JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

GEOTECHINICAL ENGINEERING

Correlation of Dynamic cone penetration with undraind shear strength of clay soils: A case study of Jimma town.

A thesis submitted to the School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Degree of Master of Science in Geotechnical Engineering

By: -Hana seifu

January, 2018

Jimma, Ethiopia

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Advisors: Main Advisor: Yoseph Birru (PhD)

Co Advisor: - Mr. Alemneh Soresa, (PhD student)

January.2018

Jimma, Ethiopia

DECLARATION

This research is my original work and has not been presented for a degree in any other university.

Hana seifu

Signature_________________________ Date_____________________

This research has been submitted for examination with my approval as university supervisor.

Advisors

Main advisor: - Yoseph Birru (PhD)

Signature_______________________ Date_____________________

Co Advisor: Alemneh Soresa, (PhD student)

Signature_______________________ Date_____________________

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Table of Contents

Lists of Symbols and Abbreviation

ASTM	American Society for Testing of Materials
AASHO	American Association of State Highway and Transportation Officials
A_o	Initial area
BS	British Standard
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
E_{eff}	Elasticity Modulus
ε	Axial Strain
GPS	Global Positioning System
Gs	Specific Gravity
γ	Unit weight of soil
h	Depth of cone tip while recording
kPa	kilo Pascal
LI	Liquidity Index
LL	Liquid Limit
MH	Inorganic Silt of High Plasticity
NGL	Natural Ground Level
NMC	Natural Moisture Content
Nc	Bearing capacity factor for cohesion
Nq	Bearing capacity factor for surcharge

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

List of Tables

List of Figures

ABSTRACT

Determination of the in-situ engineering properties of foundation materials has always been a challenge for practicing engineers in developing countries due to limited resources available for investing on sophisticated field equipment which usually leads to usage of unreliable design data. 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.

The objective of the study to correlate Dynamic Cone Penetration (DCP) with undrained shear strength (Su) and bearing capacity of typical Jimma clay soil.

In this thesis, field tests were conducted by dynamic cone penetration equipment. Laboratory tests needed to classify the soil and study the parameters that affect the dynamic cone penetration were conducted and the test results are analyzed by spss statistical software and Microsoft excel to find their correlation functions.

After analyzing the data, it has been found that parameters like unconfined compression strength, dry density, and bulk density have influence on the DCP. From field investigations and laboratory test results correlation has been developed between Unconfined Compression Strength (qu) and DCP as $qu=0.3224DCP^2-20.975DCP+457.0$ *with coefficient of determination* (R^2) *of 63%, for typical clay soils in Jimma town. From this undraind shear strength is* $C = 0.1612DCP^2-10.4875DCP+228.52$. *These correlations were further used to develop bearing capacity equation based on bearing capacity theory as* $q_{ult} = 0.829DCP^2-53.927DCP+ \gamma h +1175.049$. *These correlations can be used as a starting point in characterizing geotechnical properties of typical soils in Jimma town for the development of infrastructure and related applications.*

Key words; DCP (dynamic cone penetration),qu (unconfined compression strength) and C(undrained shear strength)

CHAPTER ONE

INTRODUCTION

1.1 Background

Proper geotechnical engineering practice requires that the scope of a site investigation be made commensurate with the type of geotechnical problem on hand. For small projects especially in developing countries, simple and economical methods of site investigation are required. Therefor the need for a simple cost effective, rapid, in situ method for characterizing the sub-surface profile. The dynamic cone penetrometer (DCP), has the potential to partially fill such a need.

The design trend should be based on a more reliable mode of determining the bearing properties and performance of soils. Measuring the strength of in situ soil and the thickness and location of underlying soil layers can be accomplished using simple, hand-held device DCP Used worldwide, DCP is a simple test device that is inexpensive, portable, easy to operate, easily transportable 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. [1]

There are various types of DCPs available in the world but operated on the same principles. In the current research, a light weight DCP device is used for evaluation of the undrained shear strength of clay soils. The device consists of an 8 kg mass dropping through a height of 575 mm and a 600 cone having a base diameter of 20 mm. The penetration of the cone is measured using a calibrated scale. [2]

1.2 Statement of problem

Soil is diverse in formation and character therefore accurate prediction of its engineering behavior is of research interest in civil engineering. The engineering behavior of soils varies from place to place and also with time. Many attempts have been made to predict undrained shear strength value from the Dynamic cone penetration. Hence determining of factors that influence the Dynamic cone penetration and studying their relationship with undrained shear strength value on representative sample may be considered as good insight of soil behavior**.**

Finding undrained shear strength requires relatively large effort to conduct the test and it is time consuming. The alternate method could be to correlate undained shear strength with simpler DCP test results are much economical and rapid than unconfined compression strength test which is used as to find out undrained shear strength.

Different investigations are conducted on this correlation by different scholars in our country for instance by Anteneh Getachew on November 2012 in Addis Ababa. But the study needs detail investigations more sample size than this research in other town because if the sample size and study areas are increase the accuracy of the result increase and the tendency to use the findings will be increase.

In Jimma area, there is no research that investigated the correlation of dynamic cone penetration with undrained shear strength to the best of my knowledge. Hence, it is required to carry out investigations to develop the correlation in order to capitalize on the aforementioned benefits.

1.3 Objective of the study

1.3.1 General Objective

To correlate Dynamic Cone Penetration (DCP) with undrained shear strength (Su) of typical Jimma clay soil.

1.3.2 Specific Objectives

- To establish relations between undrained shear strength and DCP penetration per blow for clay soil.
- To develop empirical correlation equation that enables to estimate bearing capacity.
- To determine factors and parameters of soil property that affect DCP results.

1.4 Scope and limitation of the thesis

The subject study is desired to conduct a localized research particularly on samples recovered from Jimma city. In order to conduct the proposed correlation twenty-one laboratory test results are used in this research work and two laboratory test data for validation purpose are used With regard to the regression analysis, depending on the trends of the scattering of test results the correlation is analyzed using a Single regression model. The required correlation is carried out by applying a single regression model and multiple linear regression models with the aid of SPSS Software and Microsoft excel Furthermore, the scope of the developed correlation is limited to the test procedures followed in the subject research work.

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.

CHAPTER TWO

LITERATURE REVIEW

2.1 Undraind shear strength of clay soils

2.1.1 Undraind behavior of soils

If no drainage is possible from a soil, because the soil has been sealed off, or because the load is applied so quickly and the permeability is so small that there is no time for outflow of water, there will be no consolidation of the soil. This is the *undraind behavior* of a soil [3]

2.1.1.1 Undraind tests

In an undrained triaxial test on saturated clay each increase of the cell pressure will lead to an increase of the pore water pressure. This can be described by Skempton's formula [3]

$\Delta p = B[\Delta\sigma 3 + A(\Delta\sigma 1 - \Delta\sigma 3)]$………………………………………………………………… (2.1)

The coefficient B can be expected to be about

$B = 1/1 + n\beta K$

Where β is the compressibility of the pore fluid (including possible air bubbles) and K is the compression modulus of the grain skeleton. The value of the coefficient B will be close to 1, as the water is practically incompressible .Increasing the cell pressure can be expected to result in an increment of the pore pressure by the same amount as the increment of the cell pressure, or slightly less, and thus there will be very little change in the effective stresses. If there is a possibility for drainage, and there is sufficient time for the soil to drain, the pore pressures will be gradually reduced, with a simultaneous increase of the effective stresses. This is the consolidation process. If there is no possibility for drainage, because the sample has been completely sealed off, or because the test is done so quickly that there is no time for consolidation, the test is called *unconsolidated*. In the second stage of a triaxial test, in which only the vertical stress is increased, distinction can also be made in drained or undrained tests. If in this stage no drainage can take place, the test is called *unconsolidated undrained* (a UU-test). If a second UU-test is done at a higher cell pressure, the only difference with the first test will be that the pore pressures are higher. The effective stresses in both tests will be practically the same. If the test results are plotted in a Mohr diagram, there would be just one critiacl circle for the effective stresses, but in terms of total stresses there will be two clearly distinct circles, of practically the same magnitude, in this figure the critical Mohr circles for the total stresses in the two tests have been dotted. The critical circles for the effective stresses can be obtained by subtracting the pore pressure, and these are represented by full lines. The two circles practically coincide, if the sample is saturated with water. These test results are insufficient to determine the shear strength parameters C and φ, because only one critical circle for the effective stresses is available. In order to determine the values of c and φ one of the tests should be allowed to consolidate after the first loading stage, so that the isotropic effective stress at the beginning of the second stage, the vertical loading, is different in the two tests. This would mean that this test would be Consolidated Undrained test, or a CU-test. Undrained tests may be very useful, because in engineering practice there are many situations in which no (or very little) drainage will occur, or in which it can be shown that the undrained situation is the most critical anyway. For instance, in the case of a load applied to a shallow foundation slab, it can be expected that pore pressures will be developed below the foundation, and that these pore pressures will dissipate in course of time due to consolidation. If the load remains constant, it can be expected that the pore pressures are highest, and thus the effective stresses are smallest just after the application of the load. Later, after consolidation, the effective stresses will be higher, so that the Mohr circle will be shifted to the right. This means that the most critical situation occurs immediately after application of the load, in the undrained state. In order to predict the behavior of the clay in this condition it makes sense to just consider the total stresses, and to make use of the results of an undrained test, analyzing the test results in terms of total stresses also. That there may be considerable pore pressures in the test as well [3]

2.1.2 General Mohr-coulomb failure criterion

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

$s = c + \sigma \tan ø$………………………………………………………….……….……………………… (2.2)

Where s is the shear stress at failure along any plane

σ is the normal stress on that plane and c and ø 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 2.2 is modified to equation 2.3.

$s = c' + (\sigma\text{-}u) \tan ø'$…………………….………………………………………………………….(2.3)

Or

$s = c' + \sigma' \tan ø$…………………….……………………………………………….……………… (2.4)

Where

$\sigma'= (\sigma'\text{-}u)$, the effective normal stress (on the soil skeleton) and u is pore water pressure developed c' and ø' 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 behavior which is intermediate between that for drained conditions and for saturated undrained conditions, depending on the degree of saturation [4].

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.

2.1.3 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 mold a piece of clay between the fingers and applying the observations indicated in

Table 2-1 Estimating the Shear Strength and SPT N-Value form consistency [5]

Description	**qu(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
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 remolded 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 remolded undrained shear strength to liquidity index [LI = $\frac{Wn-PI}{LL-PI}$] have been established by Skempton and Northey (1952) as cited by [4] and these are given in Figure 2-1.

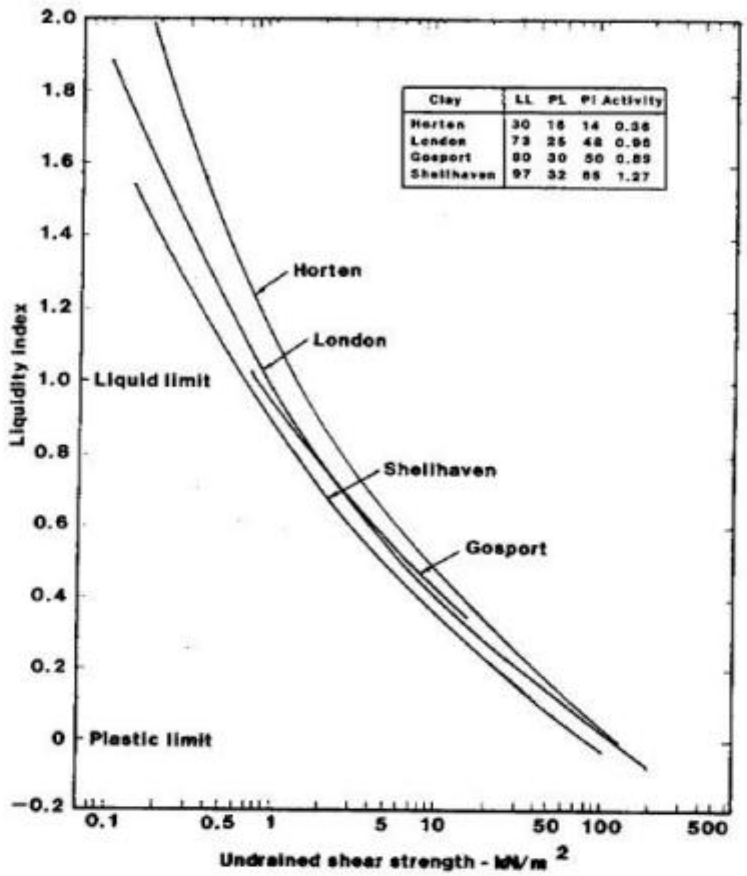

Figure 2-1 Correlation between shear strength and liquidity index established by Skempton and Northey (1952) as cited by [4]

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 remolded 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 2-2.It has been shown that both remolded 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.

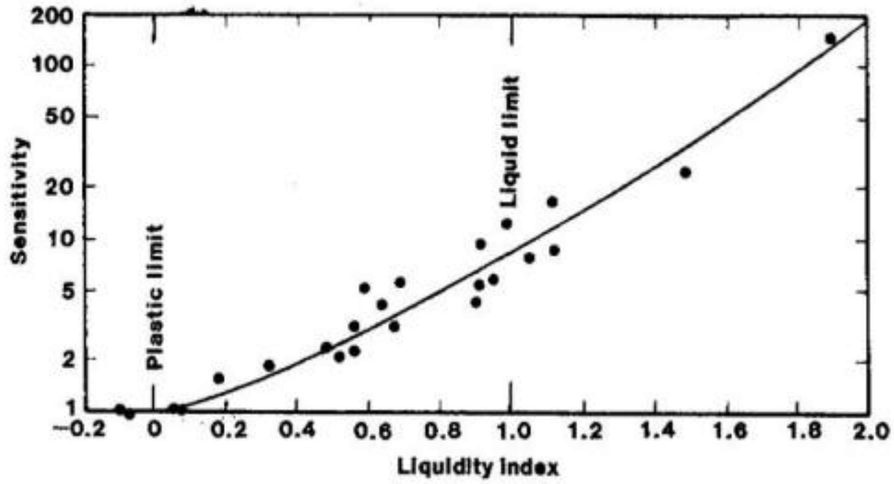

Figure2-2 Correlation between sensitivity and liquidity index after Skempton and Northey (1952) as cited by [4]

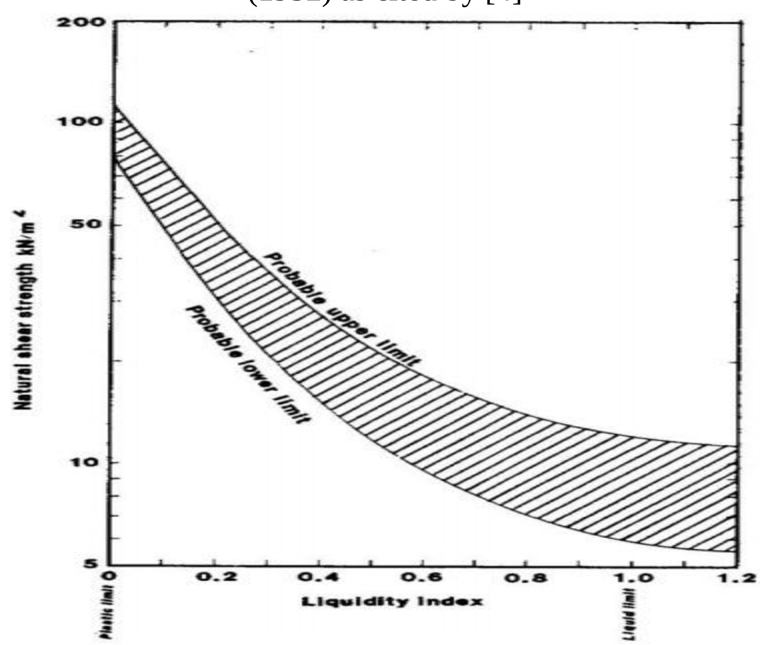

Figure 2-3 Correlation between natural shear strength and liquidity index [4]

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]:

$$\sigma'v= \frac{Su}{0.11+0.003PI} \qquad (2.5)$$

Where PI is the plasticity index

su is undraind shear strength and

$\sigma'v$ is over burden pressure

III. from SPT N-Value

For over consolidated clays, Stroud (1974) as cited by Clayton [6] has reported good correlations between N and cu. 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.

$$c = f\,N \qquad (2.6)$$

Where cu is undrained shear strength; N60 is the blow count normalized to an effective overburden pressure of 100kPa and corrected to 60% of free fall energy; f1 is a coefficient whose values depend strictly upon the plasticity of the clay (Figure 2.6). With known plasticity index, f1 could be read from the other axis since f1 is equal to cu/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 [6].

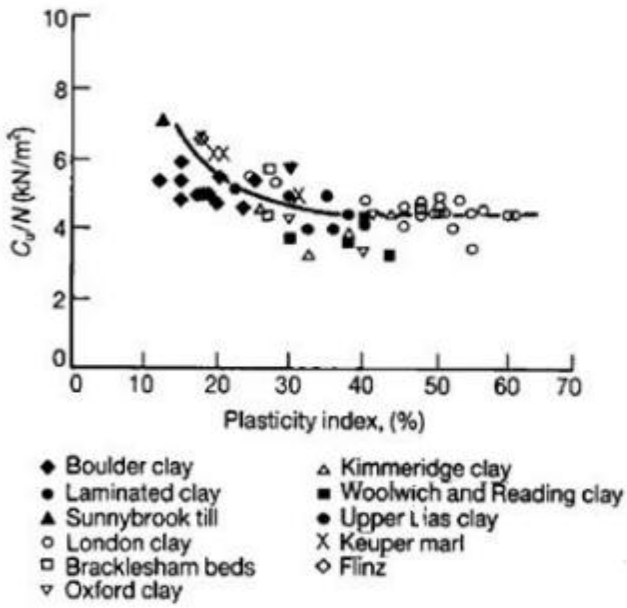

Figure 2-4 Correlation between N value and undrained shear strength for insensitive clays Stroud (1974) as cited by [6]

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 [4] and Clayton [6] shows values with cu/N ratios apparently varying between 0.4 and 20

2.2 Dynamic Cone Penetrometer

2.2.1 Background 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 [1].

The earliest record of a subsoil penetration testing device similar to the DCP is a "ram penetrometer," developed in Germany at the end of 17th century by Nicholas Goldman. The next major development again came from Germany, when Künzel in 1936 developed what was known as a "Prüfstab". This device was later used by Paproth in 1943, and eventually become standardized in 1964 as the "Light Penetrometer", German Standard DIN 4094 [1].

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 [1, 9,10 and 11].

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 [9].

2.2.2 Explanation of the Device

The Dynamic Cone Penetrometer (DCP) consists of two 16-mm (5/8-inch) diameter shafts coupled near midpoint, the lower shaft contains an anvil and appointed tip which driven in to the soil by dropping sliding hammer contained on the upper shaft onto the anvil. The underlying soil strength is determined by measuring the penetration of the lower shaft in to the soil after each hammer drop this value is recorded in millimeters (inches) per blows and known as the DCP penetration index (DPI) the penetration index can be plotted versus depth to identify thicknesses and strength of different pavement layer or can be correlated to other soil strength parameter [2].

Hardware

Competed drawing of the DCP is given in figures 2-5 the equipment is comprised of the following elements:

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.

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.

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

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.

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

Cone: The cone measures 20 mm in diameter and has a 60^o cone.

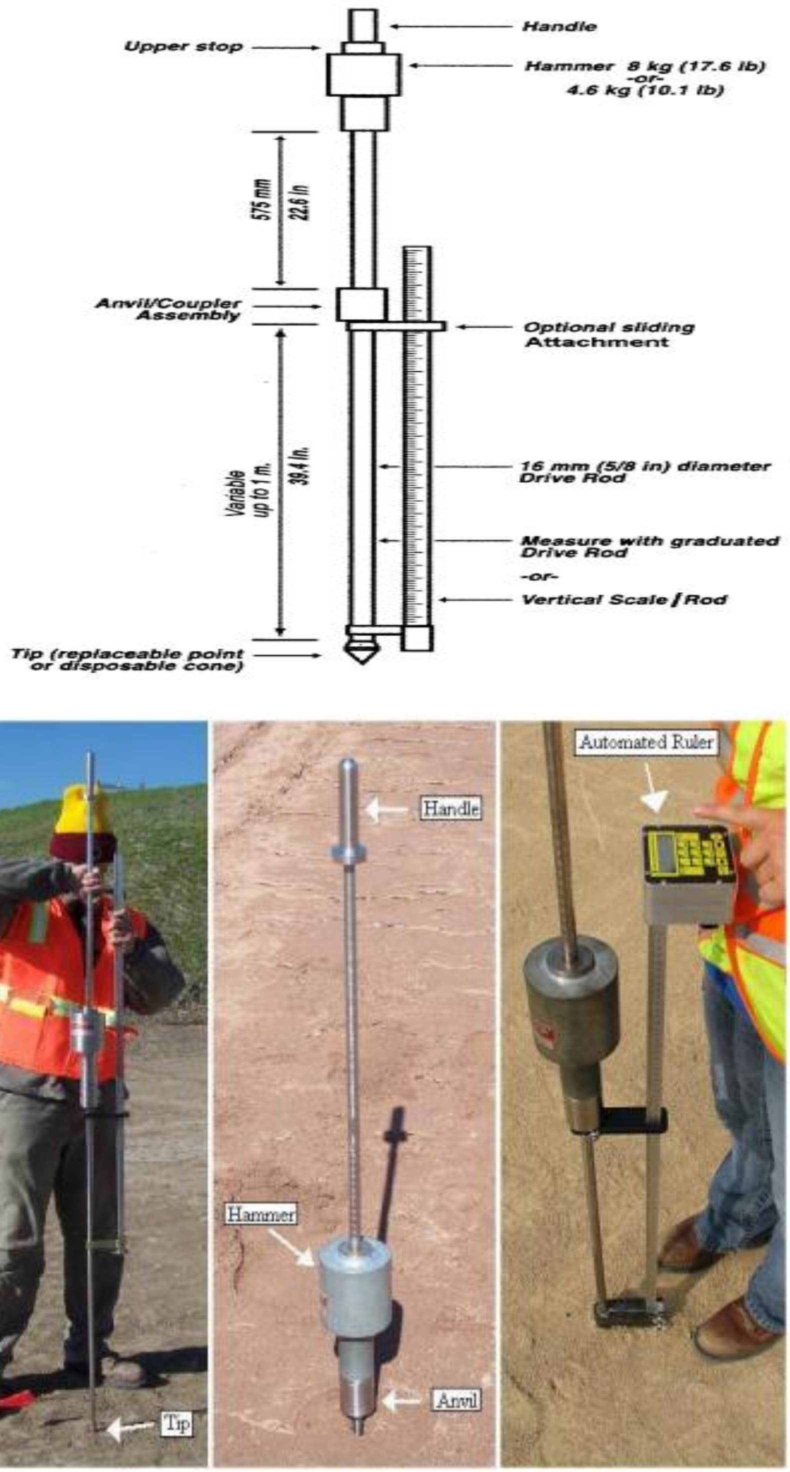

Figure 2-5 Dynamic Cone Penetration Equipment [26]

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 "below 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 millimeters 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, 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.

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 (DCP) and they have implied that the following are the factors affecting the outcome of the DCP results.

a) Material Effects: Klein and Savage as cited by Amini [12] indicated that moisture content, gradation, density, and plasticity were important material properties influencing the DCP. Hassan [13] performed a study on the effects of several variables on the DCP. He concluded that for fine-grained soils, moisture content, soil classification and dry density affect the DCP. For coarse-grained soils, coefficient of uniformity and confining pressures were important variables.

b) Vertical Confinement Effect: Livneh, et al. [14] 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 [13].

2.2.5 Advantages and Disadvantages of DCP

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: Some of the main advantages of the DCP are: [12]

1) DCP has a wide variety of applications including estimations of CBR, resilient modulus, unconfined compressive strength, and shear strengths, as well as its use in performance evaluation of the pavement layers. Other potential application of the DCP includes its use in the quality control of granular base layer compaction

2) The DCP is rapid and economical.

3) The DCP evaluations may be conducted and the results analyzed by personnel with limited training.

Some of the primary disadvantages of the DCP include:

1. High variability exists particularly in the case of large, well-graded granular materials.

2. The use of DCP for materials with a maximum aggregate size of larger than 2 inches is questionable.

3. Some of the existing strength relationships are only applicable to certain material types and conditions, and not to all case

Other limitations:

 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 types 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 DCP Correlation Studies

2.3.1 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 2.2.2 (III). Another correlation that is used for prediction is that of DCP versus unconfined compressive strength (UCS). Several graphs of the correlation between UCS and DCP can be found in literature (Figure 2-6)[15].

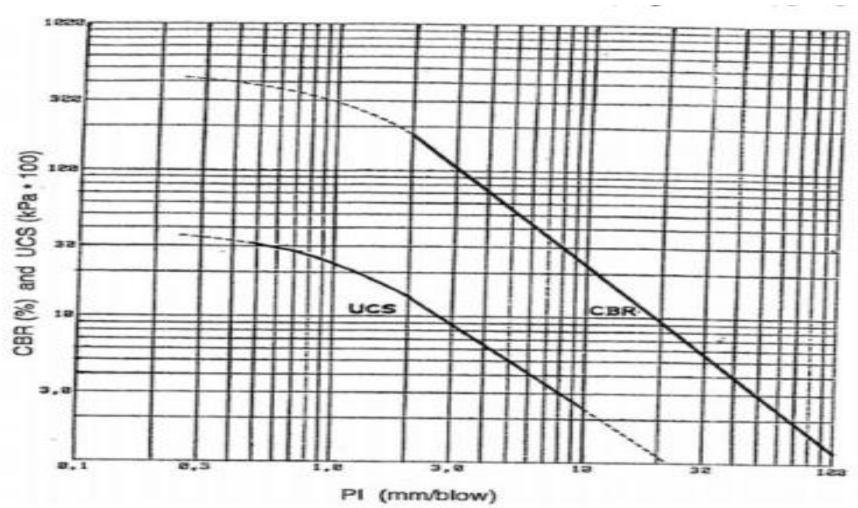

Figure2-6 Relationship between Penetration Index (PI), California Bearing Ratio (CBR) and Unconfined Compressive Strength (UCS) [15]

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 [12]:

Log10(UCS) = 3.56 - 0.807Log10(DCPI)…………………………………….……………………(2.7)

50% probability of underestimation

Log10(UCS) = 3.29 - 0.809Log10(DCPI)…………………...……….….…………………(2.8)

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

Log10(UCS) = 3.21 - 0.809Log10(DCPI)……………………………….…………………(2.9)

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).

2.3.2 Prediction of SPT N-value

Sowers and Hedges [8] and later Livneh and Ishai [16], developed a correlation between DCPI and standard penetration test (SPT) results which are only valid for SPT < 10mm/blow(Figure 2-7). The correlation equation took the form:

Log10 (DCPI)=-A+ B Log10(SPT)……………………………………….………………(2.10)

Should be noted that both studies involved the use of DCP's having slightly different type of light penetrometer. Refer to Livneh and Ishai [16] for more detail.

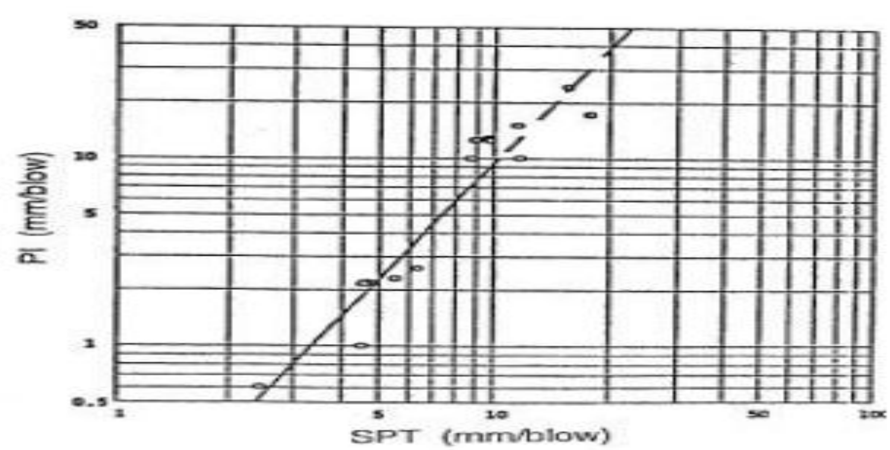

Figure 2-7 Relationship between Penetration Index (PI) and SPT [16]

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 [17] (Table 2-2).

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

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

2.4 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 behavior of the soil caused by the application of these forces.

DCP tip to soil interaction behavior 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 [18].

Chua [21] formulated his modeling 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 [18] for projectiles. Using stresses and strains from the model, Chua develop a correlation of Penetration Index (PI) versus elastic modulus for various types of soils.

