	0.3595	0.001174	306.13
2.8	3.68	257	0.3547	0.001177	301.20
3	3.95	244	0.3367	0.001181	285.18
3.2	4.21	216	0.2981	0.001184	251.76
3.4	4.47	194	0.2677	0.001187	225.50

Unconfined Compressive Strength , kPa = 341.14

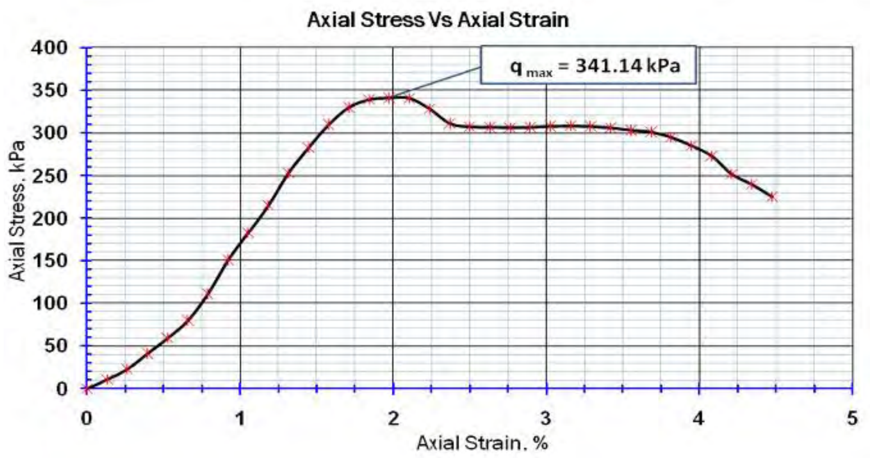

Figure B - 7.4 (a) Unconfined Compression Strength for Addis Ketema [AK1] at 0.25m

Table B - 7.4 (b) Unconfined Compression Strength for Addis Ketema [AK2] at 0.35m

Test Pit No:	Addis Ketema	Cross- Sectional Area, m^2	0.001134
Depth, m :	1.55	Ring Calibration Factor, kN/div	0.00138
Sampling	Undisturbed	Moisture content, %	32.53
Diameter of sample, mm	38	Wet unit weight, kN/m^3	18.75
Length of sample , mm	76	Dry Unit Weight, kN/m^3	14.15
		Rate of Strain, mm/min	1.70

Axial Deformation [mm]	Axial Strain [%]	Proving Ring Reading [div]	Axial Load [kN]	Corrected Area [m^2]	Axial Stress [kPa]
0	0.00	0	0.0000	0.001134	0
0.2	0.26	10	0.0138	0.001137	12.14
0.4	0.53	21	0.0290	0.001140	25.42
0.6	0.79	23	0.0317	0.001143	27.77
0.8	1.05	31	0.0428	0.001146	37.32
1	1.32	38	0.0524	0.001149	45.63
1.2	1.58	55	0.0759	0.001152	65.87
1.4	1.84	66	0.0911	0.001155	78.83
1.6	2.11	82	0.1132	0.001159	97.68
1.8	2.37	106	0.1463	0.001162	125.93
2	2.63	134	0.1849	0.001165	158.76
2.2	2.89	161	0.2222	0.001168	190.24
2.4	3.16	182	0.2512	0.001171	214.47
2.6	3.42	204	0.2815	0.001174	239.74
2.8	3.68	221	0.3050	0.001177	259.01
3	3.95	237	0.3271	0.001181	277.00
3.2	4.21	251	0.3464	0.001184	292.56
3.4	4.47	276	0.3809	0.001187	320.81
3.6	4.74	291	0.4016	0.001191	337.32
3.8	5.00	295	0.4071	0.001194	341.01
4	5.26	295	0.4071	0.001197	340.07
4.2	5.53	288	0.3974	0.001200	331.07
4.4	5.79	270	0.3726	0.001204	309.52
4.6	6.05	251	0.3464	0.001207	286.93
4.8	6.32	235	0.3243	0.001211	267.89
5	6.58	218	0.3008	0.001214	247.81
5.2	6.84	208	0.2870	0.001217	235.78

Unconfined Compressive Strength , kPa = 341.01

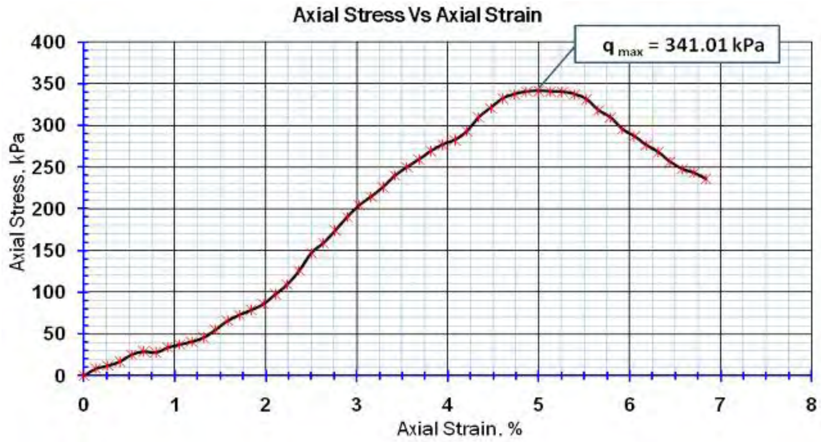

Figure B - 7.4 (b) Unconfined Compression Strength for Addis Ketema [AK2] at 0.35m

Table B - 7.4 (c) Unconfined Compression Strength for Addis Ketema [AK3] at 1.55m

Test Pit No:	Addis Ketema	Cross- Sectional Area , m^2	0.001134
Depth, m:	0.35	Ring Calibration Factor, kN/div	0.00138
Sampling	Undisturbed	Moisture content, %	21.80
Diameter of sample, mm	38	Wet unit weight, kN/m^3	15.11
Length of sample , mm	76	Dry Unit Weight, kN/m^3	12.40
		Rate of Strain, mm/min	1.70

Axial Deformation [mm]	Axial Strain [%]	Proving Ring Reading [div]	Axial Load [kN]	Corrected Area [m^2]	Axial Stress [kPa]
0	0.00	0	0.0000	0.001134	0
0.2	0.26	5	0.0069	0.001137	6.07
0.4	0.53	14	0.0193	0.001140	16.95
0.6	0.79	26	0.0359	0.001143	31.39
0.8	1.05	36	0.0497	0.001146	43.34
1	1.32	51	0.0704	0.001149	61.24
1.2	1.58	70	0.0966	0.001152	83.83
1.4	1.84	95	0.1311	0.001155	113.47
1.6	2.11	100	0.1380	0.001159	119.12
1.8	2.37	90	0.1242	0.001162	106.92
2	2.63	107	0.1477	0.001165	126.77
2.2	2.89	132	0.1822	0.001168	155.97
2.4	3.16	147	0.2029	0.001171	173.22
2.6	3.42	154	0.2125	0.001174	180.98
2.8	3.68	166	0.2291	0.001177	194.55
3	3.95	184	0.2539	0.001181	215.05
3.2	4.21	191	0.2636	0.001184	222.62
3.4	4.47	169	0.2332	0.001187	196.44
3.6	4.74	179	0.2470	0.001191	207.49
3.8	5.00	183	0.2525	0.001194	211.54
4	5.26	182	0.2512	0.001197	209.80
4.2	5.53	175	0.2415	0.001200	201.17
4.4	5.79	169	0.2332	0.001204	193.74
4.6	6.05	152	0.2098	0.001207	173.76
4.8	6.32	131	0.1808	0.001211	149.33
5	6.58	125	0.1725	0.001214	142.09