Chua and Lytton [22] also performed a "structural system" type dynamic analysis including both the DCP and its soil interaction. In the analysis, the DCP is modeled 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 [8].

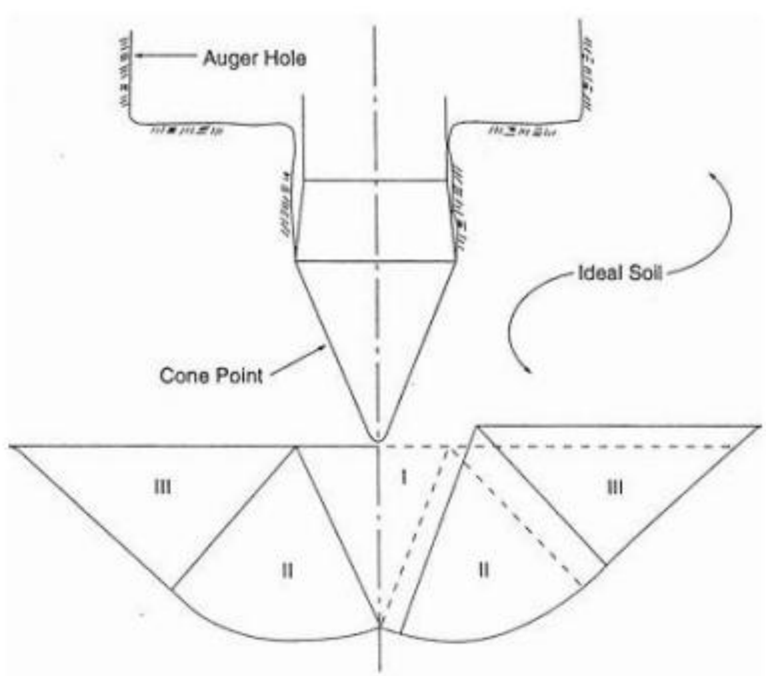

Figure 2-8 Theoretical principles of penetration equipment [8]

2.5 Bearing Capacity Theory

An important problem of foundation engineering is the computation of the maximum load (the bearing capacity) and Coulomb's method 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 [3].

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 ($\varnothing = 0$), so that its only relevant property that is considered is the cohesive strength c.

$P_c = (\pi + 2)c = 5.14c$……………………………………..……..…………………………(2.11)

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 2-9). It can be used to represent the effect of the depth of the foundation (d) below the soil surface. In that case $q = \gamma d$, where γ is the unit weight of the soil.

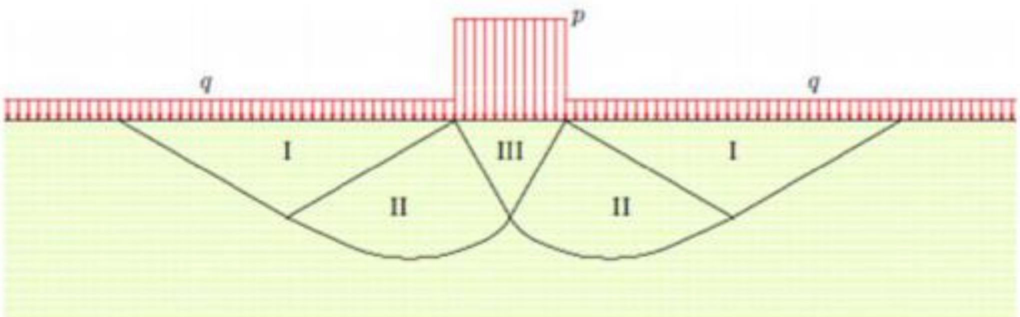

Figure 2-9 Schematization Prandtl's of strip foundation as cited by [3]

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

$p = cN_c + qN_q$……………….…………………………………………….……………………(2.12)

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

$$Nq= \frac{1 + \sin\phi -}{-1 - \sin\phi -} \exp(\pi \tan\phi) \qquad (2.13)$$

$$Nc = (Nq-1) \cot\phi \qquad (2.14)$$

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]

$$p = cNc + qNq + 0.5B\gamma N\gamma \qquad (2.15)$$

B is the total width of the loaded strip. For the coefficient $N\gamma$, various suggestions have been made on the basis of theoretical analysis or experimental evidence or depending on the safety needed, for instance [19]

$$N\gamma= 2(Nq - 1)\tan\phi \qquad (2.16)$$

or

$$N\gamma= 1.5(Nq-1)\tan\phi \qquad (2.17)$$

Even though the values of Nc, Nq and $N\gamma$ are given, as a function of the friction angle ø,since we are considering the limiting case ø = 0, the value of $Nc=2 +\pi=5.142$, $Nq= 1$ and $N\gamma = 0$. The following ultimate bearing capacity equation is used for the current thesis:

$$q_{ult} = 5.142c + \gamma h \qquad (2.18)$$

Where, qult = the ultimate bearing capacity,

c = undrained shear strength (cu)

h = depth to cone tip

γ = average unit weight of the soil

CHAPTER THEER

MATERIALS AND METHODS

3.1 Study Area

Jimma town is one of the biggest towns located in the Oromia National Regional State. The town is a center for large trunk roads passing different part of Ethiopia; Due to this reason the town is a meeting place for different nationalities, languages and a place for marketing. The town is located in western part of Ethiopia 7°40′N 36°50′E latitude and longitude. Jimma has a tropical rainforest climate (Af) under the Köppen climate classification. It features a long annual wet season from March to October. Based on the 2007 Census conducted by the Central Statistical Agency of Ethiopia (CSA), this Zone has a total population of 120,960, of whom 60,824 are men and 60,136 women with an area of 50.52 square kilometers. Temperatures at Jimma are in a comfortable range, with the daily mean staying between 20°C and 25°C year-round.

3.2 Experimental Techniques

3.2.1 Field Identification, Tests and Sampling Methods

- Description and identification of soil (visual-manual procedure) to conduct this using ASTM D 2488[23]
- Dynamic cone penetration test procedure are conducted by using overseas road Note 8,for a program analyze Dynamic cone penetration data TRRI,1992 and ASTM D 695-03[17] and[23]
- Soil sampling, disturbed and undisturbed collected by using test procedure ASTM D 4220 and ASTM D 1587[23]

3.2.2 Laboratory Test Methods

After the representative soil samples are collected, the following laboratory tests were conducted.

- Water content
- Dry density
- Specific gravity
- Atterberge limits (LL.PL,PI & SL)

- Grain size analysis
- Unconfined compression (uc) test

Atterberg Limits

Atterberg Limits were determined for air-dried samples. It was done based on the Standard Reference: ASTM D 4318 /AASHTO 89-90/BS Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of soils. The air- dried samples were prepared by spreading the specimen in the air until it dried. The room temperature was about 18-23 oc. The portions of the samples passing the No. 40 (0.425mm) sieve were used for the preparation of the sample for this test. [23 and 24]

Moisture content of soil

The water content of the soil is an important property. The characteristics of a soil, especially a fine-grained soil, change to a marked degree with a variation of its water content. The natural moisture contents of the soil under investigation were determined following ASTM D2216-98[23].

Specific gravity

In this research, specific gravities of soils of the study area are conducted following ASTM D854-98[23] The specific gravity determination of a sample of soil is made by displacement in water using pycnometer (volumetric bottle).The specific gravity we get by this method is the absolute specific gravity. In this test a known weight of oven-dried soil sample is carefully put in a pycnometer which is then half filled with distilled water. The air entrapped in the soil sample is removed by heating or by means of vacuum pump. The bottle is then topped up with distilled water up to a calibration mark and brought up to a constant temperature.

Grain size analysis

grain size of fine grained soils (clay and silt) or soils passing through sieve No 200. For grain size analysis wet sieve method is used after air drying the sample. Test was conducted by following ASTM D 422-63 procedures [7, 23]. Samples collected from the Sites were air dried and representative samples were selected by quartering. After measuring the weight of representative samples, the samples were washed on sieve No 200.The portion of soils retained on sieve No. 200 were oven dried and mechanical sieves were conducted.

Unconfined compression (uc) test

The primary purpose of this test is to determine the unconfined compressive strength, which is then used to calculate the unconsolidated undrained shear strength of the clay under unconfined conditions. According to the ASTM D 2166 standard, [23] the unconfined compressive strength (qu) is defined as the compressive stress at which an unconfined cylindrical specimen of soil will fail in a simple compression test.

Unit Weight of soil

Unit weight of soils of the study area are conducted following ASTM D 1188 and ASTM D 2216 [23]

Classification system

The classification of the study area are conducted following ASTM D-2487 for unified classification system.[23]

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 Jimma Area soils.

Figure 3-1 Locations of data collection in Jimma (Google Earth)

Samples of 23 disturbed and 23 undisturbed were collected from 12 test pits. The sampling location is selected so that it can be well represent soils found in Jimma town. Selection of these sampling locations also considers visual soil classification, economical importance of sampling area, non-uniformity of the sample locations and coverage of the section. During excavation for sampling, it was observed that the ground water table is found far below 2.5m depth.

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 divided into two Parts

- Part one is clay soils including red clay, black and gray soils of Jiimma combined
- Second part Red clay soils of Jimma

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 (Figure3-2). The figure shows how the first drops do not accurately represent the average dynamic cone penetration (DCP) at 0.0, 1.0, and 1.6 m. 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 [26].

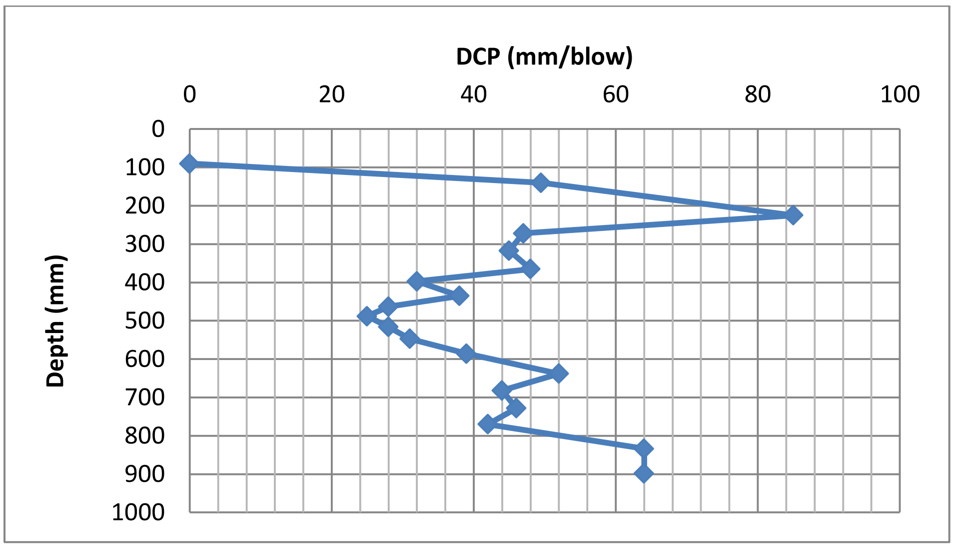

Figure 3-2 Dynamic Cone Penetration (DCP) versus depth for KitoFurdisa Indicating the unreliability of initial drops

The Penetration data obtained in the field was conducted using the procedure stated 3.2.1The 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 in-situ characteristics in an enhanced way.

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 Introduction

All the properties of soil are expressed by field density, water content, specific gravity; Atterberg's limit unconfined compression test and Dynamic cone penetration test. These have been done both in the field and laboratory.

4.2 Summary of Test Results

Table 4-1 Summary of test results of the study area

Serial no	Station	depth(m)	specific gravity	LL (%)	PL (%)	PI(%)	n.m.c (%)	liquidity index	qu(kPa)	DCP(mm/blow)	% Passing 75µm	Field Density(g/cm3)
1	Kito Furdisa	1	2.57	60	25	35	37.8	1.03	210.0	15.4	93.20	1.4
2	Kito Furdisa	1.5	2.58	72	20	52	41.1	1.18	295.0	12.6	93.30	1.4
3	Kito Furdisa	2.5	2.49	69	41.1	37.9	37.8	1.82	310.0	11.0	92.60	1.5
4	Furstale	1.5	2.58	65	31	34	30.7	0.17	272.0	12.0	87.90	1.2
5	Furstale	2.5	2.65	70	31	39	36.9	0.92	300.0	11.0	91.50	1.3
6	Agriculture	1.5	2.69	59	28	31	28.1	1.26	230.0	19.0	90.50	1.2
7	Agriculture	2.5	2.61	71.3	35	36.3	56.5	0.41	297.0	12.0	92.80	1.3
8	Merkato-1	1.5	2.73	88	36	52	50.3	0.72	106.0	21.0	88.80	1.1
9	Merkato-1	2.5	2.69	66	30.1	35.9	43.5	0.63	168.0	19.0	91.90	1.2
10	Awetu	1.5	2.75	59	27.1	31.9	42.3	0.68	217.0	22.3	89.69	1.3
11	Awetu	2.5	2.63	56	29	27	46.6	0.67	240.0	18.0	91.90	1.4
12	Main Campus	1.5	2.69	54	28	26	40.5	0.38	230.0	12.0	87.89	1.2

13	Main Campus	2.5	2.68	70	32	38	46.4	0.76	240.0	7.4	90.06	1.3
14	Kebele-5	1.5	2.72	100	38.3	61.7	65.3	0.56	137.0	18.5	89.65	1.1
15	Kebele-5	2.5	2.75	108	40.8	67.2	70.1	0.56	165.0	17.0	91.86	1.2
16	Bocho Bore	1.5	2.71	60	28	32	39.9	0.51	144.0	17.4	90.55	1.1
17	Bocho Bore	2.5	2.79	98.5	32	66.5	50.7	0.20	166.0	15.1	92.78	1.1
18	Technic Sefer	1.5	2.69	104	35.1	68.9	40.6	0.92	115.0	25.3	87.83	1.1
19	Technic Sefer	2.5	2.67	92	35.1	56.9	47.8	0.78	161.0	18.6	91.85	1.2
20	Kochi	1.5	2.69	58	21.7	36.3	39.9	0.50	138.0	38.0	86.08	1.1
21	Kochi	2.5	2.68	64	21.7	42.3	51.4	0.30	152.0	21.0	90.02	1.2
22	Merkato-2	1.5	2.70	70	21.7	48.3	49.2	0.43	112.0	35.0	87.80	1.19
23	Merkato-2	2.5	2.70	94	30.7	63.3	51.6	0.67	121.0	25.0	90.92	1.2

4.3 Discussion of Test Results

4.3.1 General

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

4.3.2 Clay Soils of Jimma

From visual observations and field tests, the soils of the area are classified as clay with high plasticity. According to the Unified Soil Classification System (USCS) the soils are classified as CH (clay with high plasticity) and according to AASHTO classification system the soil are classified as A-7-5 and A-7-6 which are clayey soils

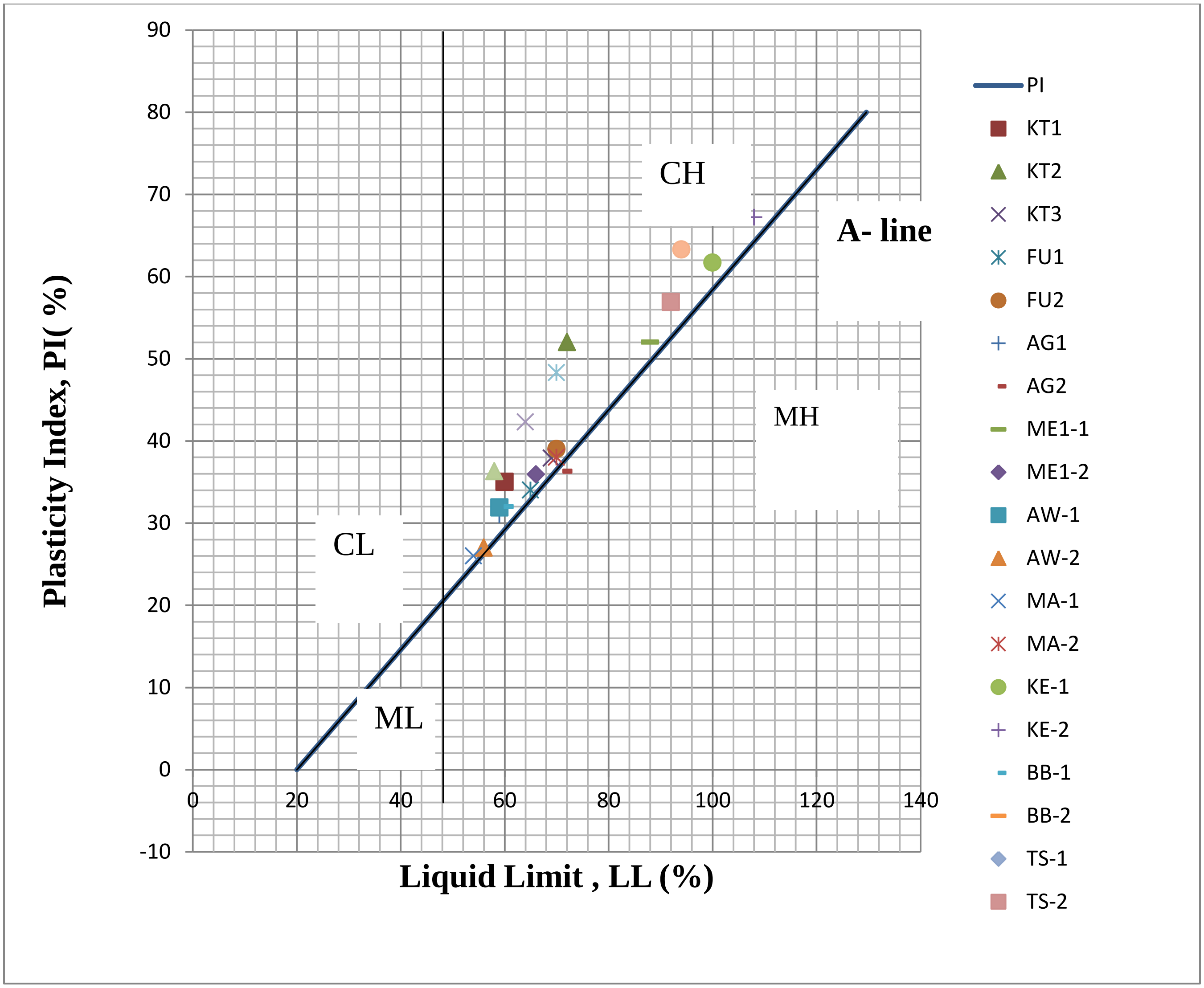

Figure 4.1 USCS classification for Jimma clay soils using plasticity chart

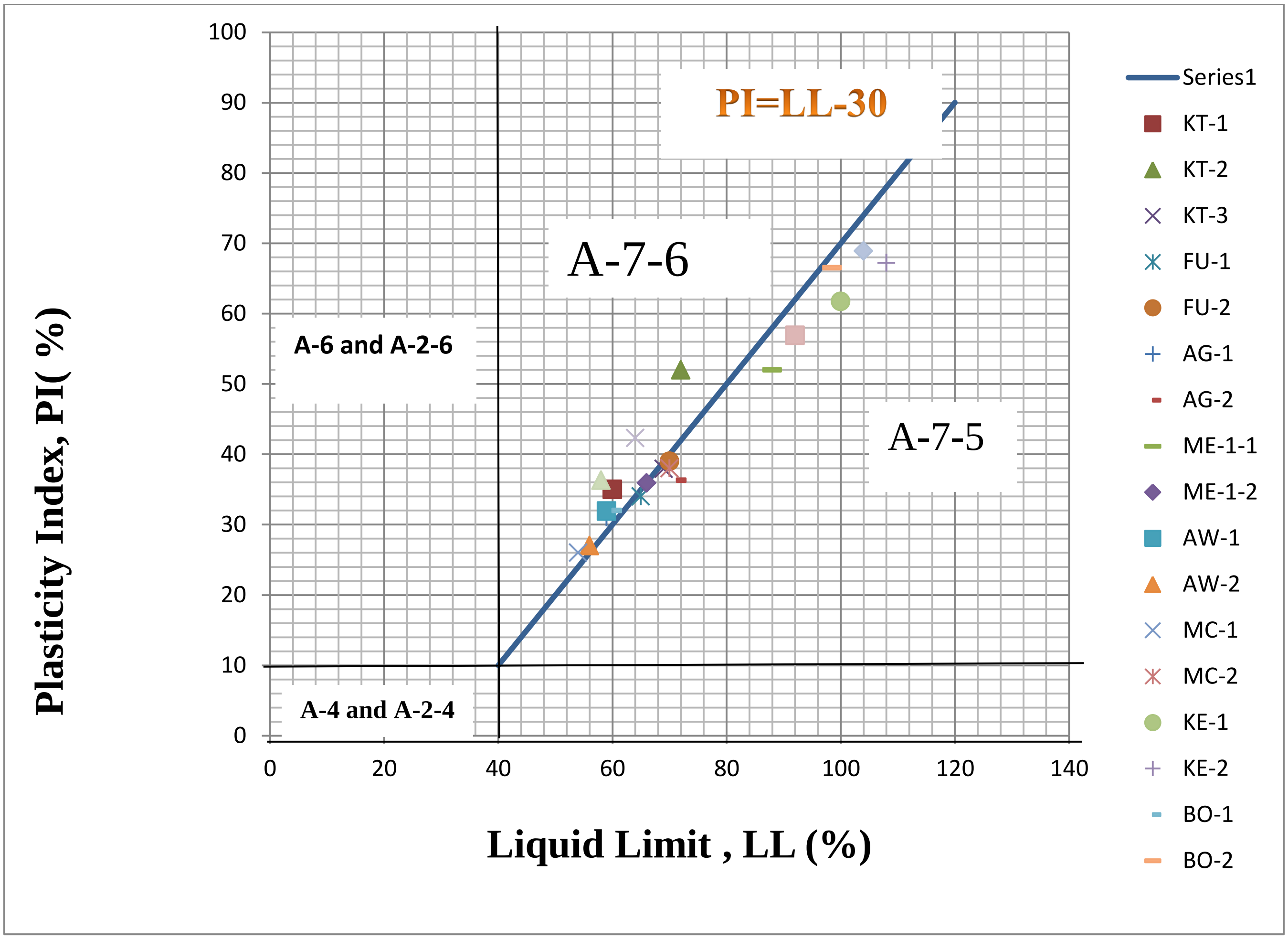

Figure 4. 2 AASHTO plasticity chart classification for the Jimma fine-grained soils

Grain size analysis and other index tests were conducted on twenty-three (23) representative samples in the current research the percentage finer of all samples are more than 50%. The plot of soil gradations conducted in current research is presented in Appendix.

Results of DCP conducted on the clay soils of Jimma converted to an equivalent SPT N-value indicated a range between 4.9 and 36 (Table 4.2).

In the laboratory unconfined compression test results indicate that the soils have a qu value ranging from 106 to 310kPa.

1. 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 [5] and Janbu (1963) as cited by [27]. Then the range of expected value for undrained shear strength is estimated based on Bowles [5] and BS 5930 [29]. 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 Kochi 1.5m (Table 4.2) the soil has a soft 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 [27] and Bowles [5], respectively The consistencies obtained have estimated shear strength ranging from 50kPa to 75kPa and 25kPa to 50kPa according to BS 5930 [29] and Bowles [5], respectively. The actual value of shear strength obtained using unconfined compression strength was 138kPa. This indicates that actual value fall outside the estimated range

Table 4-2 Consistency determinations for clay soils of Jimma

Site designation	DCP (mm/blow)	SPT N-value based on Overseas road note(table2.2)	Range of Consistency Suggested by Bowles [SPT N-value]	Consistency according to Bowles [11], S=1
Kito Furdisa 1m	15.4	17.2	17-30	VERY STIFF
Kito Furdisa 1.5m	12.6	20.8	17-30	VERY STIFF
Kito Furdisa 2.5m	11.0	21.4	17-30	VERY STIFF
Furstale 1.5m	12.0	22	17-30	VERY STIFF
Furstale 2.5m	11.0	21.4	17-30	VERY STIFF
Agriculture 1.5m	19.0	14.5	10-16	STIFF
Agriculture 2.5m	12.0	22	17-30	VERY STIFF
Merkato-1 1.5m	21.0	13.5	10-16	STIFF
Merkato-1 2.5m	19.0	14.5	10-16	STIFF
Awetu 1.5m	22.3	12.75	10-16	STIFF
Awetu 2.5m	18.0	15	10-16	STIFF
Main Campus 1.5m	12.0	23	17-30	VERY STIFF
Main Campus 2.5m	7.4	36	>30	HARD
Kebele-5 1.5m	18.5	14.75	10-16	STIFF
Kebele-5 2.5m	17.0	15.5	10-16	STIFF
Bocho Bore 1.5m	17.4	15.4	10-16	STIFF
Bocho Bore 2.5m	15.1	16.5	10-16	STIFF
Technic Sefer1.5m	25.3	11	10-16	STIFF
Technic Sefer 2.5m	18.6	14.7	10-16	STIFF
Kochi 1.5m	38.0	5.6	3-5	SOFT
Kochi 2.5m	21.0	13.5	10-16	STIFF
Merkato-2 1.5m	35.0	4.9	3-5	SOFT
Merkato-2 2.5m	25.0	12.88	10-16	STIFF

Table 4-3 Shear strength estimation based on consistency for clay soils of Jimma

Site designation	Estimated qu based on Consistency		Actual Value of qu
	According to Bowles using SPT N-value	Using Liquidity index BS 5930	
Kito Furdisa 1m	200-400	150-300	210.0
Kito Furdisa 1.5m	200-400	75-150	295.0
Kito Furdisa 2.5m	200-400	75-151	310.0
Furstale 1.5m	200-400	75-152	272.0
Furstale 2.5m	200-400	75-153	300.0
Agriculture 1.5m	200-400	15.-300	230.0
Agriculture 2.5m	200-400	50-75	297.0
Merkato-1 1.5m	100-200	75-150	106.0
Merkato-1 2.5m	100-200	75-150	168.0
Awetu 1.5m	100-200	75-150	217.0
Awetu 2.5m	100-200	75-150	240.0
Main Campus 1.5m	200-400	50-75	230.0
Main Campus 2.5m	>400	75-150	240.0
Kebele-5 1.5m	100-200	75-150	137.0
Kebele-5 2.5m	100-200	75-150	165.0
Bocho Bore 1.5m	100-200	75-150	144.0
Bocho Bore 2.5m	100-200	75-150	166.0
Technic Sefer 1.5m	100-200	75-150	115.0
Technic Sefer 2.5m	100-200	75-150	161.0
Kochi 1.5m	25-50	50-75	138.0
Kochi 2.5m	100-200	50-75	152.0
Merkato-2 1.5m	25-50	75-150	112.0
Merkato-2 2.5m	100-200	75-150	121.0

2. The index properties of Jimma clay soils from current research and previous researches are presented in Table 4.4 and specific gravity, density, percentage of fines, NMC and shear strength are presented in Table 4-5 for comparison purpose

Table 4.4 Comparison of index properties and classification of Jimma soils with the previous works [32] and [33]

Elements of comparison / Research work	Colors	Liquid limit, LL (%)	Plastic limit, PL (%)	Plastic index, PI (%	UCS Classification
Jemale J [32]	Black, Gray, Red	53-108	25-41	21-65	CH, MH
Jemale Z [33]	Black, Gray, Brown, Red	60 – 109	22 – 44	31 – 77	CH
Current	Black, Gray, Red	54-108	20-41	26-68	CH

Table 4.5 Comparison of specific gravity, density, percentage of fines, NMC and shear strength of Jimma soils with the previous research work test results [32] and [33]

Elements of comparison / Research work	Specific Gravity, Gs	Field Density, γ_{bulk} (kN/m3)	Percentage Fines (%)	NMC (%)	Undrained Shear Strength, cu (kPa)
Jemale J [32]	2.58-2.81	16.75- 17.98	61 – 99	33 – 56	43 – 157
Jemale Z [33]	2.50 – 2.82	14.30 –17.80	65 – 96	31.0 – 64.7	32.2 – 324.6
Current	2.49-2.79	14.44-18.88	86.8-93.3	28.1-70.1	53-155

4.3.3 Clay Soils after Incorporating Data of Jimma

Data selected from different part of the city were plotted along the A-line According to USCS; the soils are classified as inorganic clay with high plasticity (CH). This criterion was used for data acceptance/rejection (i.e., data that have high plasticity and more than 50% fines were incorporated).

The collected samples of Jimma also used to the parametric study that affect the result of dynamic cone penetration like bulk unit weight, dry unit weight, natural moisture content, liquidity index, plasticity index, and liquid limit.

4.4 Correlation Analysis

4.4.1 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 [30].

In this study two sets of investigations are conducted. The first set considers qu as the dependent variable whereas DCP, γdry, γbulk, NMC, LI, LL, and PI are independent variables. The second set considers DCP as the dependent variable and the independent parameters employed for the investigation of qu are used. To carry out statistical analysis SPSS and Microsoft Excel was used for 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 Correlation 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, bulk unit weight, dry unit weight, natural water content, liquidity index, plasticity index and liquid limit Table 4-6 present the data of the researcher used for analysis .

Table 4-6 Data from the study area used for analysis

item no	Station	depth(m)	qu (kPa)	DCP(mm/blow)	specific gravity	LL (%)	PI (%)	liquidity index	n.m.c. (%)	γ_{dry} (kN/m3)	% Passing 75µm	γ_{bulk} kN/m3)
1	Kito Furdisa	1	240.0	15.4	2.57	60	35	1.0	37.8	13.07	93.20	17.98
2	Kito Furdisa	1.5	295.0	12.6	2.58	72	52	0.8	41.1	13.73	93.30	18.88
3	Kito Furdisa	2.5	310.0	11.0	2.49	69	37.9	0.9	37.8	13.99	92.60	19.23
4	Furstale	1.5	272.0	12.0	2.58	65	34	0.2	30.7	11.54	87.90	17.31
5	Furstale	2.5	300.0	11.0	2.65	70	39	0.9	36.9	12.12	91.50	18.18
6	Agriculture	1.5	230.0	19.0	2.69	59	31	1.3	28.1	11.75	90.50	16.92
7	Agriculture	2.5	297.0	12.0	2.61	71.3	36.3	0.4	56.5	12.34	92.80	17.76
8	Merkato-1	1.5	106.0	21.0	2.73	88	52	0.7	50.3	10.78	88.80	14.86
9	Merkato-1	2.5	168.0	19.0	2.70	66	35.9	0.6	43.5	11.32	91.90	15.60
10	Awetu	1.5	217.0	22.3	2.75	59	31.9	0.7	42.3	12.48	89.69	17.98
11	Awetu	2.5	240.0	18.0	2.63	56	27	0.7	46.6	13.11	91.90	18.88
12	Main Campus	1.5	230.0	12.0	2.70	54	26	0.4	40.5	11.86	87.89	17.07
13	Main Campus	2.5	240.0	7.4	2.68	70	38	0.8	46.4	12.45	90.06	17.93
14	Kebele-5	1.5	137.0	18.5	2.72	100	61.7	0.6	65.3	11.00	89.65	15.15
15	Kebele-5	2.5	165.0	17.0	2.75	108	67.2	0.6	70.1	11.55	91.86	15.91
16	Bocho Bore	1.5	144.0	17.4	2.71	60	32	0.5	39.9	10.45	90.55	14.40
17	Bocho Bore	2.5	166.0	15.1	2.79	98.5	66.5	0.2	50.7	10.97	92.78	15.12
18	Technic Sefer	1.5	115.0	25.3	2.69	104	68.9	0.9	40.6	10.65	87.83	14.67
19	Technic Sefer	2.5	161.0	18.6	2.67	92	56.9	0.8	47.8	11.18	91.85	15.40
20	Kochi	1.5	138.0	38.0	2.69	58	36.3	0.5	39.9	10.97	86.08	15.11
21	Kochi	2.5	152.0	21.0	2.68	64	42.3	0.3	51.4	11.51	90.02	15.87

4.5 Single Regression

4.5.1 Scatter Plot for Part one (Clay Soils of Jimma)

In developing correlations, the first step is creating a scatter plot of the data .A scatter diagram is generated by applying the Excel Spreadshee**t**, to visually assess the strength and form of some type of relationship. In the figures below (Figure 4-2 to 4-14) the scatter plot of the dependant variables qu and DCP with independent variables γdry, γbulk, NMC, LI, PI, and LL are presented.

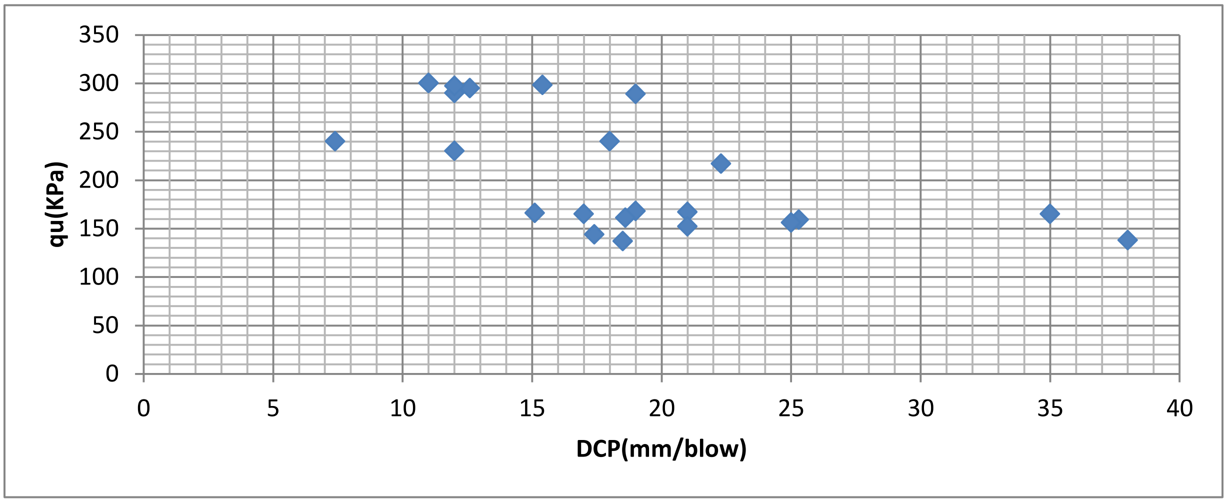

Figure 4-2 Scatter plot of qu with DCP for clay soils of Jimma

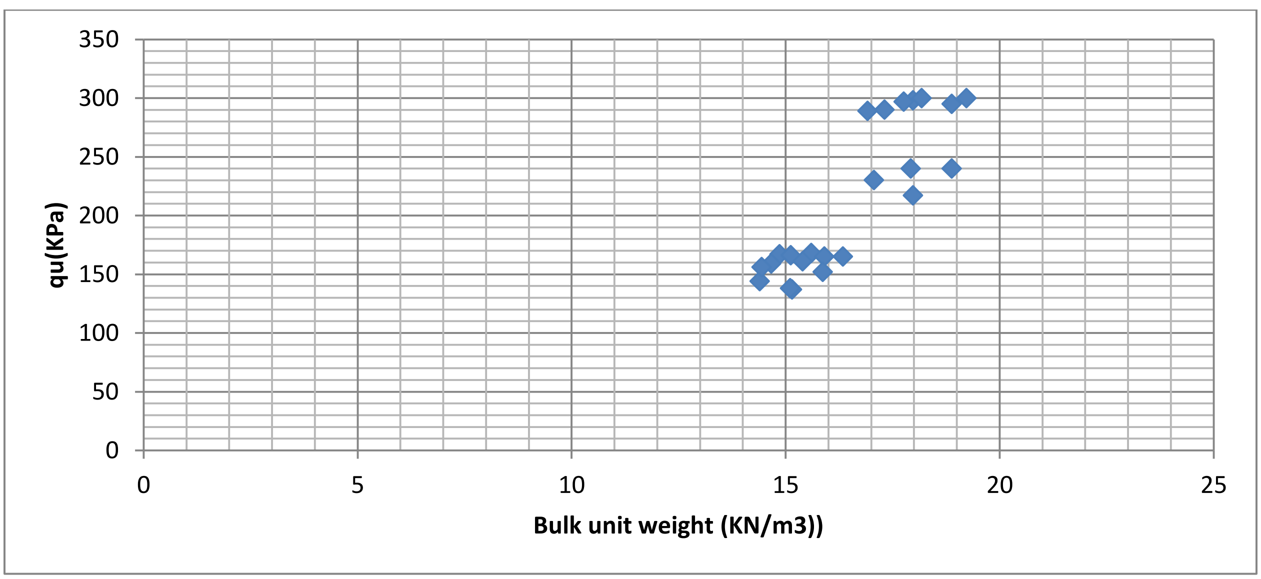

Figure 4-3 Scatter plot of qu with bulk unit weight for clay soils of Jimma

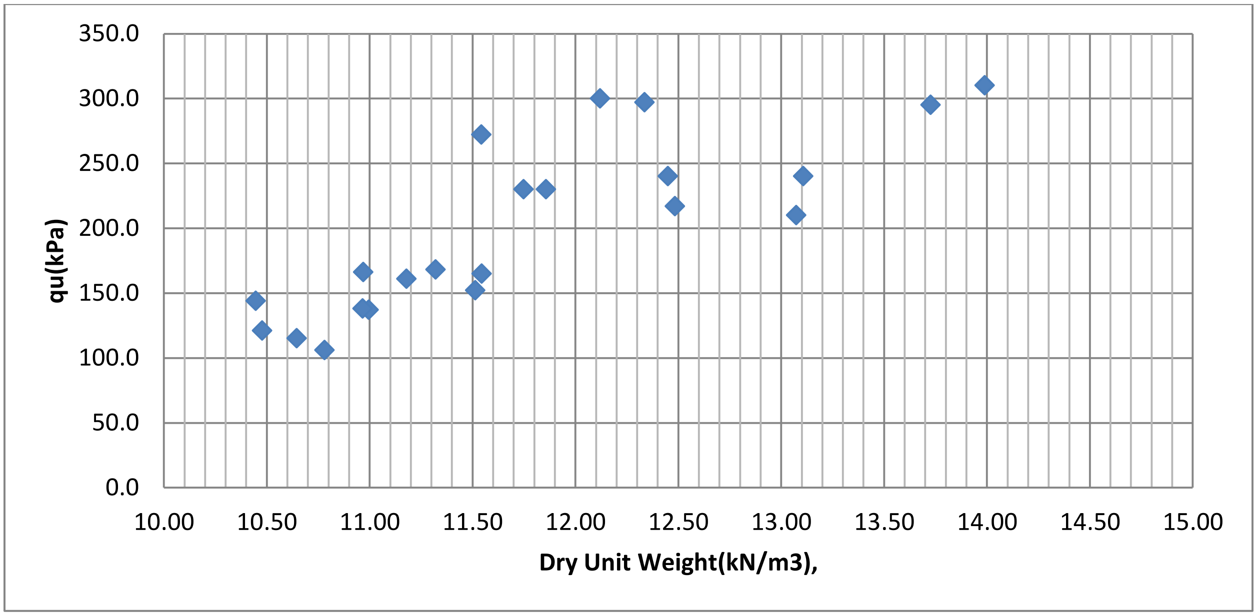

Figure 4-4 Scatter plot of qu with dry unit weight for clay soils of Jimma

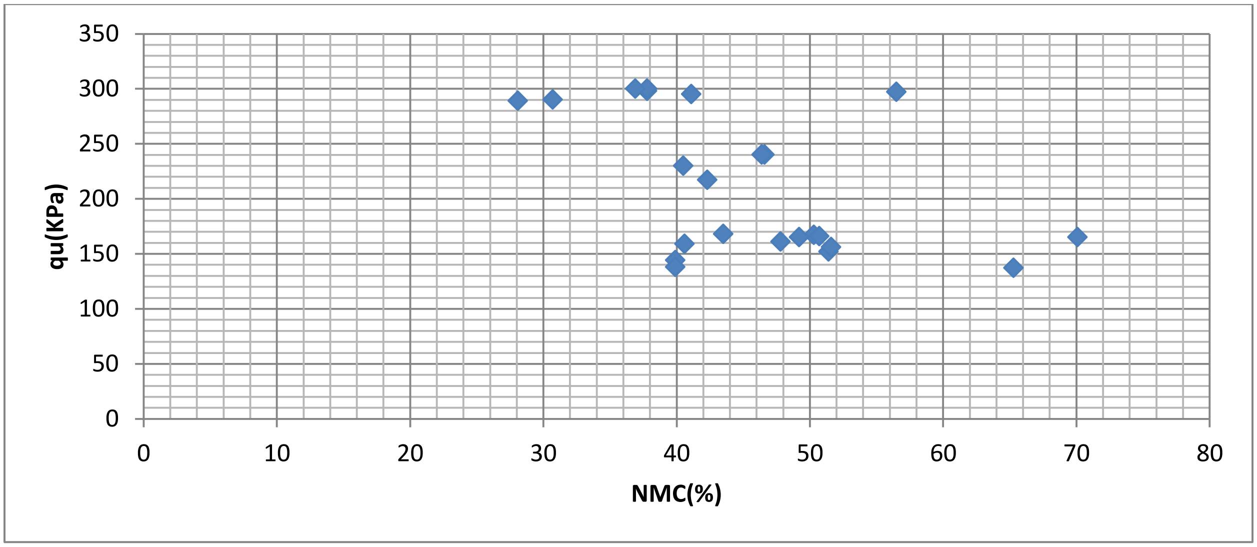

Figure 4-5 Scatter plot of qu with natural moisture content for clay soils of Jimma

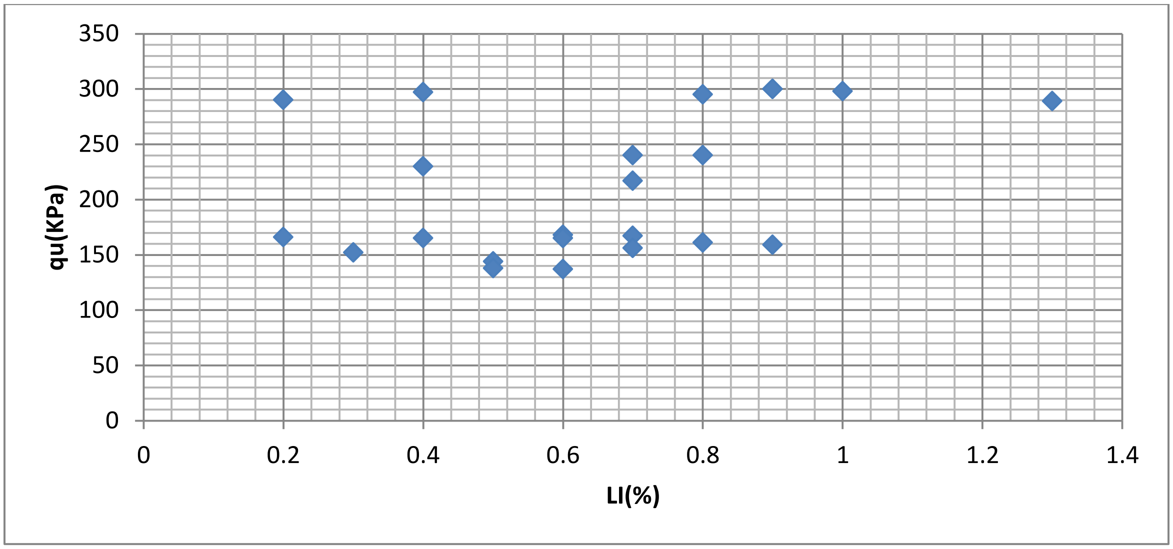

Figure 4-6 Scatter plot of qu with liquidity index for clay soils of Jimma

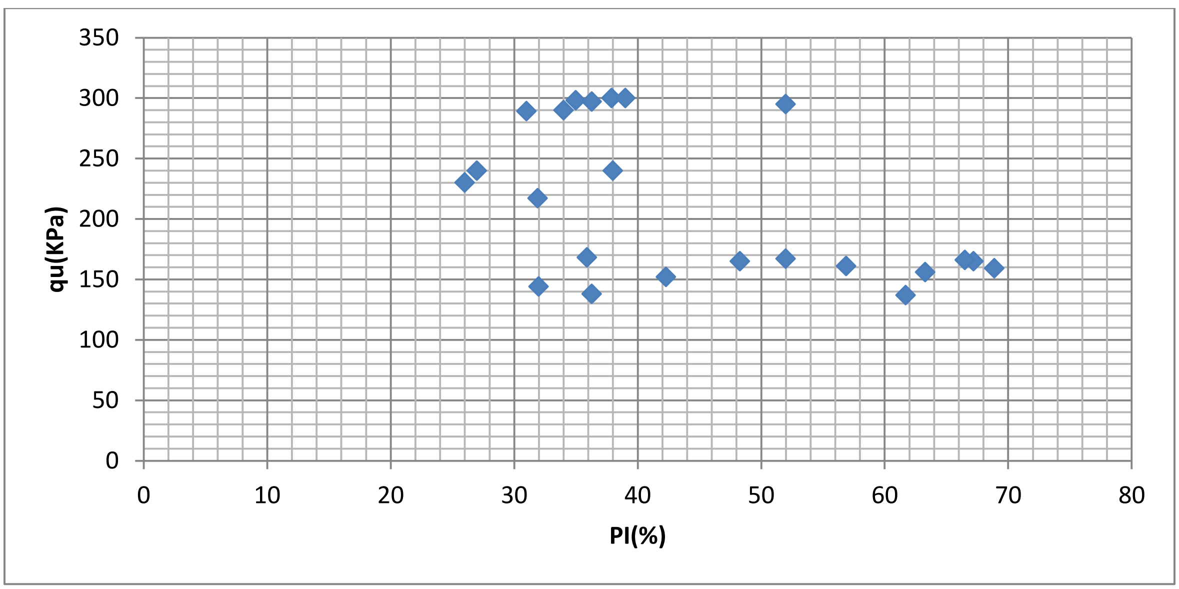

Figure 4-7 Scatter plot of qu with plasticity index for clay soils of Jimma

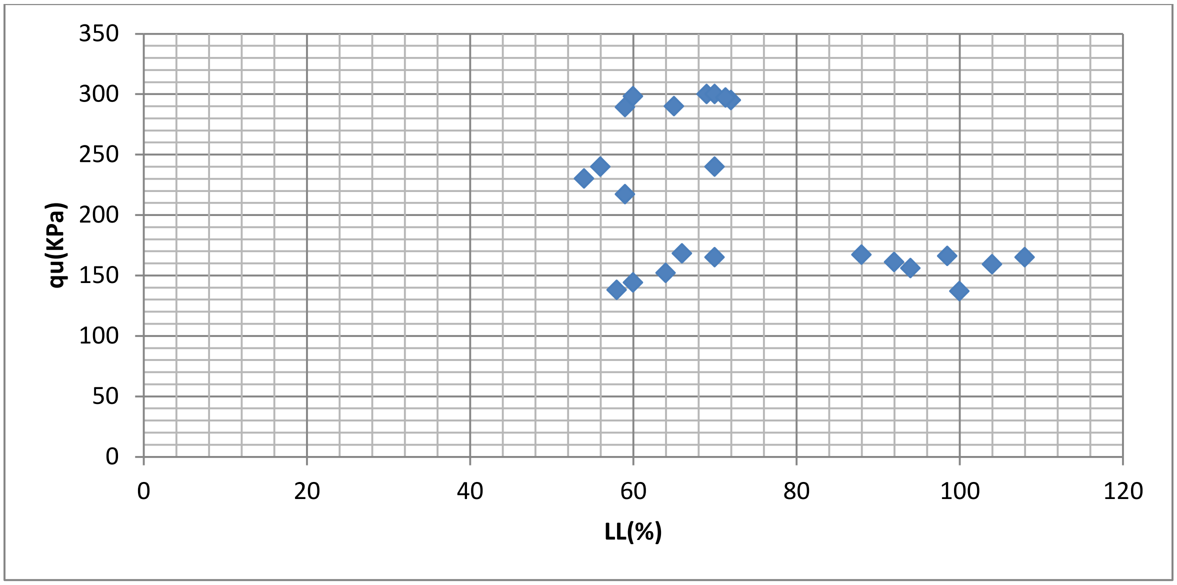

Figure 4-8 Scatter plot of qu with liquid limit for clay soils of Jimma

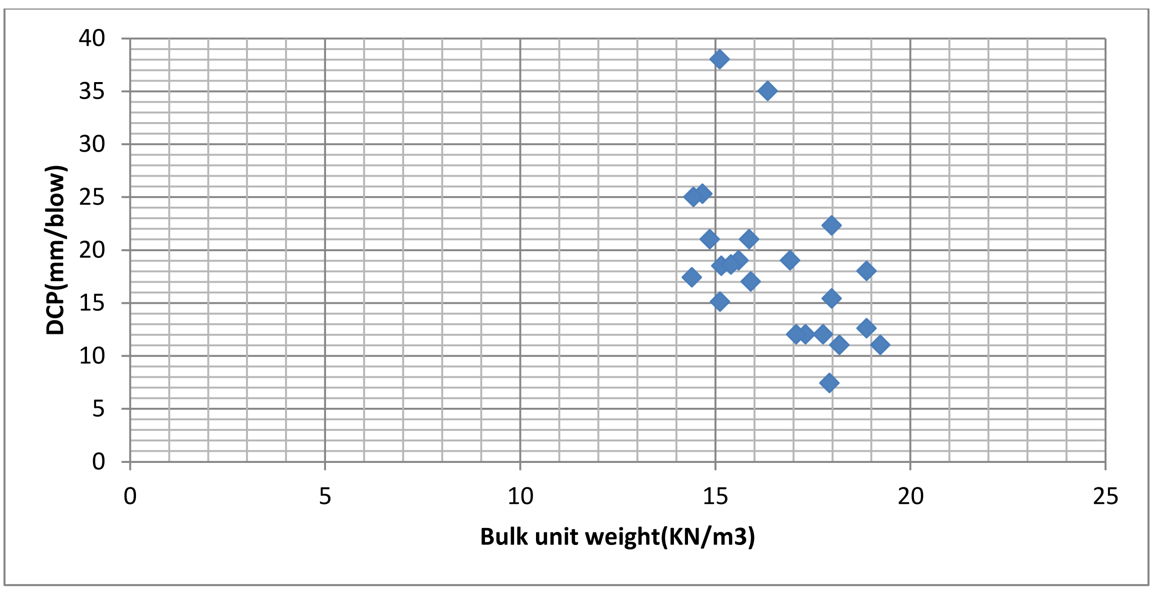

Figure 4-9 Scatter plot of DCP with bulk unit weight for clay soils of Jimma

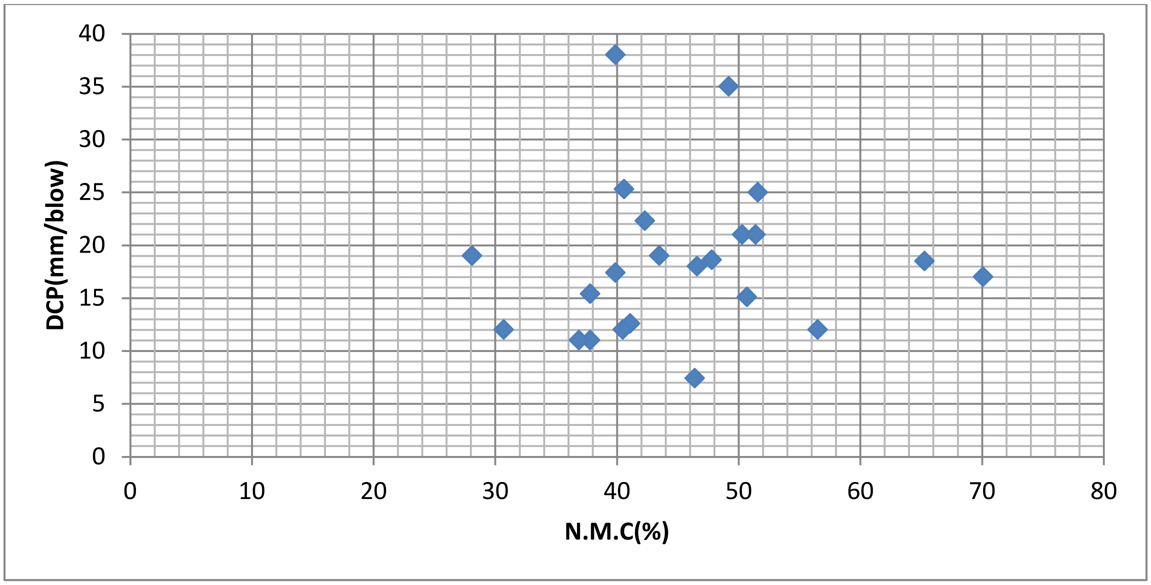

Figure 4-10 Scatter plot of DCP with natural moisture content for clay soils of Jimma