Unconfined Compressive Strength , kPa = 222.93

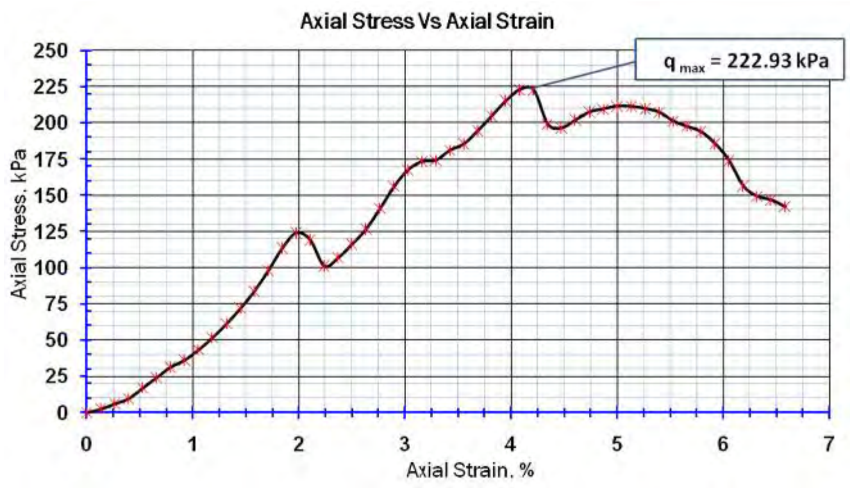

Figure B - 7.4 (c) Unconfined Compression Strength for Addis Ketema [AK3] at 1.55m

Table B - 7.4 (d) Unconfined Compression Strength for Addis Ketema[AK4] at 1.65m

Test Pit No:	Addis Ketema	Cross- Sectional Area , m^2	0.001134
Depth, m:	0.25	Ring Calibration Factor, kN/div	0.00138
Sampling	Undisturbed	Moisture content, %	20.96
Diameter of sample , mm	38	Wet unit weight, kN/m^3	15.09
Length of sample, mm	76	Dry Unit Weight, kN/m^3	12.48
		Rate of Strain, mm/min	1.70

Axial Deformation [mm]	Axial Strain [%]	Proving Ring Reading [div]	Axial Load [kN]	Corrected Area [m^2]	Axial Stress [kPa]
0	0.00	0	0.0000	0.001134	0
0.2	0.26	14	0.0193	0.001137	16.99
0.4	0.53	26	0.0359	0.001140	31.47
0.6	0.79	38	0.0524	0.001143	45.87
0.8	1.05	57	0.0787	0.001146	68.63
1	1.32	80	0.1104	0.001149	96.06
1.2	1.58	96	0.1325	0.001152	114.97
1.4	1.84	119	0.1642	0.001155	142.13
1.6	2.11	146	0.2015	0.001159	173.91
1.8	2.37	159	0.2194	0.001162	188.89
2	2.63	169	0.2332	0.001165	200.23
2.2	2.89	154	0.2125	0.001168	181.96
2.4	3.16	115	0.1587	0.001171	135.51
2.6	3.42	82	0.1132	0.001174	96.36
2.8	3.68	72	0.0994	0.001177	84.38
3	3.95	70	0.0966	0.001181	81.81
3.2	4.21	75	0.1035	0.001184	87.42
3.4	4.47	70	0.0966	0.001187	81.37

Unconfined Compressive Strength , kPa = 200.23

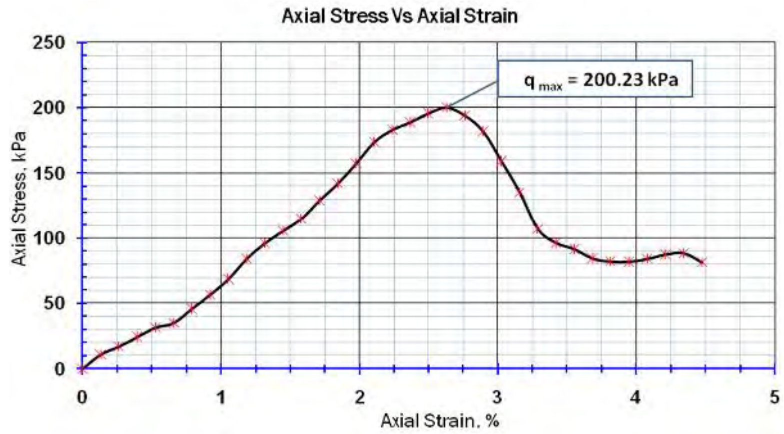

Figure B-7.4 (d) Unconfined Compression Strength for Addis Ketema[AK4] at 1.65m

Table B - 7.4 (e) Unconfined Compression Strength for Addis Ketema[AK5] at 2.6m

Test Pit No:	Addis Ketema	Cross- Sectional Area, m^2	0.001134
Depth, m:	2.60	Ring Calibration Factor, kN/div	0.00138
Sampling	Undisturbed	Moisture content, %	39.73
Diameter of sample, mm	38	Wet unit weight, kN/m^3	17.47
Length of sample, mm	76	Dry Unit Weight, kN/m^3	12.50
		Rate of Strain, mm/min	1.70

Axial Deformation [mm]	Axial Strain [%]	Proving Ring Reading [div]	Axial Load [kN]	Corrected Area [m^2]	Axial Stress [kPa]
0	0.00	0	0.0000	0.001134	0
0.2	0.26	5	0.0069	0.001137	6.07
0.4	0.53	6	0.0083	0.001140	7.26
0.6	0.79	7	0.0097	0.001143	8.45
0.8	1.05	10	0.0138	0.001146	12.04
1	1.32	14.5	0.0200	0.001149	17.41
1.2	1.58	19	0.0262	0.001152	22.75
1.4	1.84	26	0.0359	0.001155	31.05
1.6	2.11	34	0.0469	0.001159	40.50
1.8	2.37	41.5	0.0573	0.001162	49.30
2	2.63	51.5	0.0711	0.001165	61.02
2.2	2.89	62	0.0856	0.001168	73.26
2.4	3.16	73	0.1007	0.001171	86.02
2.6	3.42	82	0.1132	0.001174	96.36
2.8	3.68	92	0.1270	0.001177	107.82
3	3.95	96	0.1325	0.001181	112.20
3.2	4.21	108	0.1490	0.001184	125.88
3.4	4.47	115	0.1587	0.001187	133.67
3.6	4.74	118.5	0.1635	0.001191	137.36
3.8	5.00	121	0.1670	0.001194	139.87
4	5.26	122.5	0.1691	0.001197	141.21
4.2	5.53	121	0.1670	0.001200	139.10
4.4	5.79	118	0.1628	0.001204	135.27
4.6	6.05	114.5	0.1580	0.001207	130.89
4.8	6.32	112.5	0.1553	0.001211	128.25
5	6.58	105	0.1449	0.001214	119.36
5.2	6.84	101	0.1394	0.001217	114.49
5.4	7.11	96	0.1325	0.001221	108.51
5.6	7.37	90	0.1242	0.001224	101.44

Unconfined Compressive Strength , kPa = 141.21

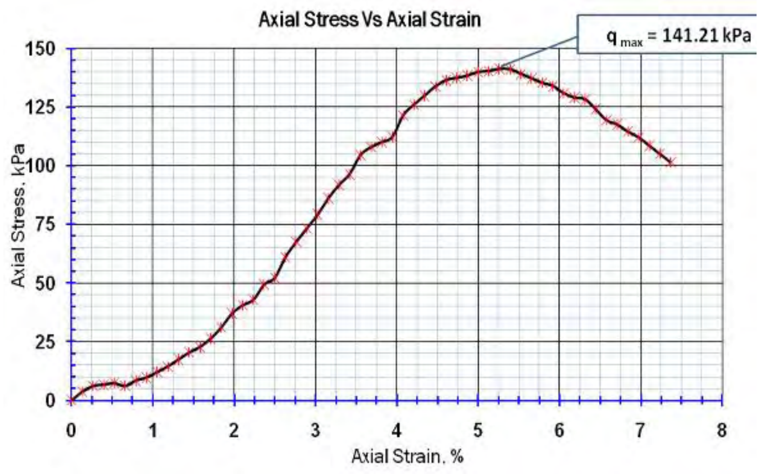

Figure B - 7.4 (e) Unconfined Compression Strength for Addis Ketema [AK5] at 2.6m

APPENDIX – C: General Overview of Work done

Addisu Gebeya (A.A.)_ DCP Test

Imperial Hotel (A.A.)_ DCP Test

Atari (A.A.)_ Hydrometer

Addisu Gebeya (A.A.) _UCS

Wodito (Gurage) _ DCP Test

Layimatheisla (Gamogofa)
_ DCP Test

DECLARATION

I, the undersigned, declare that this thesis is my original work performed under the supervision of my research advisor Professor Alemayehu Teferra 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.

Name: Anteneh Getachew Kefyalew

Signature 

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Date: November, 2012