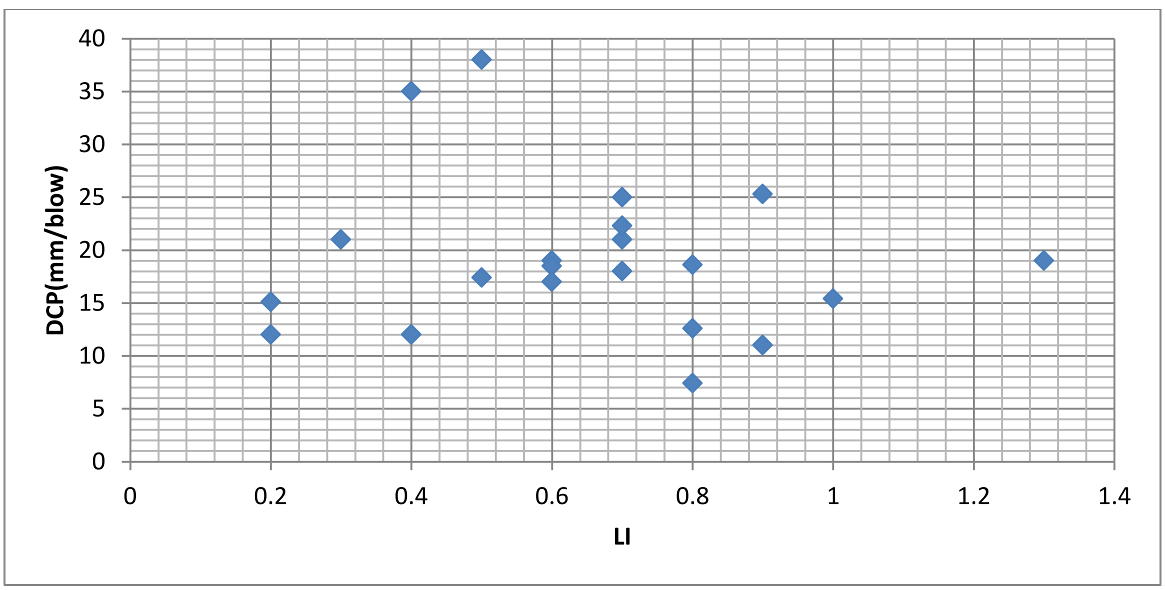

Figure 4-11 Scatter plot of DCP with liquidity index for clay soils of Jimma

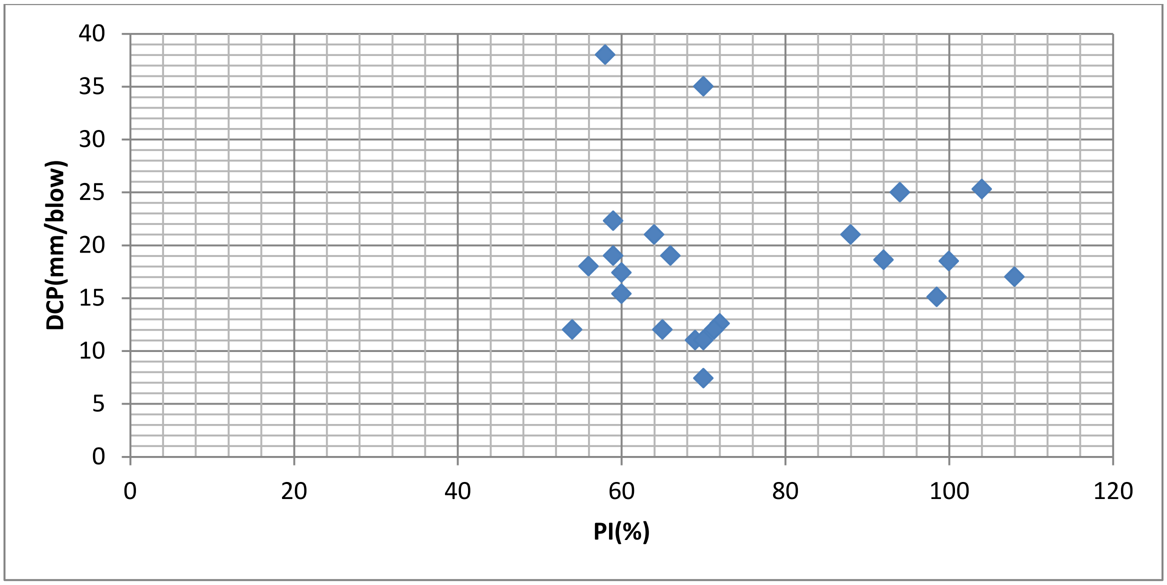

Figure 4-12 Scatter plot of DCP with plasticity index for clay soils of Jimma

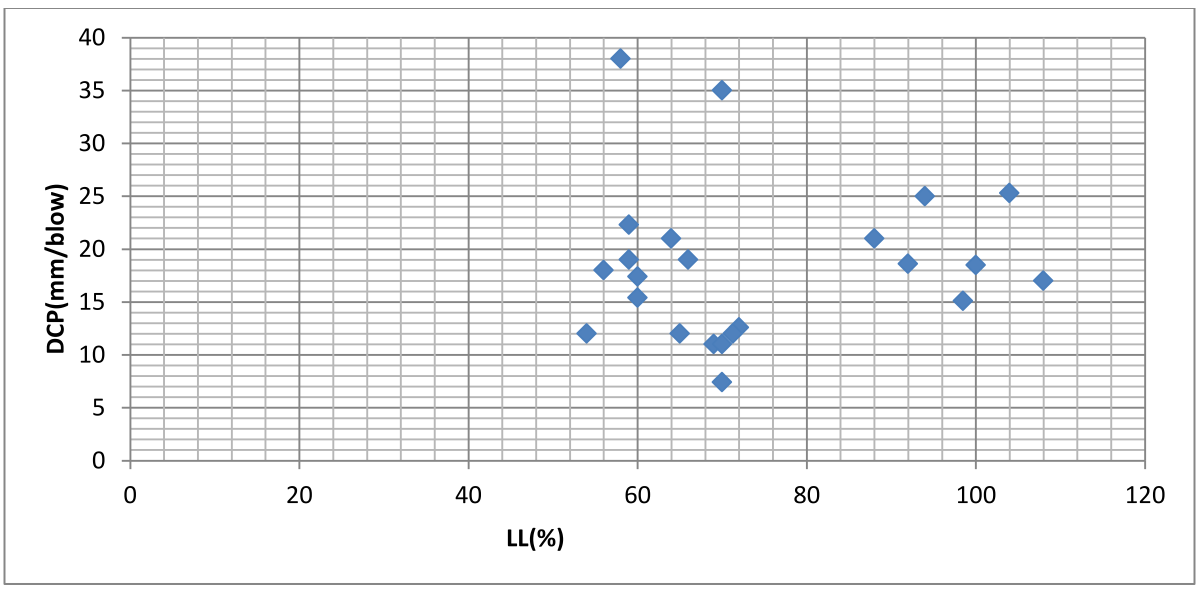

Figure 4-13 Scatter plot of DCP with liquid limit for clay soils of Jimma

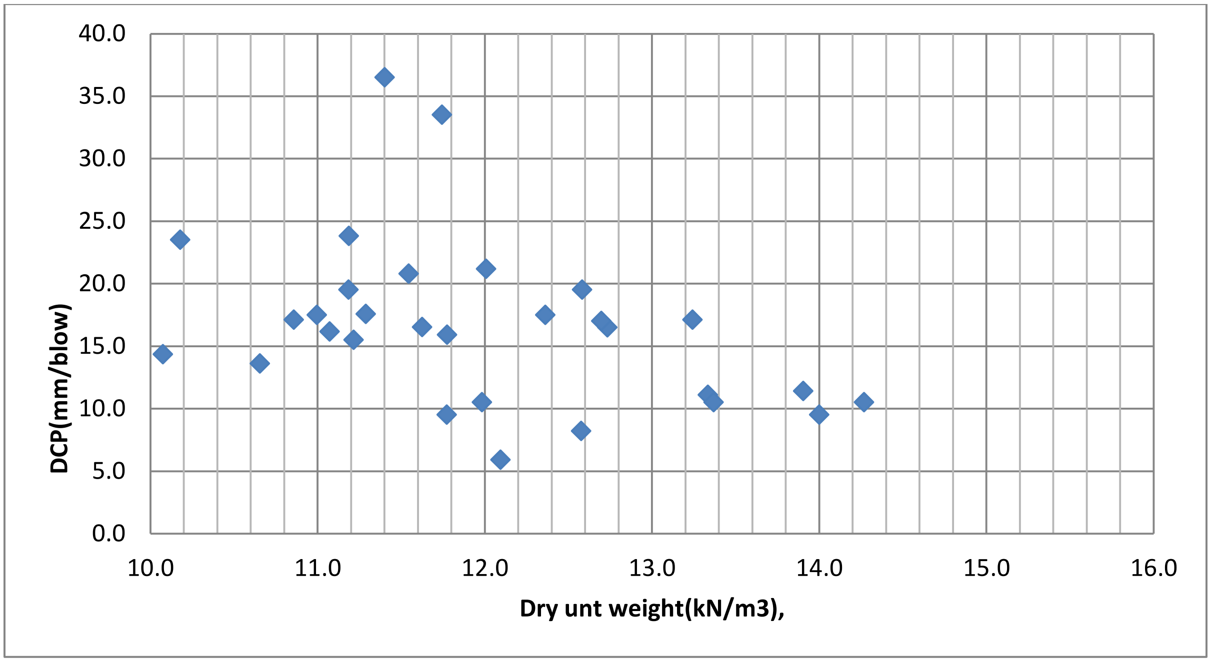

Figure 4-14 Scatter plot of DCP with dry unit weight for clay soils

4.5.1 Scatter Plot for Part-2 (Red Clay Soils of Jimma) from figure 4-15-4-26

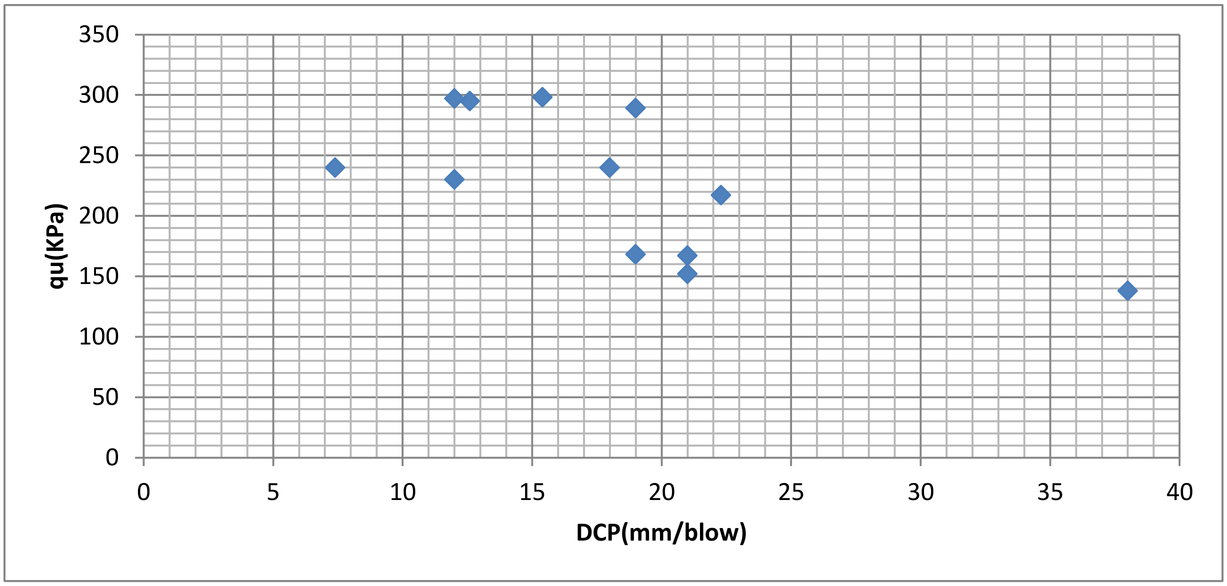

Figure 4-15 Scatter plot of qu with DCP for red clay soils of Jimma

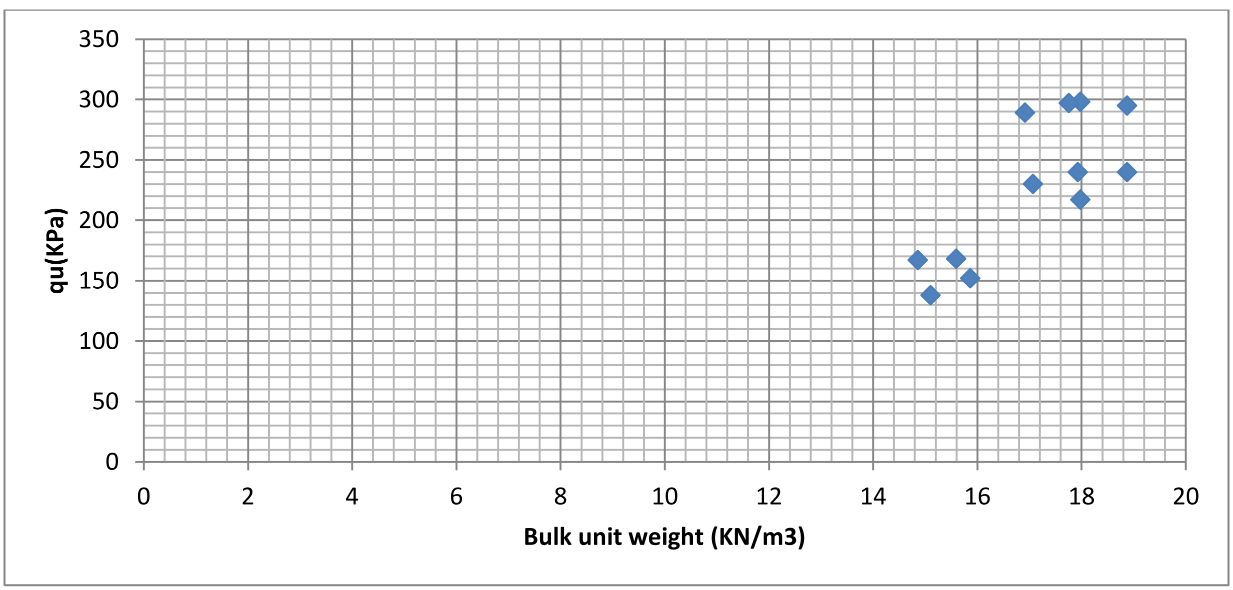

Figure 4-16 Scatter plot of qu with Bulk unit weight for red clay soils of Jimma

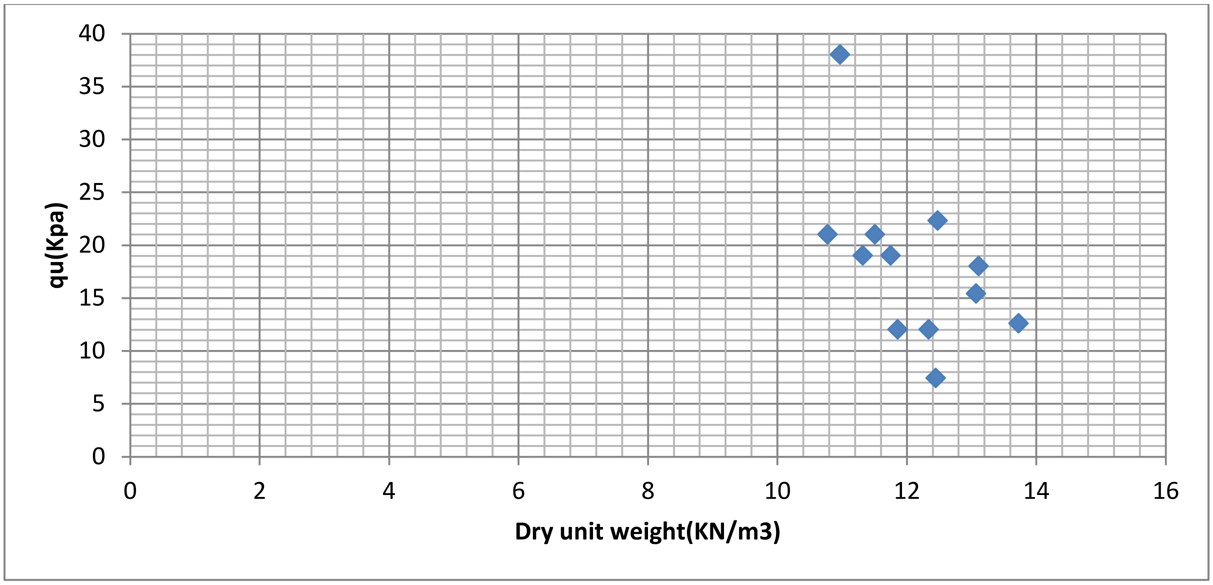

Figure 4-17 Scatter plot of qu with Dry unit weight for red clay soils of Jimma

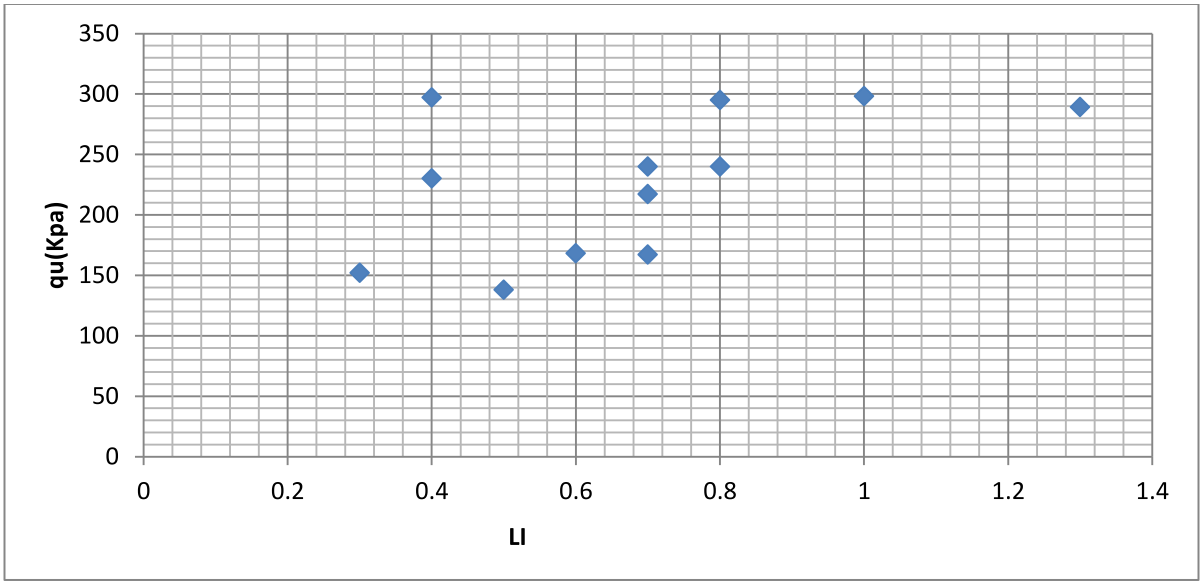

Figure 4-18 Scatter plot of qu with Liquidity index for red clay soils of Jimma

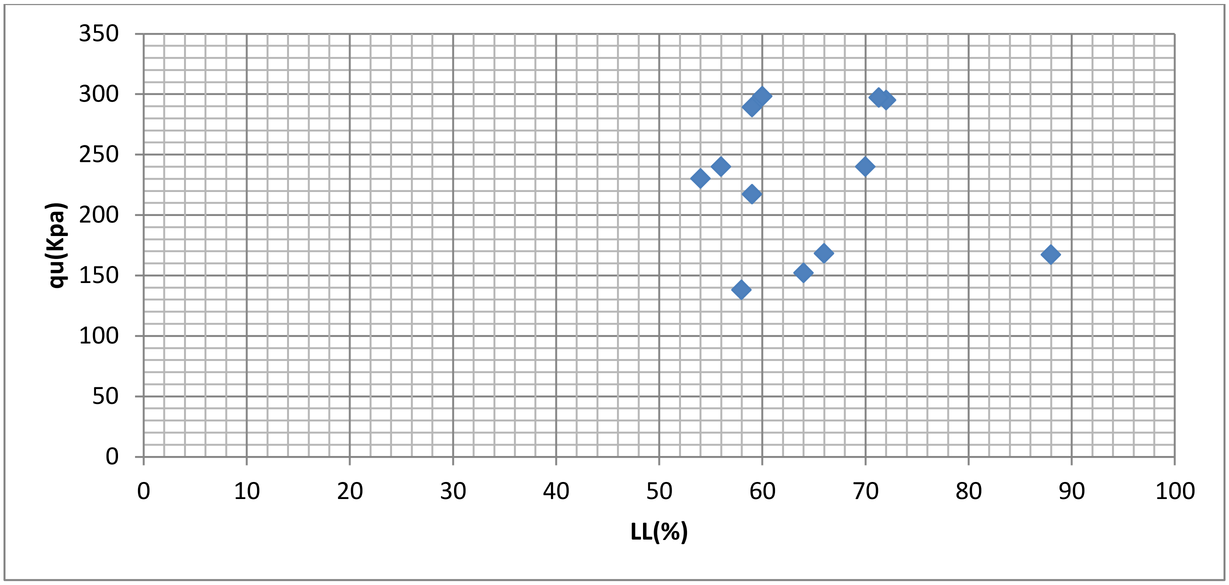

Figure 4-19 Scatter plot of qu with Liquid limit for red clay soils of Jimma

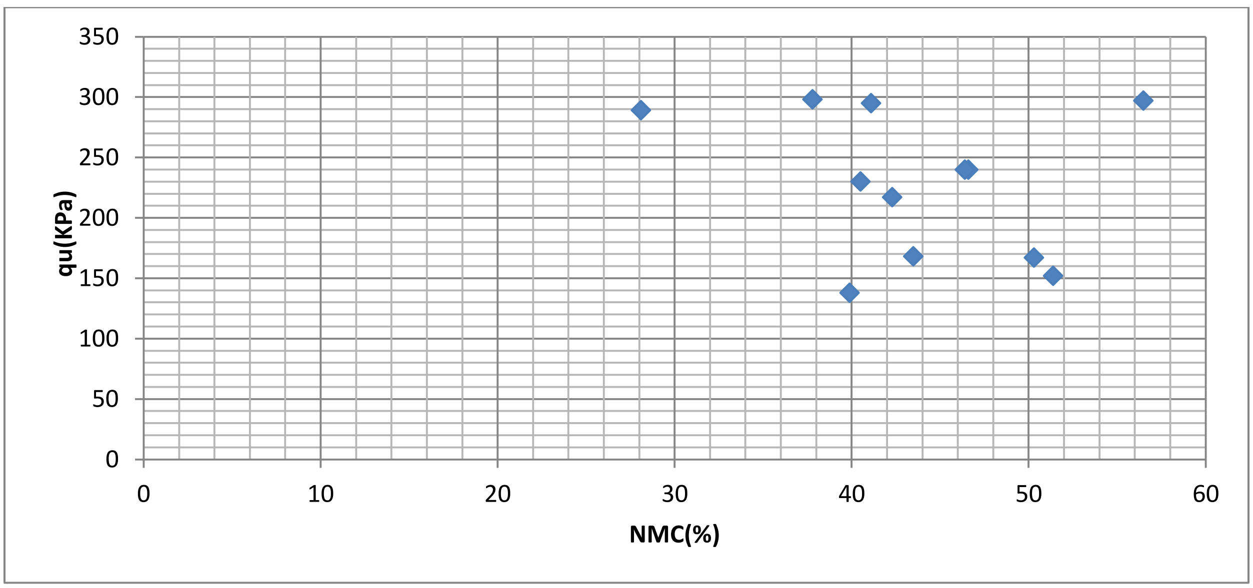

Figure 4-20 Scatter plot of qu with N.M.C red clay soils of Jimma

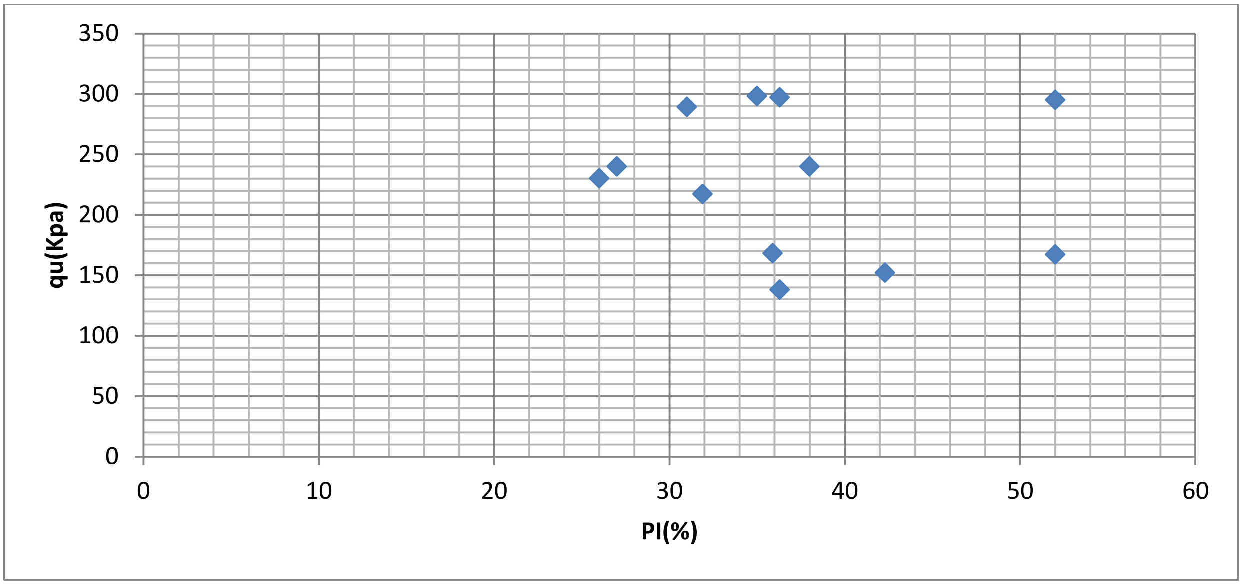

Figure 4-21 Scatter plot of qu with plastic Index for red clay soils of Jimma

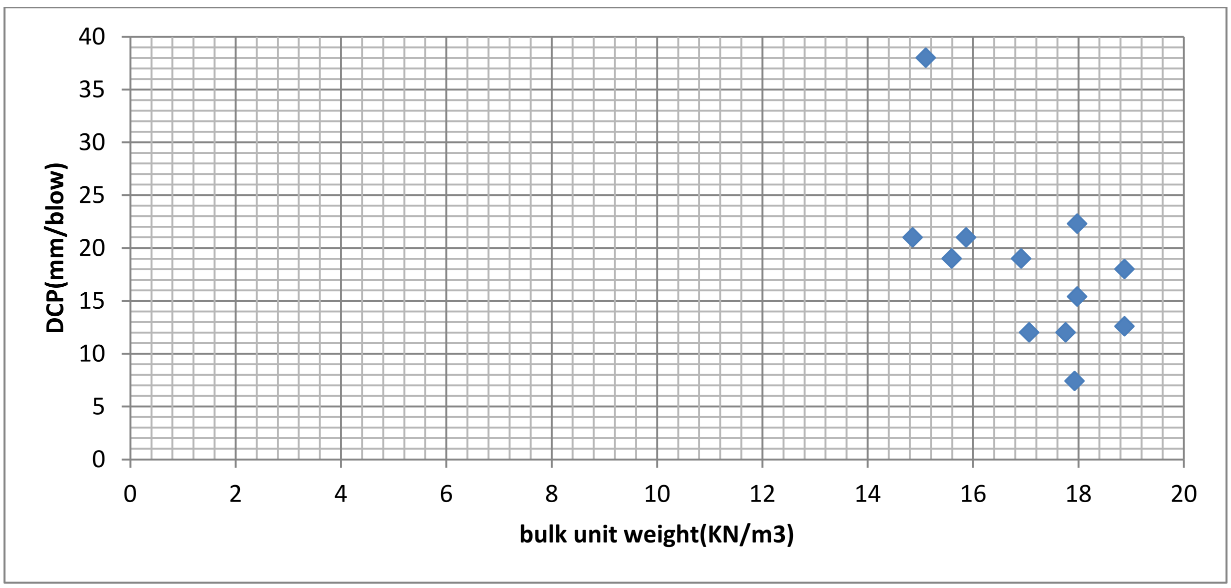

Figure 4-22 Scatter plot of DCP with Bulk unit weight for red clay soils of Jimma

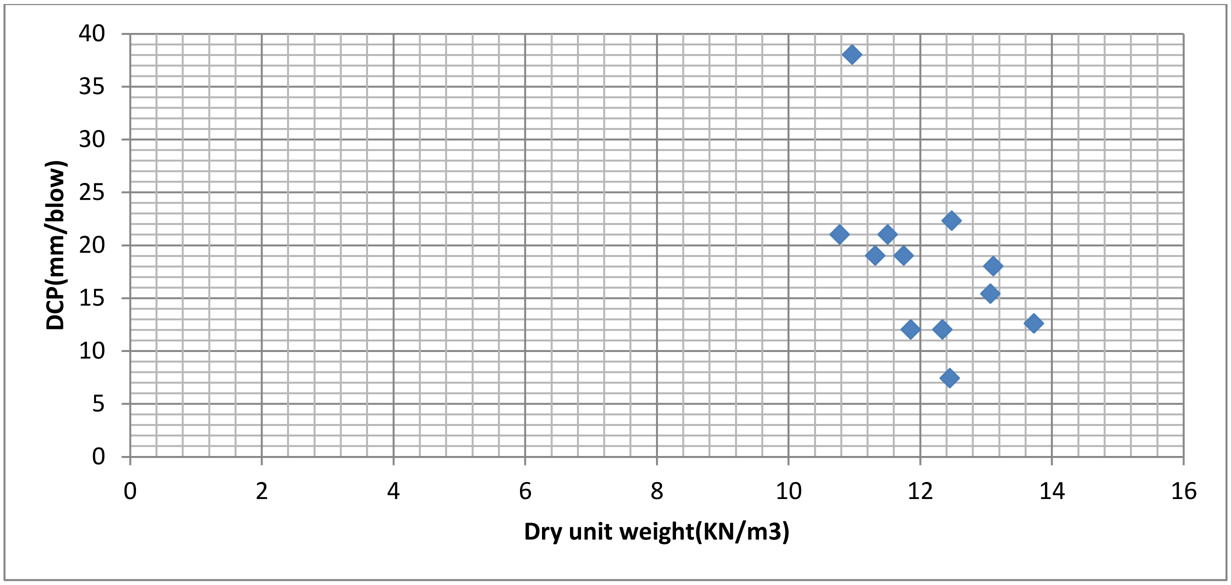

Figure 4-23 Scatter plot of DCP with Dry unit weight for red clay soils of Jimma

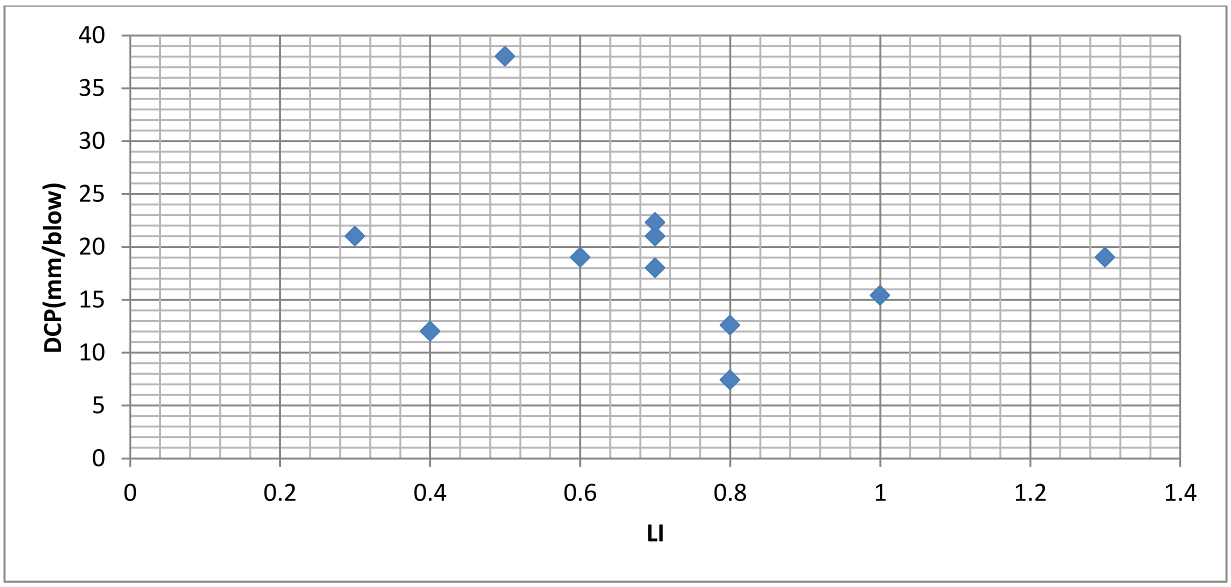

Figure 4-24 Scatter plot of DCP with Liquidity index for red clay soils of Jimma

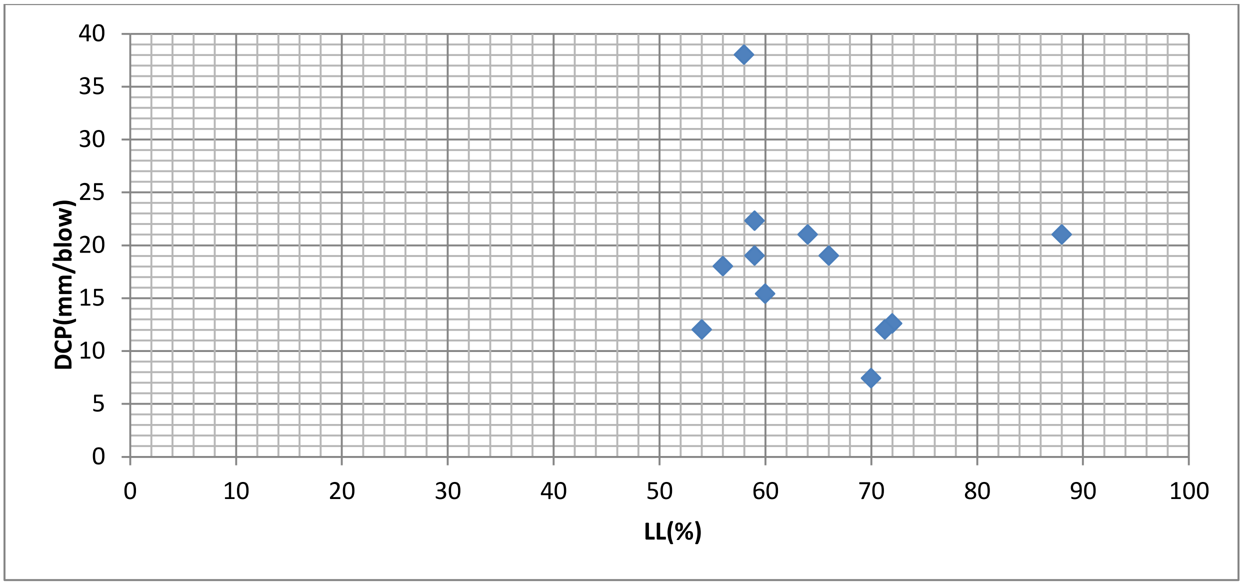

Figure 4-25 Scatter plot of DCP with Liquid limit for red clay soils of Jimma

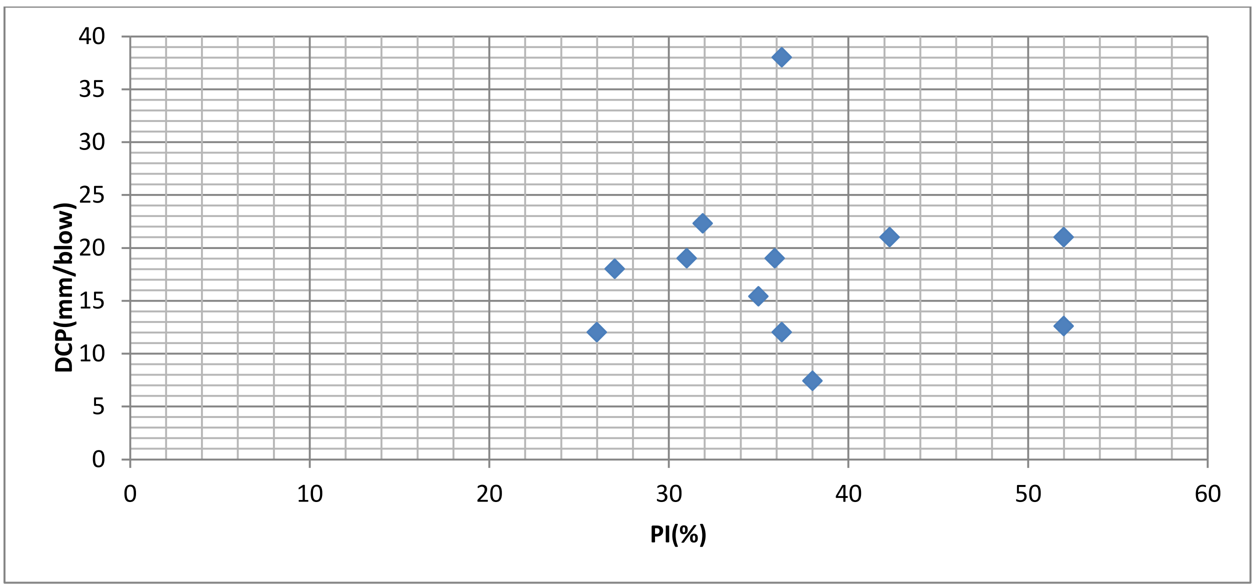

Figure 4-26 Scatter plot of DCP with plastic index for red clay soils of Jimma

4.5.2 Summary of Correlations for Clayey Soils of Jimma (Part one)

After carefully studying the data trend on the scatter plot from Figure 4-2 to Figure 4-14 and applying different models, correlations were developed for this part. The summery of the correlations is presented in Tables 4.7

Table 4-7 Summery of Correlations for (Clay Soils of Jimma)

EQUATION	R^2	Sample Size
$DCP=11.033e^{0.0084LL}$	0.105	21
$DCP=3.6625PI^{0.434}$	0.2648	21
$DCP=-11.857Li^2+13.491Li+15.445$	0.029	21
$DCP=-0.0095NMC^2+0.9952NMC-6.2177$	0.038	21
$DCP=1.7768\gamma_{dry}^2-46.74\ \gamma_{dry}+320.15$	0.4468	21
$DCP=0.7991\gamma_{bulk}^2-29.348\ \gamma_{bulk}-282.82$	0.3812	21
$qu=0.3224DCP^2-20.975DCP+457.04$	0.6305	20
$qu=4899.6LL^{0.765}$	0.3834	20
$qu=0.0562PI^2-7.8342PI+400.67$	0.6314	20
$qu=147.85Li^2-153.81Li+224.63$	0.0705	20
$qu=0.0923NMC^2+11.666NMC+527.8$	0.1856	20
$qu=0.0567\gamma_{dry}^{3.296}$	0.7213	20
$qu=0.0254\gamma_{bulk}^{3.1826}$	0.863	20

4.5.3 Summary of Correlations for red Clay Soils of Jimma (Part Two)

Table 4-8 Summery of Correlations for (Red Clay Soils of Jimma)

EQUATION	R^2	Sample Size
$qu=0.0119DCP^2-5.63DCP+325.23$	0.433	12
$qu=572.39\ \mathbf{ln}\ \boldsymbol{\gamma}\mathbf{bulk}-1394.7$	0.641	12
$qu=0.1489\gamma dry^{2.93}$	0.613	12
$qu=1123.1NMC^{-0.43}$	0.0767	12
$qu=7.19\ln(LI)+260.5$	0.25	12
$qu=-44.23\ln(PI)+386.3$	0.026	12
$qu=442.38LL^{-0.17}$	0.001	12
$DCP=63751\gamma bulk^{-2.91}$	0.350	12
$DCP=79.98NMC^{-0.4}$	0.032	12
$DCP=3.1\gamma dry^2-80.1\ \gamma dry+532.1$	0.410	12
$DCP=5913.6-\boldsymbol{LI^2}13.2LI+23.9$	0.026	12
$DCP=0.02PI^2+1.65PI-13.2$	0.037	12

4.6. Multiple Regressions

4.6.1. General

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

$y = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + \ldots + \beta_k x_k + \varepsilon$………………………………………………………………… (4.1)

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

The multiple regressions were conducted clay soils after incorporating data inside of Jimma is used for parametric study on dynamic cone penetration.

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

4.6.2. Multiple Regressions for Clay Soils of Jimma (Part One)

Developed equation for multiple regression of qu(kPa) with DCP(mm/blow), γbulk (kN/m3), γdry(kN/m 3), NMC(%),LI(%), PI(%), LL(%)and d (m), for clay soils of Jimma, with N=20 and adjusted R^2 =0.847 is:

$qu = -1454.1 - 449LL - 2.23PI - 168.4\,\gamma_{dry} + 1250.7\,\gamma_{bulk} - 16.23NMC + 44.4LI - 184.46Gs - 2.44DCP$…….(4.2)

Developed equation for multiple regressions of DCP (mm/blow) with PI (%), LL (%) Gs, NMC(%) $\gamma_{bulk(KN/m3)}$ and $\gamma_{dry(KN/m3)}$ for clay soils of Jimma, with N=21 and adjusted R^2 =0.7 is:

$DCP = 199.272 + 4.793Gs - 0.448NMC - 0.691LL + 0.541PI + 21.428LI - 7.06\gamma_{bulk} + 8.046\gamma_{dry}$……...(4.3)

4.6.3 Multiple Regressions for red Clay Soils of Jimma (part Two)

For Multiple regression of qu(KPa) with DCP(mm/blow), γbulk (kN/m3), γdry(kN/m3),PI(%), and LL(%) for red clay soil of Jimma with N=12 and the adjusted $\boldsymbol{R^2}$=0.69 is:

qu=306.88-1.69DCP-2.14PI+1.7LL+34.861 γdry+6.55 γbulk……………………………….(4.4)

Developed equation for multiple regressions of DCP (mm/blow) with PI (%), LL (%),LI, γbulk γdry and NMC for red clay soils of Jimma, with N=12 and adjusted R^2=0.64 is:

DCP=118.7+0.92PI-1.25LL-4.53 γdry-2.2 γbulk+15.6LI+0.64NMC…………………...(4.5)

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 γ_{bulk}, Liquidity index; PI, LL,and NMC in to equation between qu and DCP will improve the prediction of shear strength. If the above mentioned parameters are available, it is advisable to use the multiple regression developed equation 4.2 for clay soils of Jimma.

4.7. Discussion

4.7.1 Clay soils of jimma (Part one)

4.7.1.1. Single Regression

After carefully studying the data trend on the scatter plot and applying different models, this Part revealed that unconfined compression strength is influenced by dynamic cone penetration , Specific gravity, Plasticity index, bulk unit weight and dry unit weight by achieving coefficient of determination of 63%, 48.94%, 63%, 86.3% and 72.13%, respectively.

The dynamic cone penetration index influenced by undrained shear strength, bulk unit weight and dry unit weight by achieving coefficient of determination of 63%, 44.68%and 38.12% respectively.

This Part also revealed that correlation of qu with Liquidity index, natural moisture content, and liquid limit gave poor to fair result. While correlation of DCP with, natural moisture content, liquidity index, plasticity index and liquid limit gave a poor result. The summery of the correlations is presented in Tables 4-7.

4.7.1.2. Multiple Regressions

The multiple regression of qu with DCP, γ_{bulk}, γ_{dry}, NMC, LI, PI,and LL indicated that qu has fair correlation with the parameters by achieving adjusted coefficient of determination of 84.7% with 20 samples.

The multiple regression of DCP with γ_{bulk}, γ_{dry}, NMC, LI, PI, and LL indicated that DCP has good correlation by achieving adjusted coefficient of determination of 70% with 21 samples.

4.7.2 Red clay soils of Jimma (part two)

4.7.2.1 Single Regression

This part revealed that unconfined compression strength is significantly influenced by dynamic cone penetration index, bulk unit weight and dry unit weight by achieving coefficient of determination of 43%, 64% and 61%, respectively

The dynamic cone penetration is also significantly influenced by dry unit weight and bulk unit weight in addition to unconfined compression strength by achieving coefficient of determination of 40% dry unit weight , 35% for bulk unit weight; and 43% for unconfined compression strength.

This part also revealed that correlation of qu with bulk unit weight and dry unit weight in this part gave a fair result and correlation of qu with NMC, plasticity index and liquid limit gave a poor result. While correlation of DCP with dry unit weight and bulk unit weight gave a fair result and correlation of DCP with NMC and plasticity index gave a poor result. The summery of the correlations is presented in Tables 4-8.

4.7.2.2 Multiple Regressions

The multiple regression of qu with DCP, γbulk, γdry, NMC, LI, PI and LL indicated that qu has good correlation with the parameters by achieving adjusted coefficient of determination of 69% with 12 samples (refer to Equation 4.4).

4.8. Development of equation based on bearing capacity theory

Some good correlations between undrained shear strength and the DCP are obtained (refer to Table 4-7). It is tempting to develop a bearing capacity equation from the correlations developed.

Converting unconfined compression strength into cohesion (i.e., c=qu/2), as discussed in the literature review section 2.5, 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 4-7 one can observe a correlation between qu (kPa) and DCP (mm/blow) gave:

- $qu=0.3224DCP^2-20.975DCP+457.04$……………………………………………..(4.6)
  with R^2=63.05% and N=20 for clay soils of Jimma

Converting equations 4.6 into cohesion, the corresponding relation will be, C=qu/2

- $C=0.1612DCP^2 10.4875DCP+228.52$,…………………………………………...(4.7)
  For clay soils of Jimma

After inserting equations 4.7 into equation 2.18, the corresponding relations of bearing

Capacity equations for initial loading condition will be: $q_{ult} = 5.142c + \gamma h$,

- $q_{ult} = 0.829DCP^2-53.927DCP+ \gamma h +1175.049$……………………………………(4.8)
  For clay soils of Jimma

From Table 4-8 one can observe a correlation between qu (kPa) and DCP (mm/blow) gave

- $qu=0.0119DCP^2 5.63DCP+325.23$……………………………………………….(4.9)
  With R2 of 43%. And N=12 for red clay soil of Jimma from this the corresponding bearing capacity equation is
- $q_{ult}=0.028DCP^2 14.5DCP+835.84+\gamma h$……………………………………………(4.10)

4.10 Validation of the Developed Equation.

These control test results were obtained from the current study at Merkato 2-1 and merkato 2-2. The validation of the developed correlation is conducted by using two known test results which follows similar testing procedures with the subject research. Current Study Model qu=0.3224DCP2-20.975DCP+457.04.Using the control test results and the developed correlation equation, the predicted qu is determined so as to compare it with the actual qu value as shown in Table 4.9

Table 4.9: Validation of the developed correlation of Current Study

Sample Location	**Depth(m)**	**DCP (mm/blows)**	**Actual qu value**	**Developed qu value**	**Variation (%)**
Merkato 2-1	1.5	35	112	117.9	5
Merkato 2-2	2.5	25	121	134.2	9.84

From the above performed result it was determined that a significant correlation can be developed between among the actual and predicted qu value.

4.10.1 Comparison of findings and current study on parametric study

Table 4.10 Comparison of findings of past studies with the current study on parametric study

Past study (Anteneh Getachew, November, 2012)	Current Study
Unconfined compressive strength, liquidity index and depth are the important parameters influencing the DCP	Unconfined compressive strength, bulk unit weight and Dry unit weight are the important parameters influencing the DCP
Bulk unit weight and natural moisture content are the next important parameters	Specific gravity, depth and Plasticity Index are the next important parameters
Dry unit weight has little effect while plasticity index and liquid limit have no influence	other parameters has little effect

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

Based on the analysis of data obtained from laboratory soil testing and field test the following Conclusions and Recommendations are drawn

5.1 Conclusion:

The main objective of this thesis was to obtain valid relationships between Dynamic cone penetration and undraind shear strength of Jimma clay soil. However, as additional information, qu has a very strong correlation with Bulk unit weight, Dry unit weight and plasticity index by achieving coefficient of determination of 86.3%, 72%,and 63% respectively .Therefore, unconfined compression strength can be computed from known value of Bulk unit weight, Dry unit weight and plasticity index by the correlation equations.

- ***Part-1 (Clay Soils of Jimma)*** revealed that DCP is influenced by qu , for this category ,qu can be estimated from DCP by qu=0.3224DCP^2-20.975DCP+457.04, R^2=63.05% and N=20 The corresponding bearing capacity equation of qult = 0.829DCP^2-53.927DCP+ γh +1175.049
- ***Part-2 (Red Clay Soils Jimma)*** revealed that DCP is influenced by qu, for this category, qu, can be estimated from DCP by qu=0.0119DCP^2-5.63DCP+325.23 with R2 of 43%. The corresponding bearing capacity equation of qult=0.028DCP^2-14.5DCP +835.84+γh

From parametric study of clay soil of Jimma the dry unit weight and bulk unit weight have satisfactory correlation with Dynamic cone penetration by achieving coefficient of determination of 44.7% and 38%, while parameters like Liquid Limit, plasticity index, N.M.C, and liquidity index have little influence on the DCP.

From parametric study of red clay soil of Jimma the dry unit weight and bulk unit weight have satisfactory correlation with Dynamic cone penetration by achieving coefficient of determination of 41% and 35%, while parameters like Liquid Limit, plasticity index, N.M.C, and liquidity index have little influence on the DCP of red clay soil of Jimma.

5.2 Recommendation

The exposure encountered in trying to conduct the current research has revealed areas where further efforts may be proved in the future. Following are some of the recommendations in relation to the subject study:

1. It is recommended to carry out this correlation with a large number of samples including geographical areas in Jimma which are not covered by this research.

2. It is also recommended to carry out such a study in other parts of Ethiopia

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APPENDIX

APPENDIX – A: Field Tests Result

Table A - 1.1 Penetration Data Report for Furdisa

Test Depth (m): 2.5m.   Cone Angle (deg.): 60

SN.n	number of blows	cumulative blows	depth of penetration(mm)	DCP	mm/blows
			90		0
1	1	1	140	50	49.5
2	1	2	225	85	85
3	1	3	272	47	47
4	1	4	317	45	45
5	1	5	365	48	48
6	1	6	397	32	32
7	1	7	435	38	38
8	1	8	463	28	28
9	1	9	488	25	25
10	1	10	516	28	28
11	1	11	547	31	31
12	1	12	586	39	39
13	1	13	638	52	52
14	1	14	682	44	44
15	1	15	728	46	46
16	1	16	770	42	42
17	1	17	834	64	64
18	1	18	898	64	64

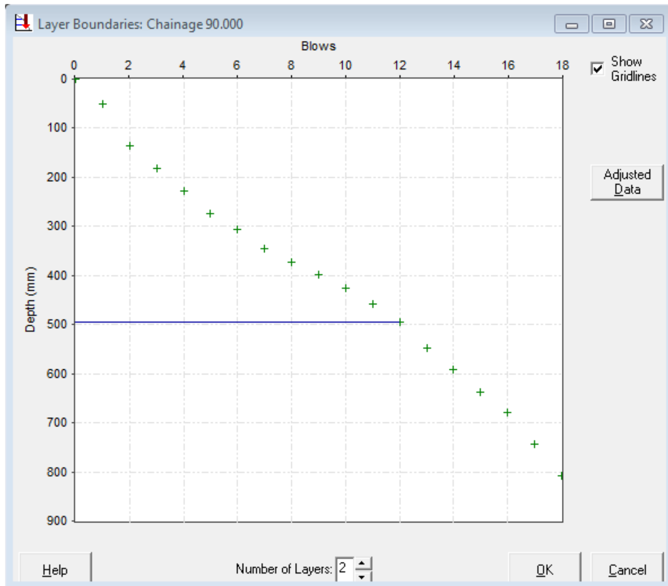


Table A - 1.2 Penetration Data Report for Furdisa

Test Depth (m): 1.5m. Cone Angle (deg.): 60

SN. n	number of blows	cumulative blows	depth of penetration(mm)	DCP	penetration rate
		0	75		0
1	1	1	135	135	134.5
2	1	2	190	55	55
3	1	3	241	51	51
4	1	4	290	49	49
5	1	5	334	44	44
6	1	6	367	33	33
7	1	7	440	73	73
8	1	8	450	10	10
9	1	9	480	30	30
10	1	10	510	30	30
11	1	11	547	37	37
12	1	12	594	47	47
13	1	13	634	40	40

14	1	14	670	36	36
15	1	15	710	40	40
16	1	16	740	30	30
17	1	17	770	30	30
18	1	18	798	28	28
19	1	19	830	32	32
20	1	20	860	30	30

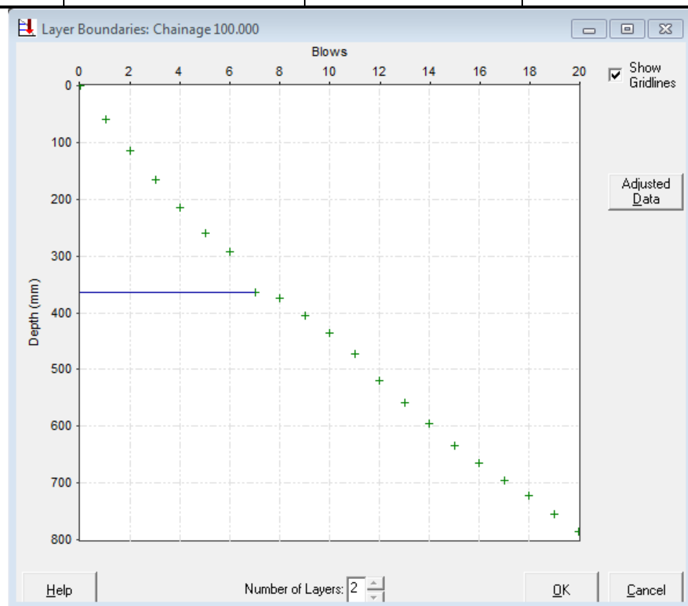


Table A - 1.3 Penetration Data Report for Furstale

Test Depth (m): 2.5m. Cone Angle (deg.): 60

SN.n	number of blows	cumulative blows	depth of penetration(mm)	DCP	penetration rate
	0	0	85		0
1	1	1	155	155	154.5
2	1	2	220	65	65
3	1	3	290	70	70
4	1	4	365	75	75
5	1	5	440	75	75
6	1	6	510	70	70
7	1	7	570	60	60
8	1	8	640	70	70
9	1	9	685	45	45
10	1	10	753	68	68
11	1	11	813	60	60
12	1	12	815	2	2
13	1	13	860	45	45
14	1	14	882	22	22

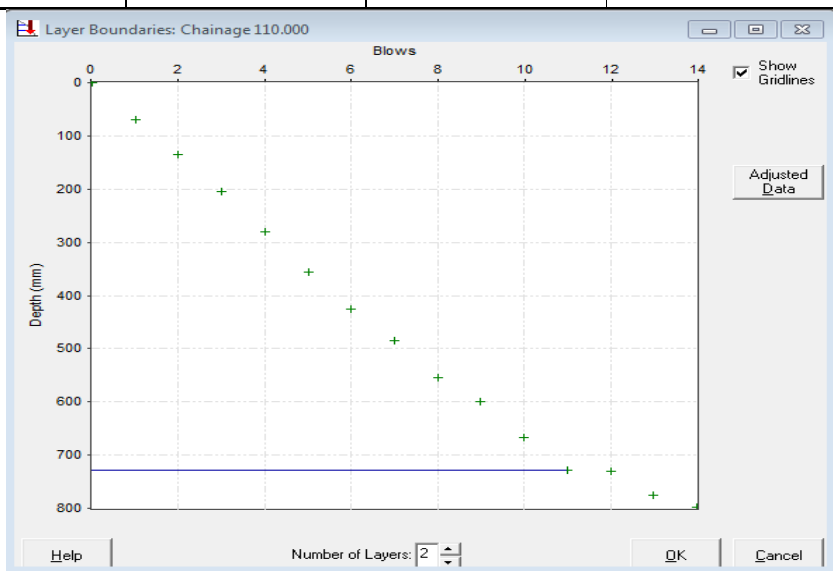


Table A - 1.4 Penetration Data Report for Furstale

Test Depth (m): 1.5m.   Cone Angle (deg.): 60

SN.n	number of blows	cumulative blows	depth of penetration(mm)	DCP	penetration rate
	0	0	50		0
1	1	1	110	110	109.5
2	1	2	155	45	45
3	1	3	190	35	35
4	1	4	218	28	28
5	1	5	240	22	22
6	1	6	275	35	35
7	1	7	327	52	52
8	1	8	337	10	10
9	1	9	365	28	28
10	1	10	390	25	25
11	1	11	420	30	30
12	1	12	450	30	30
13	1	13	480	30	30
14	1	14	510	30	30
15	1	15	545	35	35
16	1	16	570	25	25
17	1	17	600	30	30
18	1	18	630	30	30
19	1	19	660	30	30
20	1	20	684	24	24
21	1	21	710	26	26
22	1	22	744	34	34
23	1	23	770	26	26
24	1	24	800	30	30

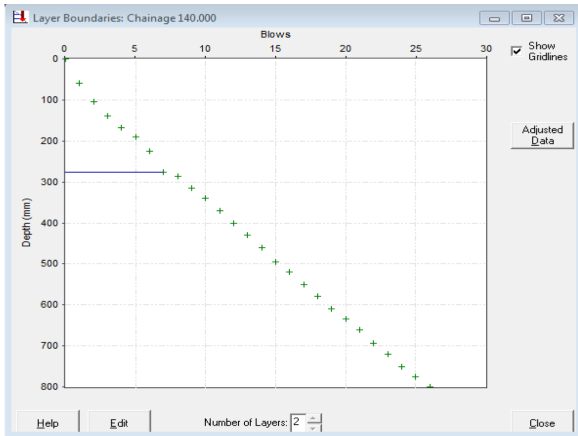


Table A - 1.5 Penetration Data Report for Agriculture

Test Depth (m): 1.5m.   Cone Angle (deg.): 60

SN.N	number of blows	cumulative blows	depth of penetration(mm)	DCP	penetration rate
			65		0
1	1	1	110	110	109.5
2	1	2	150	40	40
3	1	3	190	40	40
4	1	4	240	50	50
5	1	5	300	60	60
6	1	6	340	40	40
7	1	7	370	30	30
8	1	8	440	70	70
9	1	9	455	15	15
10	1	10	510	55	55
11	1	11	555	45	45
12	1	12	585	30	30
13	1	13	612	27	27
14	1	14	645	33	33
15	1	15	675	30	30
16	1	16	704	29	29
17	1	17	740	36	36

18	1	18	780	40	40
19	1	19	820	40	40
20	1	20	850	30	30
21	1	21	880	30	30

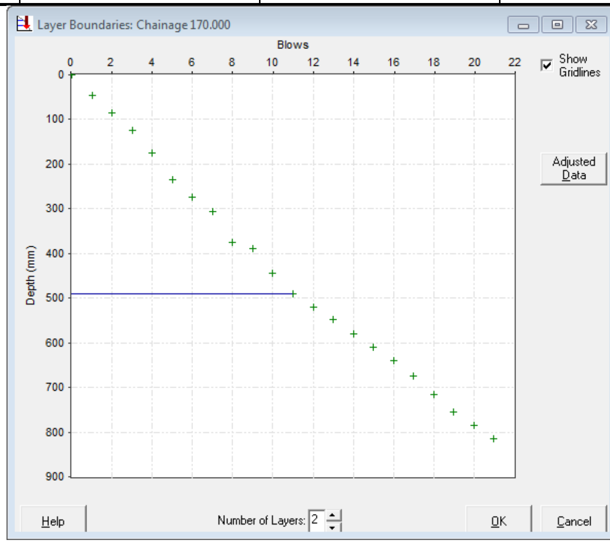


Table A - 1.6 Penetration Data Report for merkato

Test Depth (m): 1.5m. Cone Angle (deg.): 60

SN.n	number of blows	cumulative blows	depth of penetration(mm)	DCP	penetration rate
	0	0	190		
1	1	1	260	260	259.5
2	1	2	320	60	60
	1	3	366	46	46
3	1	4	404	38	38
4	1	5	435	31	31
5	1	6	470	35	35
6	1	7	495	25	25
7	1	8	520	25	25
8	1	9	540	20	20
9	1	10	565	25	25
10	1	11	585	20	20

11	1	12	605	20	20
12	1	13	630	25	25
13	1	14	655	25	25
14	1	15	680	25	25
15	1	16	710	30	30
16	1	17	739	29	29
17	1	18	771	32	32
18	1	19	801	30	30
19	1	20	830	29	29

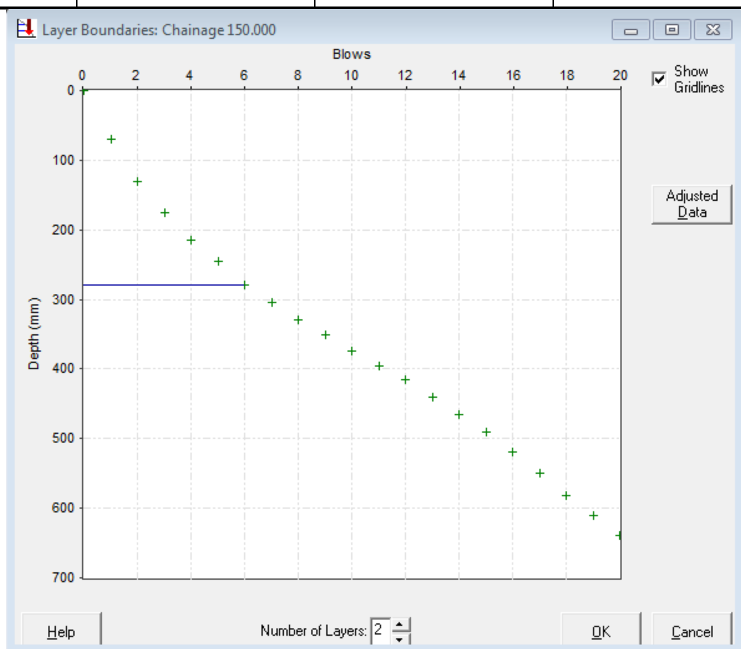


Table A - 1.7 Penetration Data Report for merkato

Test Depth (m): 2.5m. Cone Angle (deg.): 60

SN.n	number of blows	cumulative blows	depth of penetration(mm)	DCP	penetration rate
			50		
1	1	1	132	82	82
2	1	2	181	49	49
3	1	3	221	40	40
4	1	4	272	51	51
5	1	5	310	38	38
6	1	6	347	37	37
7	1	7	395	48	48
8	1	8	440	45	45
9	1	9	485	45	45
10	1	10	550	65	65
11	1	11	590	40	40
12	1	12	620	30	30
13	1	13	650	30	30
14	1	14	680	30	30
15	1	15	710	30	30
16	1	16	740	30	30
17	1	17	770	30	30
18	1	18	800	30	30
19	1	19	830	30	30
20	1	20	830	0	0

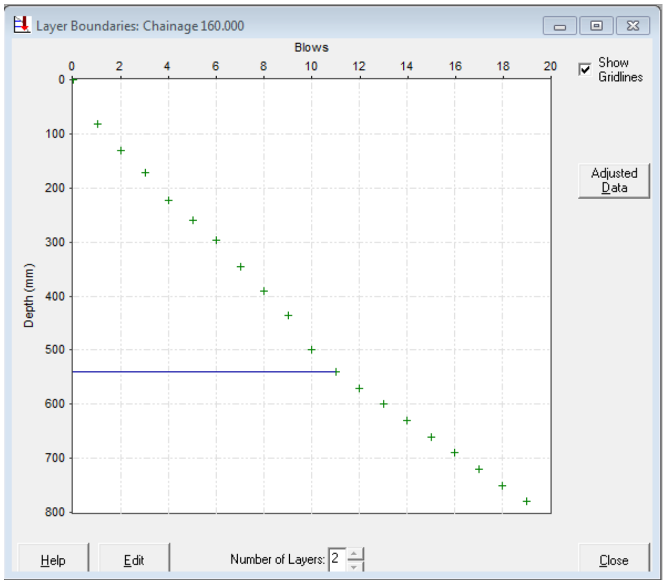


Table A - 1.8 Penetration Data Report for main cumpus

Test Depth (m): 1.5m.   Cone Angle (deg.): 60

SN num.	number of blows	cumulative blows	depth of penetration(mm)	DCP	penetration rate
		0	20		
1	2	2	45	45	22.0
2	2	4	80	35	17.5
3	2	6	110	30	15.0
4	2	8	160	50	25.0
5	2	10	195	35	17.5
6	2	12	230	35	17.5
7	2	14	260	30	15.0
8	2	16	295	35	17.5
9	2	18	330	35	17.5
10	2	20	360	30	15.0
11	2	22	390	30	15.0
12	2	24	430	40	20.0
13	2	26	470	40	20.0
14	2	28	505	35	17.5

15	2	30	550	45	22.5
16	2	32	590	40	20.0
17	2	34	630	40	20.0
18	2	36	660	30	15.0
19	2	38	690	30	15.0
20	2	40	725	35	17.5
21	2	42	770	45	22.5
22	2	44	815	45	22.5
23	2	46	865	50	25.0

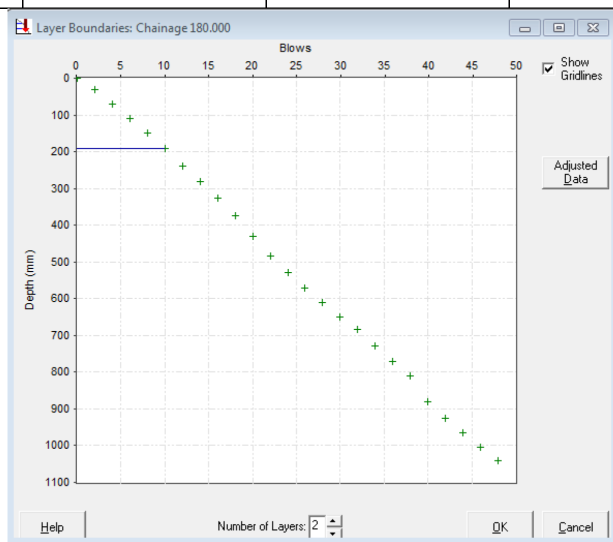


Table A - 1.9 Penetration Data Report for main cumpus

Test Depth (m): 2.5m. Cone Angle (deg.): 60

SN.N	number of blows	cumulative blows	depth of penetration(mm)	DCP	penetration rate
	0	0	110		
1	2	2	166	166	82.50
2	2	4	260	94	47.00
3	2	6	340	80	40.00
4	2	8	420	80	40.00
5	2	10	490	70	35.00
6	2	12	560	70	35.00

7	2	12	630	70	35.00
8	2	16	680	70	35.00
9	3	19	760	80	26.67
10	3	22	840	80	26.67
11	3	25	915	75	25.00
12	3	28	995	80	26.67
13	3	31	1095	100	33.33
14	3	34	1160	65	21.67
15	3	37	1230	70	23.33
16	3	40	1295	65	21.67
17	3	43	1360	65	21.67
18	3	46	1435	75	25.00
19	3	49	1500	65	21.67

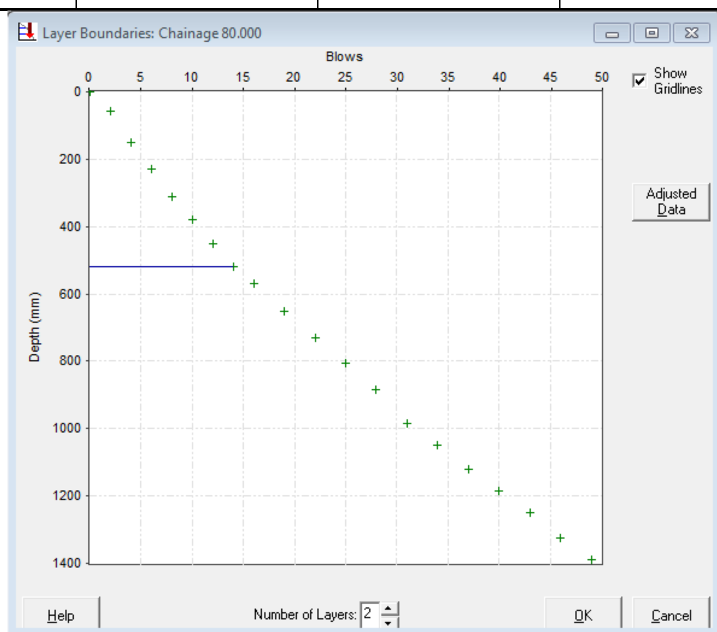


Table A - 1.10 Penetration Data Report for kebele-5

Test Depth (m): 1.5m.   Cone Angle (deg.): 60

SN n.	number of blows	cumulative blows	depth of penetration(mm)	DCPI	penetration rate
		0	20		
1	2	2	50	50	24.5
2	2	4	90	40	20.0
3	2	6	130	40	20.0
4	2	8	170	40	20.0
5	2	10	210	40	20.0
6	2	12	258	48	24.0
7	2	12	300	42	21.0
8	2	16	345	45	22.5
9	2	18	395	50	25.0
10	2	21	450	55	27.5
11	2	22	505	55	27.5
12	2	24	550	45	22.5
13	2	26	590	40	20.0
14	2	28	630	40	20.0
15	2	30	670	40	20.0
16	2	32	705	35	17.5
17	2	34	750	45	22.5
18	2	36	790	40	20.0
19	2	38	830	40	20.0
20	2	40	900	70	35.0
21	2	42	945	45	22.5
22	2	44	985	40	20.0
23	2	46	1025	40	20.0

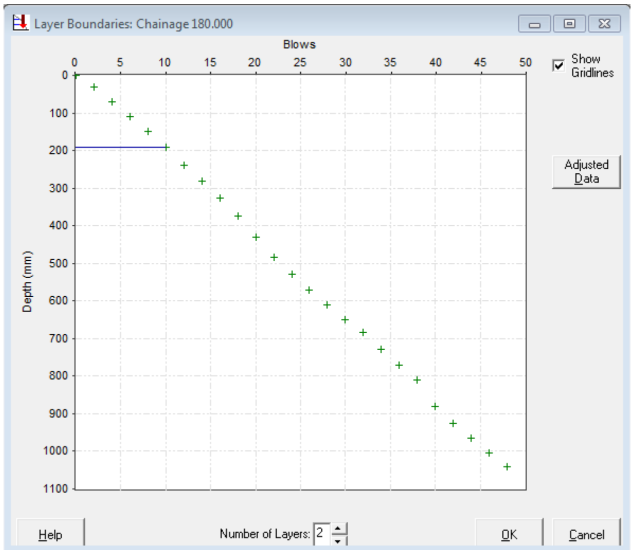


Table A - 1.11 Penetration Data Report for Bocho bore

Test Depth (m): 2.5m. Cone Angle (deg.): 60

SN num.	number of blows	cumulative blows	depth of penetration(mm)	DCPI	penetration rate
		0	30		
1	2	2	70	40	34.50
2	2	4	120	50	25.00
3	2	6	170	50	25.00
4	2	8	210	40	20.00
5	2	10	245	35	17.50
6	2	12	280	35	17.50
7	2	14	325	45	22.50
8	2	16	360	35	17.50
9	2	18	393	33	16.50
10	2	20	420	27	13.50
11	2	22	445	25	12.50
12	2	24	475	30	15.00
13	2	26	500	25	12.50

14	2	28	524	24	12.00
15	2	30	545	21	10.50
16	2	32	574	29	14.50
17	2	34	600	26	13.00
18	2	36	625	25	12.50
19	2	38	650	25	12.50
20	2	40	675	25	12.50
21	2	42	700	25	12.50
22	2	44	725	25	12.50
23	2	46	755	30	15.00

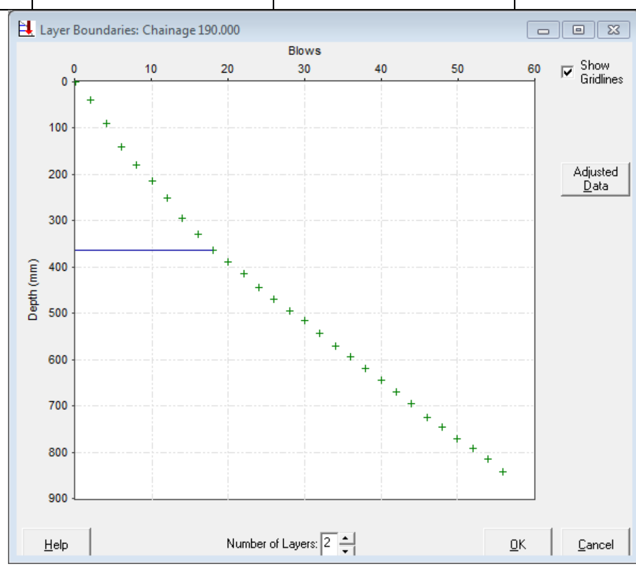


Table A - 1.12 Penetration Data Report for Bocho bore

Test Depth (m): 1.5m. Cone Angle (deg.): 60

SN num.	number of blows	cumulaltive blows	depth of penetration(mm)	DCPI	penatration rate
1		0	15		
2	4	4	85	85	21.25
3	4	8	155	70	17.50
4	4	12	225	70	17.50
5	4	16	285	60	15.00
6	4	20	360	75	18.75

7	4	24	425	65	16.25
8	4	14	485	60	15.00
9	4	32	555	70	17.50
10	4	36	610	55	13.75
11	4	20	680	70	17.50
12	4	44	775	95	23.75
13	6	50	815	40	6.67
14	6	56	905	90	15.00
15	6	62	990	85	14.17
16	6	68	1080	90	15.00
17	6	74	1160	80	13.33
18	6	80	1215	55	9.17
19	6	86	1315	100	16.67
20	6	92	1395	80	13.33
21	6	98	1495	100	16.67

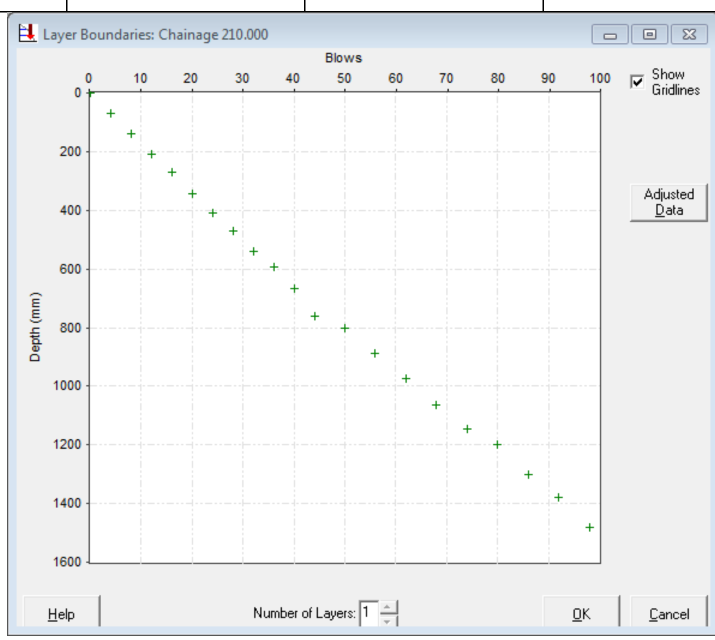


Table A - 1.13 Penetration Data Report for Technic

Test Depth (m): 2.5m. Cone Angle (deg.): 60

SN num.	number of blows	cumulaltive blows	depth of penetration(mm)	DCPI	penatration rate
1		0	50	50	22.00
2	2	2	115	65	32.50
3	4	6	245	130	32.50
4	4	10	305	60	15.00
5	4	14	365	60	15.00
6	4	18	415	50	12.50
7	4	14	470	55	13.75
8	4	26	540	70	17.50
9	4	30	595	55	13.75
10	4	20	650	55	13.75
11	4	38	695	45	11.25
12	4	42	740	45	11.25
13	4	46	795	55	13.75

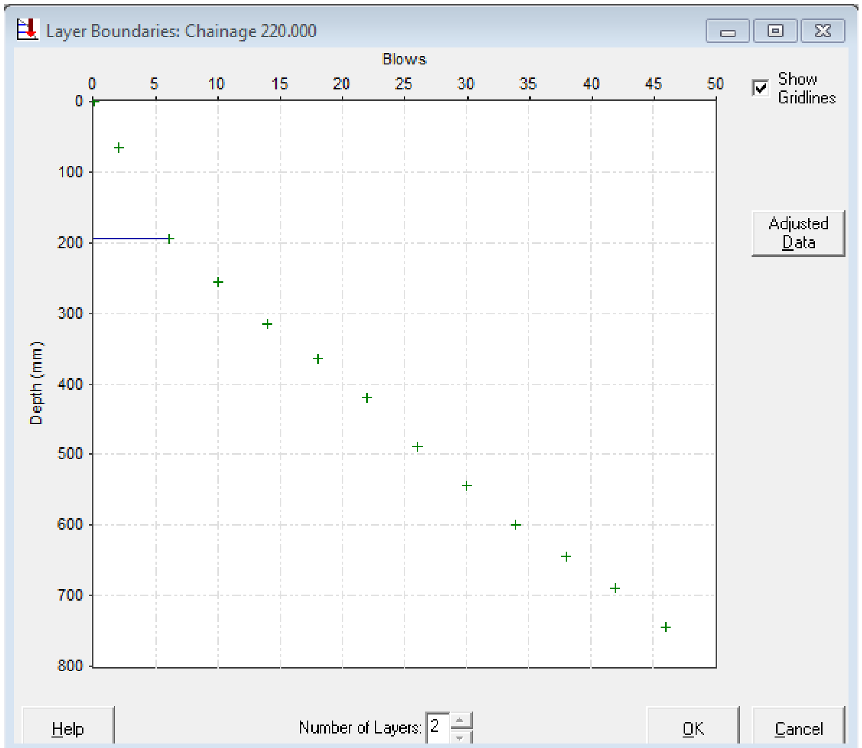


Table A - 1.14 Penetration Data Report for Awetu

Test Depth (m): 1.5m.   Cone Angle (deg.): 60

SN num.	number of blows	cumulative blows	depth of penetration(mm)	DCPI	penetration rate
1		0	15		
2	2	2	19	19	9.0
3	2	4	305	286	143.0
4	2	6	380	75	37.5
5	2	8	460	80	40.0
6	2	10	550	90	45.0
7	2	12	640	90	45.0
8	2	14	725	85	42.5
9	2	16	810	85	42.5
10	2	18	880	70	35.0
11	2	20	945	65	32.5
12	2	22	1010	65	32.5

13	2	24	1070	60	30.0
14	2	26	1120	50	25.0
15	2	28	1180	60	30.0
16	2	30	1225	45	22.5
17	2	32	1275	50	25.0
18	2	34	1325	50	25.0
19	2	36	1380	55	27.5
20	2	38	1430	50	25.0
21	2	40	1475	45	22.5

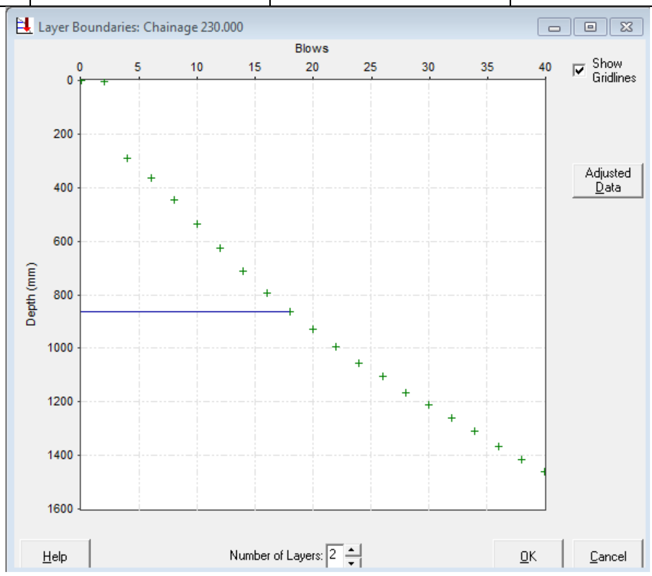


Table A - 1.15 Penetration Data Report for kochi

Test Depth (m): 1.5m. Cone Angle (deg.): 60

SN num.	number of blows	cumulative blows	depth of penetration(mm)	DCPI	penetration rate
1		0	150		
2	2	2	575	575	287
3	2	4	750	175	87.5
4	1	5	860	110	110
5	2	7	925	65	32.5
6	2	9	980	55	27.5

7	2	11	1005	25	12.5
8	2	14	1045	40	20
9	2	15	1065	20	10
10	2	17	1090	25	12.5
11	2	20	1110	20	10
12	2	21	1135	25	12.5
13	2	23	1165	30	15
14	2	25	1195	30	15
15	2	27	1225	30	15
16	2	29	1255	30	15
17	2	31	1295	40	20
18	2	33	1320	25	12.5
19	2	35	1350	30	15
20	2	37	1390	40	20
21	2	39	1435	45	22.5
22	2	41	1475	40	20

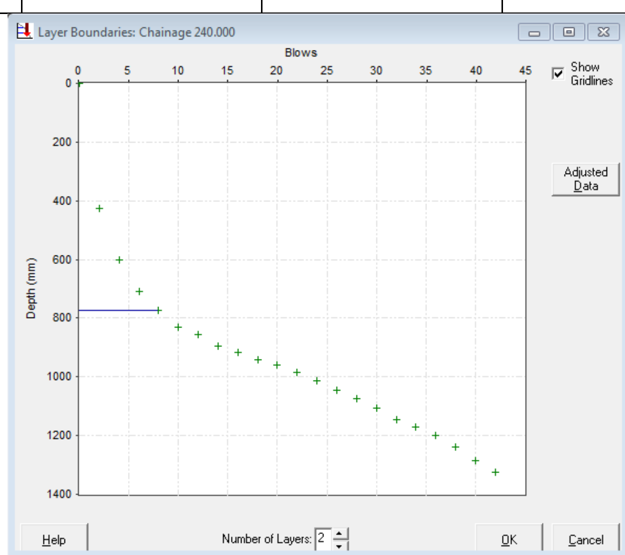
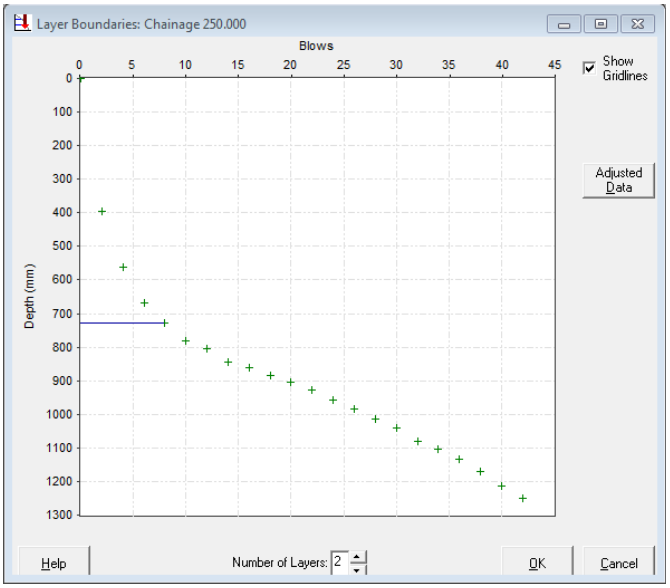


Table A - 1.16 Penetration Data Report for Mercator-2

Test Depth (m): 1.5m.   Cone Angle (deg.): 60

SN num.	number of blows	cumulative blows	depth of penetration(mm)	DCPI	penetration rate
1		0	150		
2	2	2	546	546	287
3	2	4	713	166.25	83.125
4	1	5	817	104.5	104.5
5	2	7	879	61.75	30.875
6	2	9	931	52.25	26.125
7	2	11	955	23.75	11.875
8	2	14	993	38	19
9	2	15	1012	19	9.5
10	2	17	1036	23.75	11.875
11	2	20	1055	19	9.5
12	2	21	1078	23.75	11.875
13	2	23	1107	28.5	14.25
14	2	25	1135	28.5	14.25
15	2	27	1164	28.5	14.25
16	2	29	1192	28.5	14.25
17	2	31	1230	38	19
18	2	33	1254	23.75	11.875
19	2	35	1283	28.5	14.25
20	2	37	1321	38	19
21	2	39	1363	42.75	21.375
22	2	41	1401	38	19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Blows
0 5 10 15 20 25 30 35 40 45
Show Gridlines
Adjusted Data
0 100 200 300 400 500 600 700 800 900 1000 1100 1200 1300
Depth (mm)
Help
Number of Layers: 2
OK
Cancel

APPENDIX – B: Laboratory Test Results

B – 1) Moisture Content Determination

Table B - 1.1 Moisture Content for Furdissa at 1m depth

mass of container+ wet soil	3.68
mass of container+ dry soil	2.735
mass of container	0.235
N.M.C(W%)	37.8

Table B - 1.2 Moisture Content for Furdissa at 1.5m depth

mass of container+ wet soil	3.72
mass of container+ dry soil	2.705
mass of container	0.235
N.M.C(W%)	41.09

Table B - 1.3 Moisture Content for Furdissa at 2.5m depth

mass of soil container +wet soil	3.54
mass of soil container +dry soil	2.63
mass container	0.225
N.M.C(W%)	37.84

Table B - 1.4 Moisture Content for Furstale at 1.5m depth

mass of container+ wet soil	1.42
mass of container+ dry soil	1.19
mass of container	0.44
N.M.C(W%)	30.67

Table B - 1.5 Moisture Content for Furstale at 2.5m depth

mass of container+ wet soil	1.88
mass of container+ dry soil	1.5
mass of container	0.47
N.M.C(W%)	36.89

Table B - 1.6 Moisture Content for Agriculture at 1.5m depth

mass of container +wet soil	2.33
mass of container +dry soil	1.745
mass of container	0.71
N.M.C(W%)	56.52

Table B - 1.7 Moisture Content for Agriculture at 2.5m depth

mass of container +wet soil	3.425
mass of container +dry soil	2.81
mass of container	0.625
N.M.C(W%)	28.15

Table B - 1.8 Moisture Content for Merkato at 1.5m depth

mass of soil container +wet soil	3.69
mass of soil container +dry soil	2.53
mass container	0.225
N.M.C(W%)	50.33

Table B - 1.9 Moisture Content for Merkato at 2.5m depth

mass of soil container +wet soil	3.49
mass of soil container +dry soil	2.5
mass container	0.225
N.M.C(W%)	43.52

Table B - 1.10 Moisture Content for Merkato-2 at 1.5m depth

mass of soil container +wet soil	3.59
mass of soil container +dry soil	2.48
mass container	0.225
N.M.C(W%)	49.22

Table B - 1.11 Moisture Content for Merkato-2 at 2.5m depth

mass of soil container +wet soil	3.69
mass of soil container +dry soil	2.51
mass container	0.225
N.M.C(W%)	51.64

Table B - 1.12 Moisture Content for Awetu at 1.5m depth

mass of soil container +wet soil	3.79
mass of soil container +dry soil	2.73
mass container	0.225
N.M.C(W%)	42.32

Table B - 1.13 Moisture Content for Awetu at 2.5m depth

mass of soil container +wet soil	3.78
mass of soil container +dry soil	2.65
mass container	0.225
N.M.C(W%)	46.59794

B - 1.14 Moisture Content for Teknic at 1.5m depth

mass of soil container +wet soil	3.62
mass of soil container +dry soil	2.64
mass container	0.225
N.M.C(W%)	40.58

Table B - 1.15 Moisture Content for Teknic at 2.5m depth

mass of soil container +wet soil	3.72
mass of soil container +dry soil	2.59
mass container	0.225
N.M.C(W%)	47.78

Table B - 1.16 Moisture Content for Kebele-5 at 1.5m depth

mass of soil container +wet soil	3.92
mass of soil container +dry soil	2.46
mass container	0.225
N.M.C(W%)	65.32

Table B - 1.17 Moisture Content for Kebele-5 at 2.5m depth

mass of soil container +wet soil	3.99
mass of soil container +dry soil	2.439
mass container	0.225
N.M.C(W%)	70.05

Table B - 1.18 Moisture Content for main campus at 1.5m depth

mass of soil container +wet soil	3.59
mass of soil container +dry soil	2.62
mass container	0.225
N.M.C(W%)	40.50

Table B - 1.19 Moisture Content for main campus at 2.5m depth

mass of soil container +wet soil	3.79
mass of soil container +dry soil	2.66
mass container	0.225
N.M.C(W%)	46.41

Table B - 1.20 Moisture Content for Bocho bore at 1.5m depth

mass of soil container +wet soil	3.59
mass of soil container +dry soil	2.62
mass container	0.225
N.M.C(W%)	40.50

Table B - 1.21 Moisture Content for Bocho bore at 2.5m depth

mass of soil container +wet soil	3.79
mass of soil container +dry soil	2.59
mass container	0.225
N.M.C (W %)	50.74

Table B - 1.22 Moisture Content for Kochi at 1.5m depth

mass of soil container +wet soil	3.59
mass of soil container +dry soil	2.63
mass container	0.225
N.M.C (W %)	39.92

Table B - 1.23 Moisture Content for Kochi at 2.5m depth

mass of soil container +wet soil	3.79
mass of soil container +dry soil	2.58
mass container	0.225
N.M.C (W %)	51.38

B – 2) Bulk Density Determination

Table B - 2.2 Bulk Density for Furstale

Sample from Furstale			
Depth(m)	1.5m	Depth(m)	2.5m
Weight of (Wt) jar sand after	7533	Weight of (Wt) jar	7910
weight of mold + sand before	5360	weight of mold + sand	5628
Mass (wt) of the specimen(g)	2173	Mass (wt) of the specimen(g)	2282
volume of extracted soil	1425	volume of extracted soil	1496
Wt of can	435	Wt of can	456.75
Wt of can + Wt of moist Soil(g)	2950	Wt of can + Wt of moist Soil(g)	3097.5
Wt of can + Wt of dry Soil(g)	2109	Wt of can + Wt of dry Soil(g)	2214.45
Wt of Dry Soil(g)	1674	Wt of Dry Soil(g)	1757.7
Wt of pore water(g)	841	Wt of pore water(g)	883.05
Water content(%)	50.24	Water content(%)	41.9
Bulk Density(g/cm3)	1.731	Bulk Density(g/cm3)	1.818
Dry Density(g/cm3)	1.154	Dry Density(g/cm3)	1.212

Table B - 2.3 Bulk Densities for Agriculture

Sample from Agri			
Depth(m)	1.5m	Depth(m)	2.5m
Weight of (Wt) jar+ sand after	7375	Weight of (Wt) jar	7743
weight of mold + sand before	5270	weight of mold + sand	5534
Mass (wt) of the specimen(g)	2105	Mass (wt) of the specimen(g)	2210
volume of extracted soil	1380	volume of extracted soil	1449
Wt of can	425	Wt of can	446.25
Wt of can + Wt of moist Soil(g)	2805	Wt of can + Wt of moist Soil(g)	2945.25
Wt of can + Wt of dry Soil(g)	2075	Wt of can + Wt of dry Soil(g)	2178.75
Wt of Dry Soil(g)	1650	Wt of Dry Soil(g)	1732.5
Wt of pore water(g)	730	Wt of pore water(g)	766.5
Water content (%)	44.24	Water content (%)	36.9
Bulk Density(g/cm3)	1.692	Bulk Density(g/cm3)	1.776
Dry Density(g/cm3)	1.175	Dry Density(g/cm3)	1.234

B - 2.4Bulk Density for Merkato

Sample from merkato-1			
Depth(m)	1.5m	Depth(m)	2.5m
Weight of (Wt) jar+sand after	7928	Weight of (Wt) jar	8325
weight of mold + sand before	4590	weight of mold + sand	4820
Mass (wt) of the specimen(g)	3338	Mass (wt) of the specimen(g)	3505
volume of extracted soil	2189	volume of extracted soil	2298
Wt of can	225	Wt of can	236.25
Wt of can + Wt of moist Soil(g)	3540	Wt of can + Wt of moist Soil(g)	3717
Wt of can + Wt of dry Soil(g)	2630	Wt of can + Wt of dry Soil(g)	2761.5
Wt of Dry Soil(g)	2405	Wt of Dry Soil(g)	2525.25
Wt of pore water(g)	910	Wt of pore water(g)	955.5
Water content (%)	37.84	Water content(%)	31.5
Bulk Density(g/cm3)	1.486	Bulk Density(g/cm3)	1.56
Dry Density(g/cm3)	1.078	Dry Density(g/cm3)	1.132

Table B - 2.5 Bulk Densities for Awetu

Sample from Awetu			
Depth(m)	1.5m	Depth(m)	2.5m
Weight of (Wt) jar+sand after	7034	Weight of (Wt) jar	7385
weight of mold + sand before	5112	weight of mold + sand	5368
Mass (wt) of the specimen(g)	1922	Mass (wt) of the specimen(g)	2018
volume of extracted soil	1260	volume of extracted soil	1323
Wt of can	412	Wt of can	432.6
Wt of can + Wt of moist Soil(g)	2721	Wt of can + Wt of moist Soil(g)	2857.05
Wt of can + Wt of dry Soil(g)	2013	Wt of can + Wt of dry Soil(g)	2113.65
Wt of Dry Soil(g)	1601	Wt of Dry Soil(g)	1681.05
Wt of pore water(g)	708	Wt of pore water(g)	743.4
Water content(%)	44.22	Water content (%)	36.9
Bulk Density(g/cm3)	1.798	Bulk Density(g/cm3)	1.888
Dry Density(g/cm3)	1.248	Dry Density(g/cm3)	1.311

Table B - 2.6 Bulk Density for Main Campus

Sample from main campus			
Depth(m)	1.5m	Depth(m)	2.5m
Weight of (Wt) jar+ sand after	7034	Weight of (Wt) jar	7385
weight of mold + sand before	5112	weight of mold + sand	5368
Mass (wt) of the specimen(g)	1922	Mass (wt) of the specimen(g)	2018
volume of extracted soil	1260	volume of extracted soil	1323
Wt of can	392	Wt of can	411.6
Wt of can + Wt of moist Soil(g)	2585	Wt of can + Wt of moist Soil(g)	2714.25
Wt of can + Wt of dry Soil(g)	1912	Wt of can + Wt of dry Soil(g)	2007.6
Wt of Dry Soil(g)	1520	Wt of Dry Soil(g)	1596
Wt of pore water(g)	673	Wt of pore water(g)	706.65
Water content(%)	44.28	Water content (%)	36.9
Bulk Density(g/cm3)	1.707	Bulk Density(g/cm3)	1.793
Dry Density(g/cm3)	1.186	Dry Density(g/cm3)	1.245

Table B - 2.7 Bulk Density for Kebele-5

Sample from kebele 5			
Depth(m)	1.5m	Depth(m)	2.5m
Weight of (Wt) jar+sand after	7928	Weight of (Wt) jar	8325
weight of mold + sand before	4590	weight of mold + sand	4820
Mass (wt) of the specimen(g)	3338	Mass (wt) of the specimen(g)	3505
volume of extracted soil	2189	volume of extracted soil	2298
Wt of can	230	Wt of can	242
Wt of can + Wt of moist Soil(g)	3611	Wt of can + Wt of moist Soil(g)	3792
Wt of can + Wt of dry Soil(g)	2683	Wt of can + Wt of dry Soil(g)	2817
Wt of Dry Soil(g)	2453	Wt of Dry Soil(g)	2576
Wt of pore water(g)	928	Wt of pore water(g)	974
Water content(%)	37.83	Water content(%)	32
Bulk Density(g/cm3)	1.515	Bulk Density(g/cm3)	1.591
Dry Density(g/cm3)	1.1	Dry Density(g/cm3)	1.155

Table B - 2.8 Bulk Density for bocho bore

Sample from bocho Bore				
Depth(m)	1.5m	Depth(m)		2.5m
Weight of (Wt) jar+ sand after	7928	Weight of (Wt) jar		8325
weight of mold + sand before	4590	weight of mold + sand		4820
Mass (wt) of the specimen(g)	3338	Mass (wt) of the specimen(g)		3505
volume of extracted soil	2189	volume of extracted soil		2298
Wt of can	218	Wt of can		229
Wt of can + Wt of moist Soil(g)	3430	Wt of can + Wt of moist Soil(g)		3602
Wt of can + Wt of dry Soil(g)	2548	Wt of can + Wt of dry Soil(g)		2676
Wt of Dry Soil(g)	2330	Wt of Dry Soil(g)		2447
Wt of pore water(g)	881.79	Wt of pore water(g)		926
Water content(%)	37.84	Water content (%)		32
Bulk Density(g/cm3)	1.47	Bulk Density(g/cm3)		1.514
Dry Density(g/cm3)	1.08	Dry Density(g/cm3)		1.097

Table B - 2.9 Bulk Density for Technic

Sample from Technic				
Depth(m)	1.5m	Depth(m)		2.5m
Weight of (Wt) jar+sand after	7800	Weight of (Wt) jar		8190
weight of mold + sand before	4590	weight of mold + sand		4820
Mass (wt) of the specimen(g)	3210	Mass (wt) of the specimen(g)		3371
volume of extracted soil	2105	volume of extracted soil		2210
Wt of can	214	Wt of can		224
Wt of can + Wt of moist Soil(g)	3362	Wt of can + Wt of moist Soil(g)		3530
Wt of can + Wt of dry Soil(g)	2498	Wt of can + Wt of dry Soil(g)		2622
Wt of Dry Soil(g)	2284	Wt of Dry Soil(g)		2398
Wt of pore water(g)	864	Wt of pore water(g)		907
Water content (%)	37.84	Water content(%)		32
Bulk Density(g/cm3)	1.50	Bulk Dencity(g/cm3)		1.57
Dry Density(g/cm3)	1.01	Dry Dencity(g/cm3)		1.18

Table B - 2.10 Bulk Density for Kochi

Sample from Kochi			
Depth(m)	1.5m	Depth(m)	2.5m
Weight of (Wt) jar+ sand after	7800	Weight of (Wt) jar	8190
weight of mold + sand before	4590	weight of mold + sand	4820
Mass (wt) of the specimen(g)	3210	Mass (wt) of the specimen(g)	3371
volume of extracted soil	2105	volume of extracted soil	2210
Wt of can	220	Wt of can	231
Wt of can + Wt of moist Soil(g)	3463	Wt of can + Wt of moist Soil(g)	3636
Wt of can + Wt of dry Soil(g)	2572	Wt of can + Wt of dry Soil(g)	2701
Wt of Dry Soil(g)	2352	Wt of Dry Soil(g)	2470
Wt of pore water(g)	890	Wt of pore water(g)	935
Water content (%)	37.84	Water content(%)	32
Bulk Density(g/cm3)	1.54	Bulk Density(g/cm3)	1.56
Dry Density(g/cm3)	1.05	Dry Density(g/cm3)	1.15

Table B - 2.11 Bulk Density for Merkato-2

Sample from merkato 2			
Depth(m)	1.5m	Depth(m)	2.5m
Weight of (Wt) jar+sand after	7800	Weight of (Wt) jar	8190
weight of mold + sand before	4590	weight of mold + sand	4820
Mass (wt) of the specimen(g)	3210	Mass (wt) of the specimen(g)	3371
volume of extracted soil	2105	volume of extracted soil	2210
Wt of can	200	Wt of can	210
Wt of can + Wt of moist Soil(g)	3151	Wt of can + Wt of moist Soil(g)	3308
Wt of can + Wt of dry Soil(g)	2341	Wt of can + Wt of dry Soil(g)	2458
Wt of Dry Soil(g)	2141	Wt of Dry Soil(g)	2248
Wt of pore water(g)	810	Wt of pore water(g)	850
Water content(%)	37.84	Water content(%)	32
Bulk Density(g/cm3)	1.40	Bulk Density(g/cm3)	1.47
Dry Density(g/cm3)	1.32	Dry Density(g/cm3)	1.39

B – 3) Specific Gravity Determination

Table B-3.1 Specific Gravity for Furdisa at 1.5m depth

Trial	1	2
bottle code	1	2
mass of bottle	28.912	27.5105
mass of bottle +soil	38.772	37.705
mass of bottle +soil +water	85.852	84.796
mass of bottle +water	79.83	78.585
specific gravity	2.513826	2.639219
average Gs	2.576	

Table B-3.2 Specific Gravity for Furstale at 1.5m depth

Trial	1	2
bottle code	Tb	G
mass of B	28.808	27.772
mass of BS	38.813	37.772
mass of BSW	85.766	84.914
mass of BW	79.761	78.48
specific gravity(Gs)	2.503129	2.804262
average Gs	2.654	

Table B-3.3 Specific Gravity for Agriculture at 1.5m depth

Trial	1	2
bottle code	A	T48
mass of B	28.15	28.715
mass of BS	38.05	38.796
mass of BSW	85.93	86.047
mass of BW	79.68	79.738
specific gravity(Gs)	2.666667	2.709293
average Gs	2.688	

Table B-3.4 Specific Gravity for Merkato at 1.5m depth

Trial	1	2
bottle code	B	C
mass of B	28.15	28.715
mass of BS	38.15	38.796
mass of BSW	85.98	86.107
mass of BW	79.68	79.738
specific gravity(Gs)	2.702703	2.754062
average Gs	2.728	

Table B-3.5 Specific Gravity for Awetu at 1.5m depth

Trial	1	2
bottle code	B	C
mass of B	28.222	28.715
mass of BS	38.222	38.715
mass of BSW	85.99	86.157
mass of BW	79.68	79.738
specific gravity(Gs)	2.710027	2.792516
average Gs	2.751	

Table B-3.6 Specific Gravity for Main Campus at 1.5m depth

Trial	1	2
bottle code	B	C
mass of B	28.15	28.715
mass of BS	38.05	38.796
mass of BSW	85.95	86.057
mass of BW	79.68	79.738
specific gravity(Gs)	2.680965	2.716653
average Gs	2.699	

Table B-3.7 Specific Gravity for Kebele-5 at 1.5m depth

Trial	1	2
bottle code	B	C
mass of B	28.222	28.715
mass of BS	38.222	38.715
mass of BSW	85.89	86.157
mass of BW	79.68	79.738
specific gravity(Gs)	2.638522	2.792516
average Gs	2.716	

Table B-3.8 Specific Gravity for Bocho bore at 1.5m depth

Trial	1	2
bottle code	B	C
mass of B	28.222	28.715
mass of BS	38.222	38.715
mass of BSW	85.99	86.157
mass of BW	79.68	79.838
specific gravity(Gs)	2.710027	2.716653
average Gs	2.713	

Table B-3.9 Specific Gravity for Technic at 1.5m depth

Trial	1	2
bottle code	A	T48
mass of B	28.15	28.715
mass of BS	38.05	38.796
mass of BSW	85.93	86.047
mass of BW	79.68	79.738
specific gravity(Gs)	2.666667	2.709293
average Gs	2.688	

Table B-3.10 Specific Gravity for Kochi at 1.5m depth

Trial	1	2
bottle code	A	T48
mass of B	28.15	28.715
mass of BS	38.05	38.796
mass of BSW	85.93	86.047
mass of BW	79.68	79.728
specific gravity(Gs)	2.666667	2.716653
average Gs	2.692	

Table B-3.11 Specific Gravity for Merkato-2 at 1.5m depth

Trial	1	2
bottle code	B	C
mass of B	28.15	28.715
mass of BS	38.05	38.796
mass of BSW	85.95	86.057
mass of BW	79.68	79.73
specific gravity(Gs)	2.680965	2.72257
average Gs	2.702	

B – 4) Atterberg Limits Determination

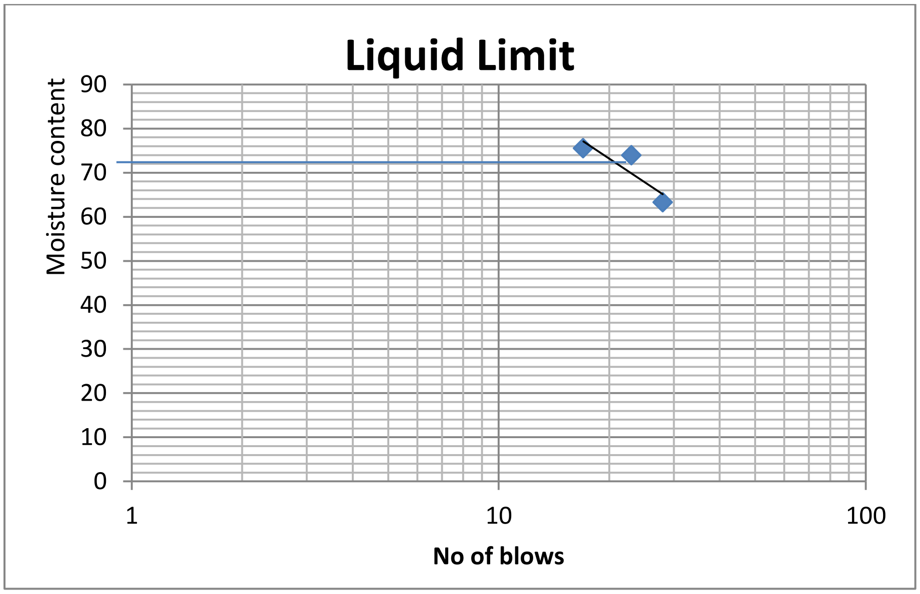


Figure B - 4.1Liquid Limits for Furdisa at 1.5m depth

Plastic Limits for Furdisa at 1.5m depth

Trial	1	2
cane code	NC	11
Mc	6.166	6.437
Mc +wet soil	14.804	13.781
Mc +dry soil	12.743	12.0179
moisture content	19.96	21.49
Average	20.12	

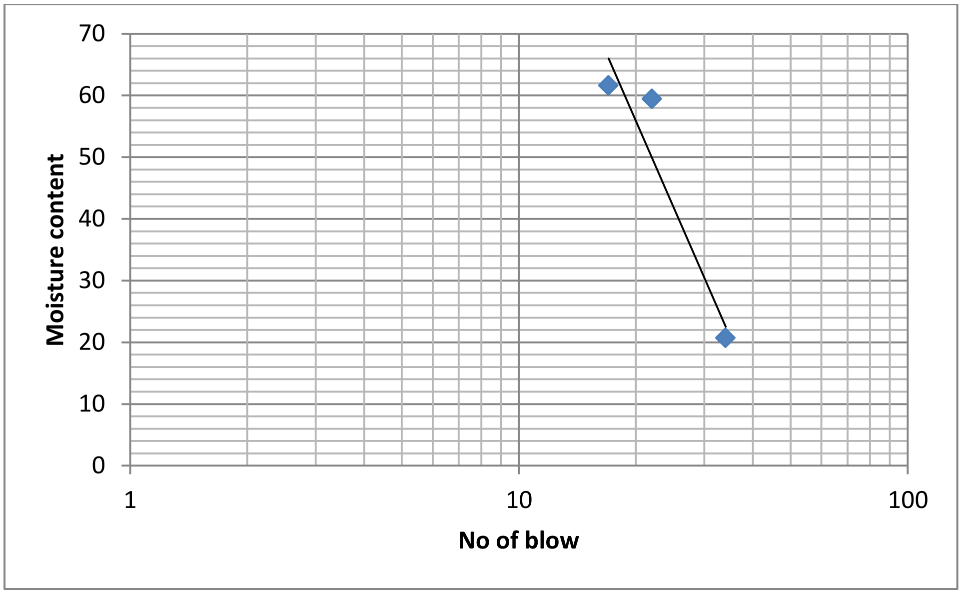


Figure B - 4.2 Liquid Limits for Agriculture at 1.5m depth

Plastic Limits for Agriculture at 1.5m depth

Trial	1	2
cane code	T6	TC-2
Mc	6.1	6.34
Mc +wet soil	13.831	15.167
Mc +dry soil	11.831	12.8
moisture content	27.59	28.03
Average	28	

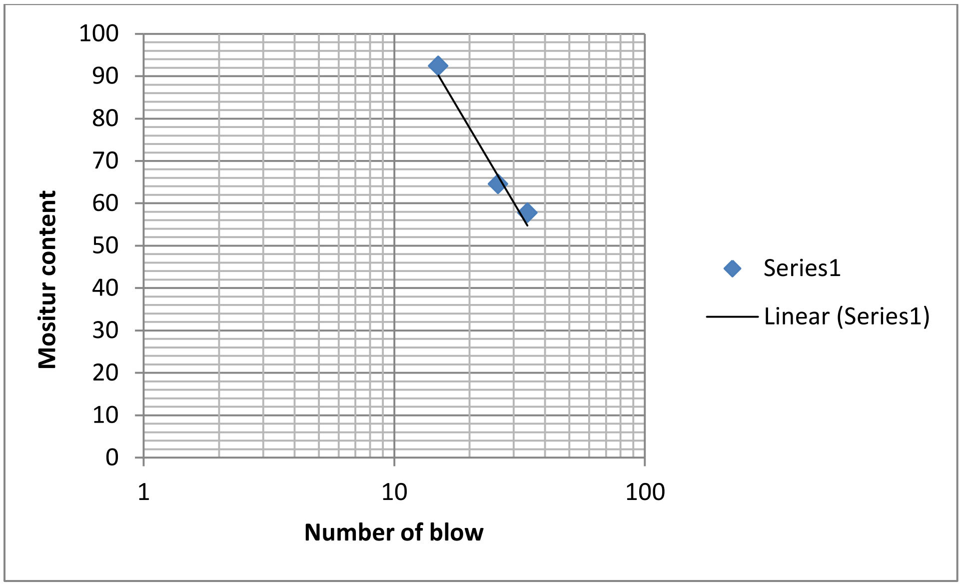


Figure B - 4.3Liquid Limits for Furstale at 1.5m depth

Plastic Limits for Furstale at 1.5m depth

Trial	1	2
cane code	LL1	A1
Mc	6.408	6.062
mc+ wet soil	13.313	13.548
mc+ dry soil	11.667	11.687
moisture content	31.298726	31.08444
Average	31.19	

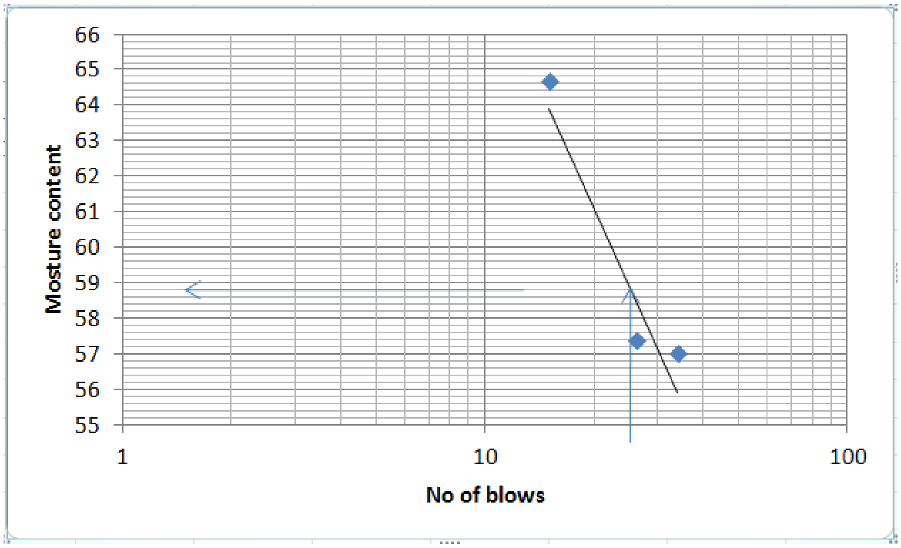


Figure B - 4.3 Liquid Limits for Kochi at 1.5m depth

Plastic Limits for Kochi at 1.5m depth

Trial	1	2	3
cane code	L3	A3	2
Mc	6.98	6.96	6.673
Mc +wet soil	13.463	13.728	14.019
Mc +dry soil	12.364	12.663	12.503
moisture content	20.4123328	18.67438	26.00343
Average			21.69

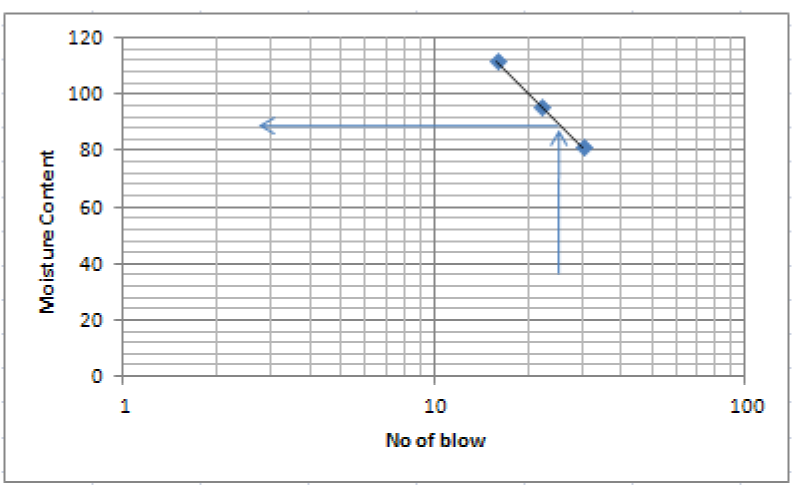


Figure B - 4.4Liquid Limits for Merkato at 1.5m depth

Plastic Limits for Merkato at 1.5m depth

Trial	1	2	3
cane code	SD	ED	AW
Mc	5.819	6.394	5.595
Mc +wet soil	13.3	15.3	11.6
Mc +dry soil	11.368	12.933	9.98
moisture content	34.8170842	36.1982	36.94413
Average	35.99		

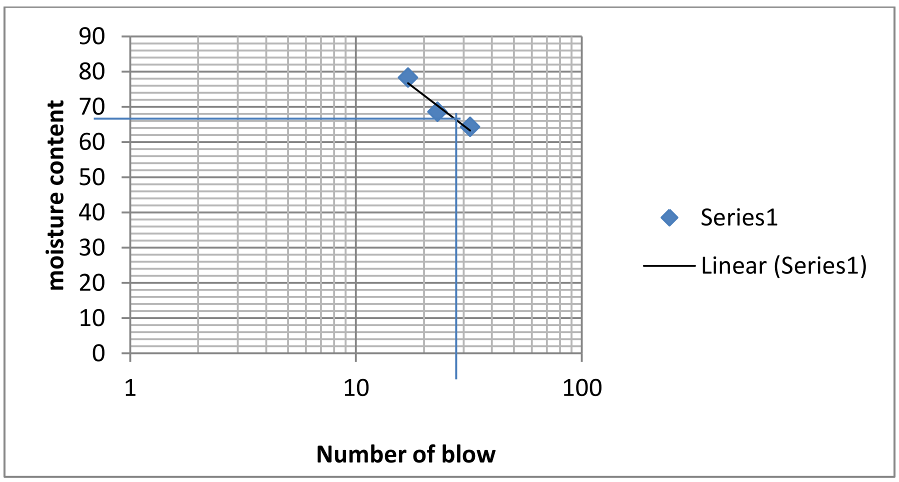


Figure B - 4.5 Liquid Limits for Merkato at 2.5m depth

plastic limit in merkato 2.5m

trial	1	2	3
cane code	SD	ED	AW
Mc	5.819	6.394	5.595
Mc +wet soil	12.3	15.3	11.6
Mc +dry soil	11.068	13.033	10.12
moisture content	23.4711374	34.14671	32.70718
average	30.10834284		

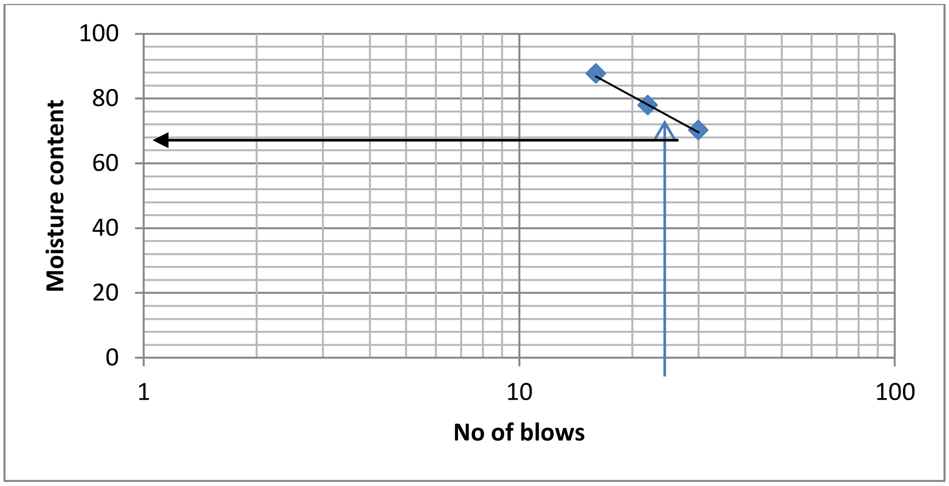


Figure B - 4.6 Liquid Limits for Awetu at 1.5m depth

Plastic Limits for Awetu at 1.5m depth

Trial	1	2	3
cane code	SD	ED	AW
Mc	5.819	6.394	5.595
Mc+ wet soil	12.21	15.22	11.32
Mc+ dry soil	11.068	13.033	10.12
moisture content	21.7565251	32.94171	26.51934
Average			27.07

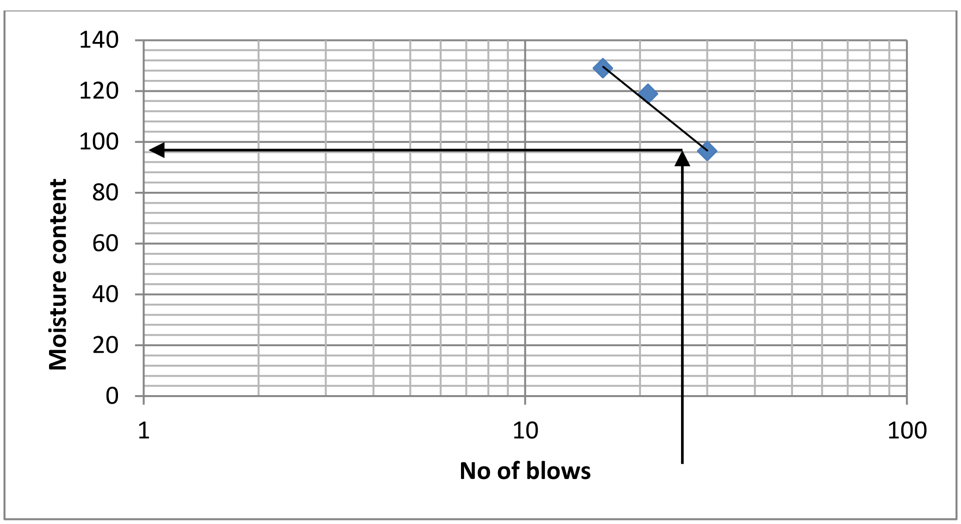


Figure B - 4.6Liquid Limits for Technic at 2.5m depth

Plastic Limits for Technic at 2.5m depth

trial	1	2	3
cane code	T6	TC-2	F1
mc	6.1	6.34	6.101
Mc +wet soil	13.831	15.167	13.301
Mc +dry soil	11.831	12.8	11.486
moisture content	34.8979236	36.64087	33.70474
average			35.08

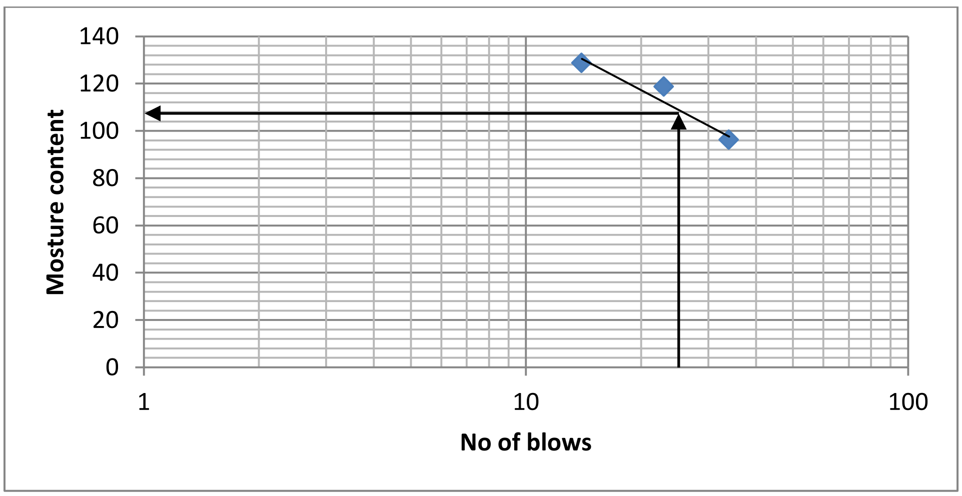


Figure B - 4.7Liquid Limits for Kebele-5 at 2.5m depth

Plastic Limits for Kebele-5 at 2.5m depth

trial	1	2	3
cane code	T6	TC-2	F1
mc	6.1	6.34	6.101
Mc +wet soil	13.931	16.167	13.301
Mc +dry soil	11.831	12.8	11.486
moisture containte	36.6428198	52.12074	33.70474
average			40.82

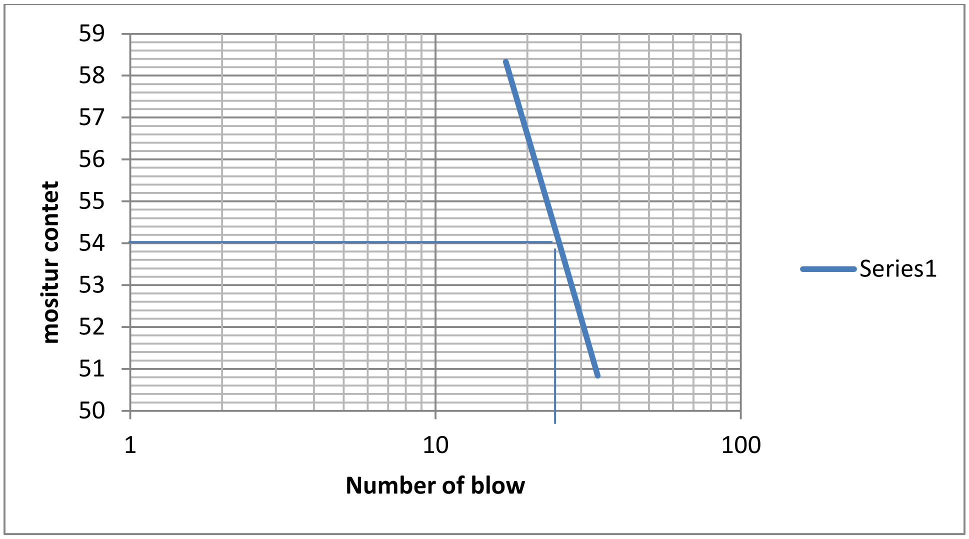


Figure B - 4.8Liquid Limits for Main campus at 1.5m depth

Plastic Limits for Main campus at 1.5m depth

trial	1	2
cane code	L3	A3
mc	16.68	16.56
Mc +wet soil	27.42	27.42
Mc + dry soil	25.10	25.06
moisture content	27.55	27.78
average	27.7	

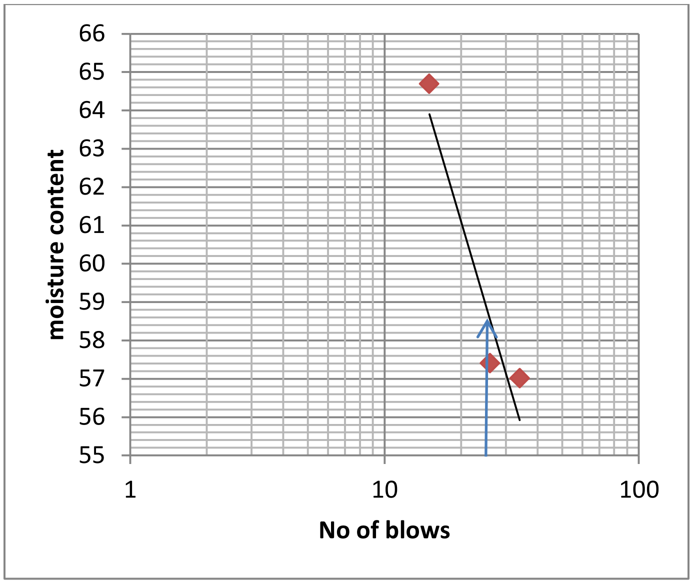
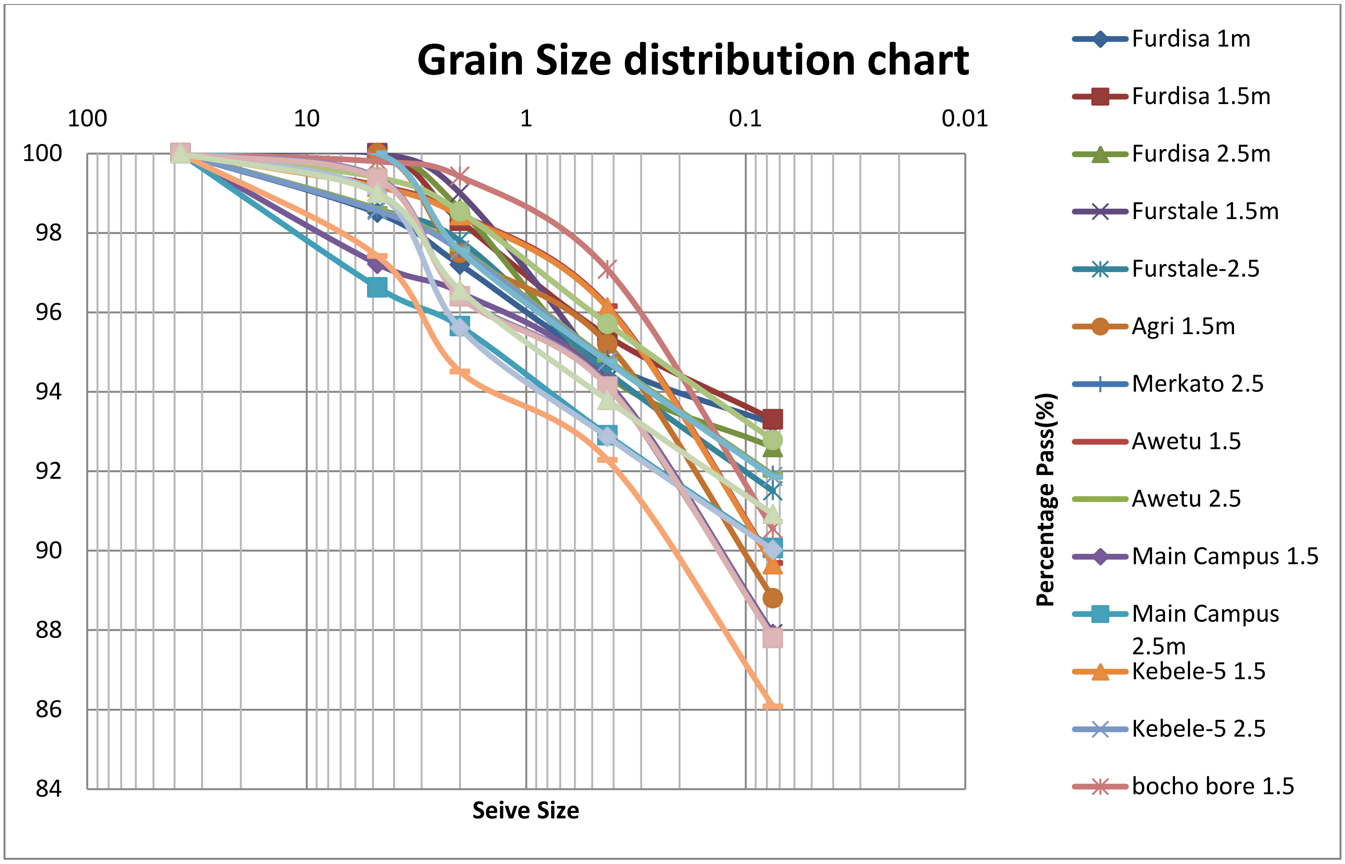


Figure B - 9.8Liquid Limits for bocho bore at 1.5m depth

B – 5) Particle Size Determination



B – 6) Unconfined Compression Strength Determination

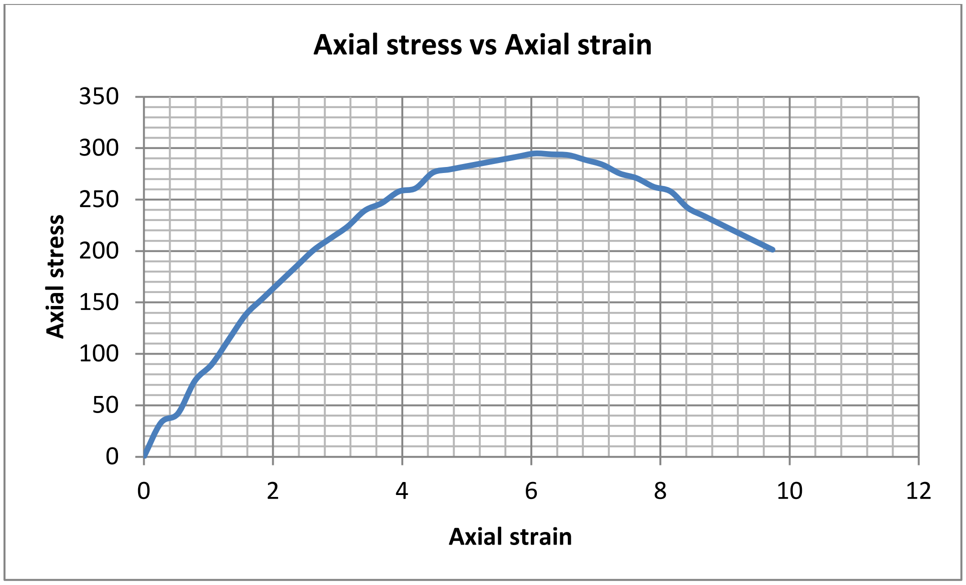


Figure B - 6.1 Unconfined Compression Strength for furdisa at 1.5m depth

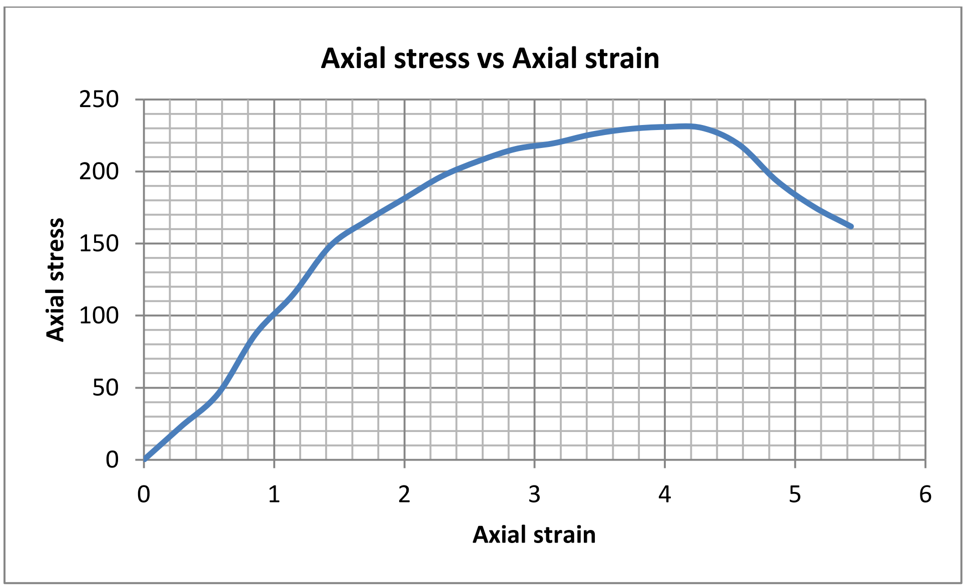


Figure B - 6.2 Unconfined Compression Strength for Agriculture at 1.5m depth

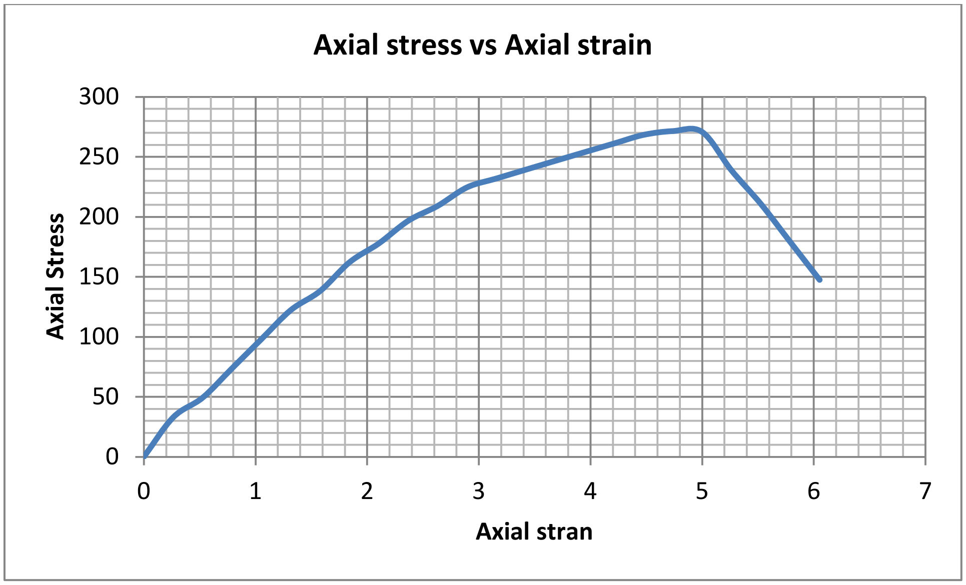


Figure B – 6.3 Unconfined Compression Strength for Furstale at 1.5m depth

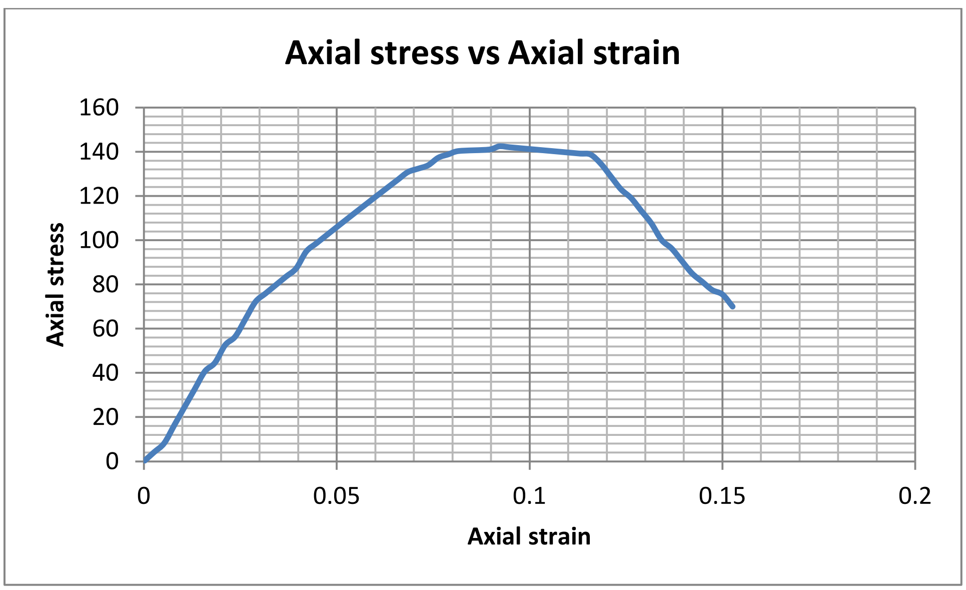


Figure B - 6.4 Unconfined Compression Strength for bocho bore at 1.5m depth

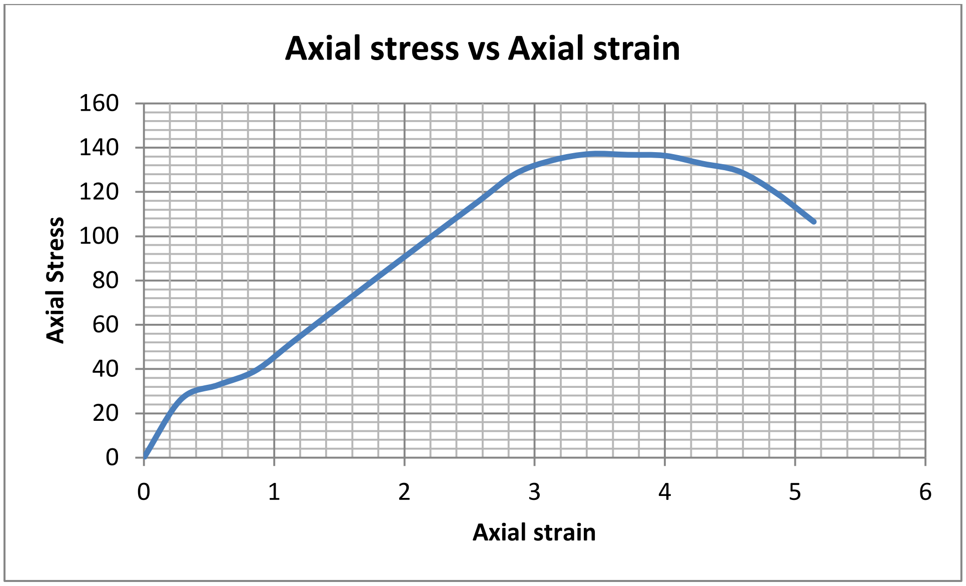


Figure B -6.5 Unconfined Compression Strength for Kochi at 1.5m depth

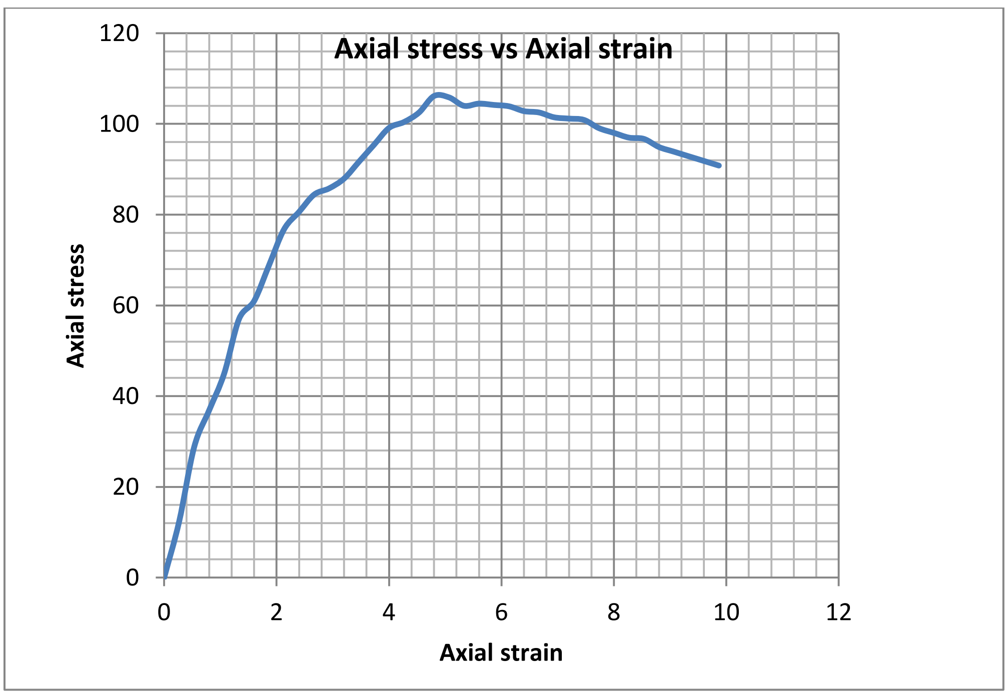


Figure B - 6.6 Unconfined Compression Strength for Merkato-2 at 1.5m depth

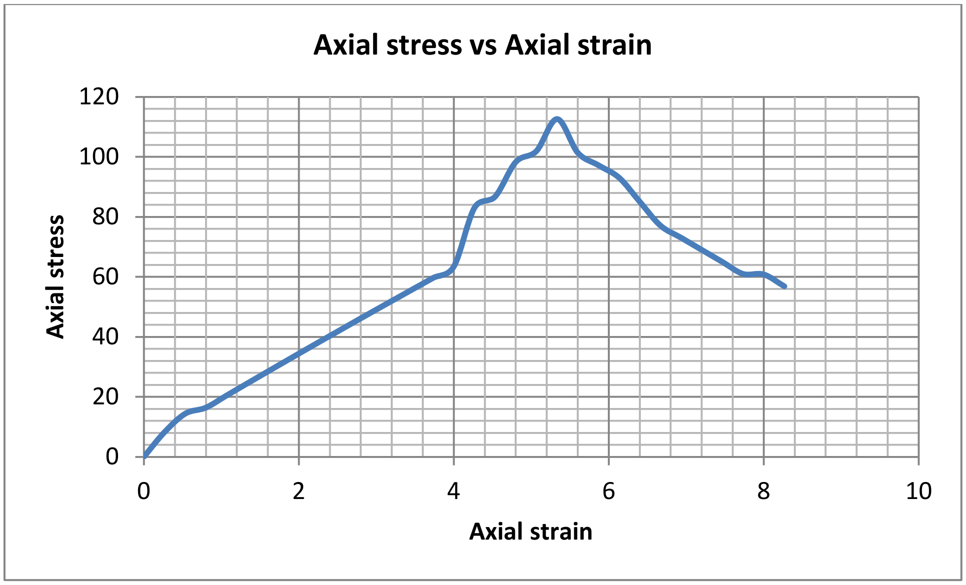


Figure B - 6.7 Unconfined Compression Strength for Merkato at 1.5m depth

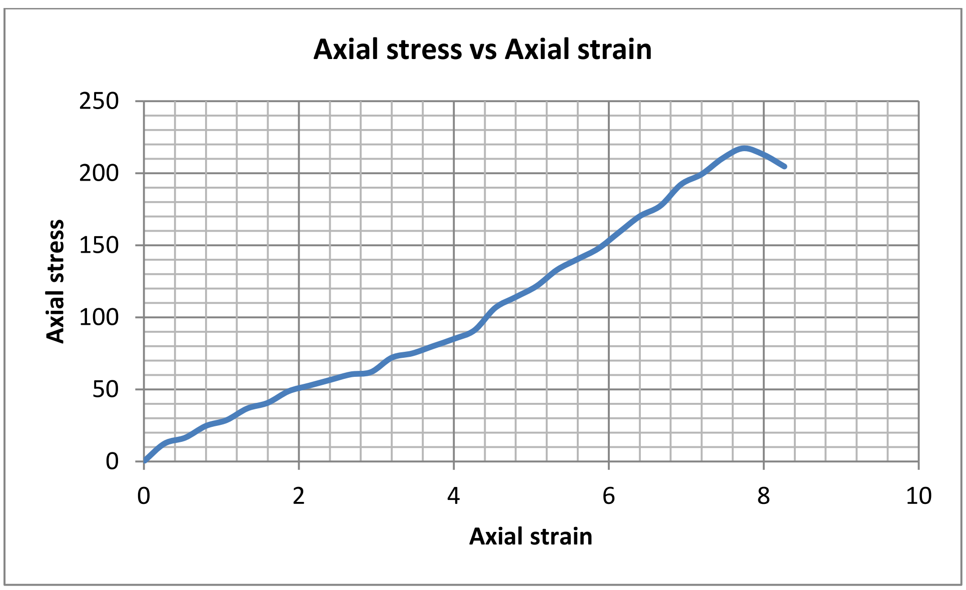


Figure B - 6.8 Unconfined Compression Strength for Awetu at 1.5m depth

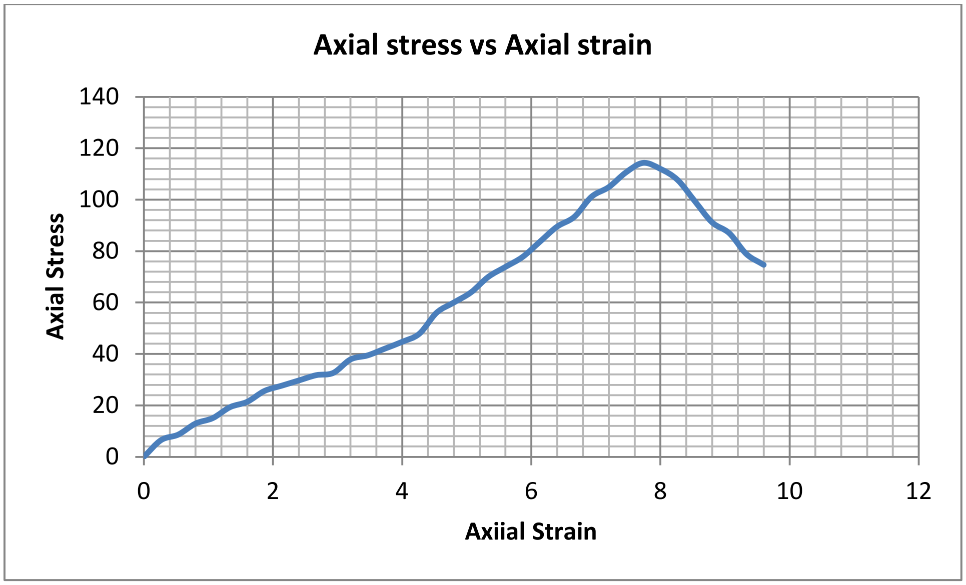


Figure B - 6.9 Unconfined Compression Strength for Teknic at 1.5m depth

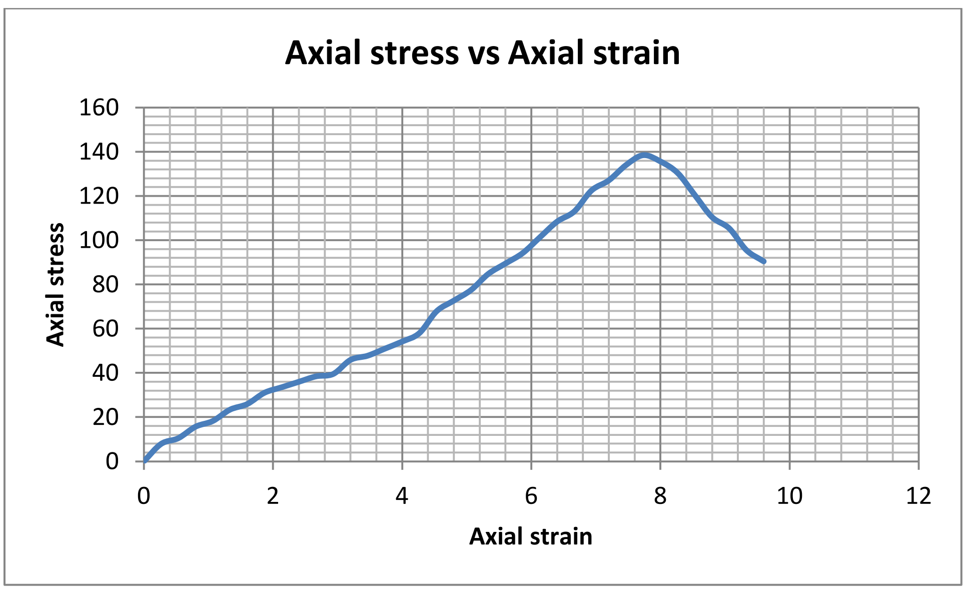


Figure B - 7.10 Unconfined Compression Strength for Kebele-5 at 1,5m depth

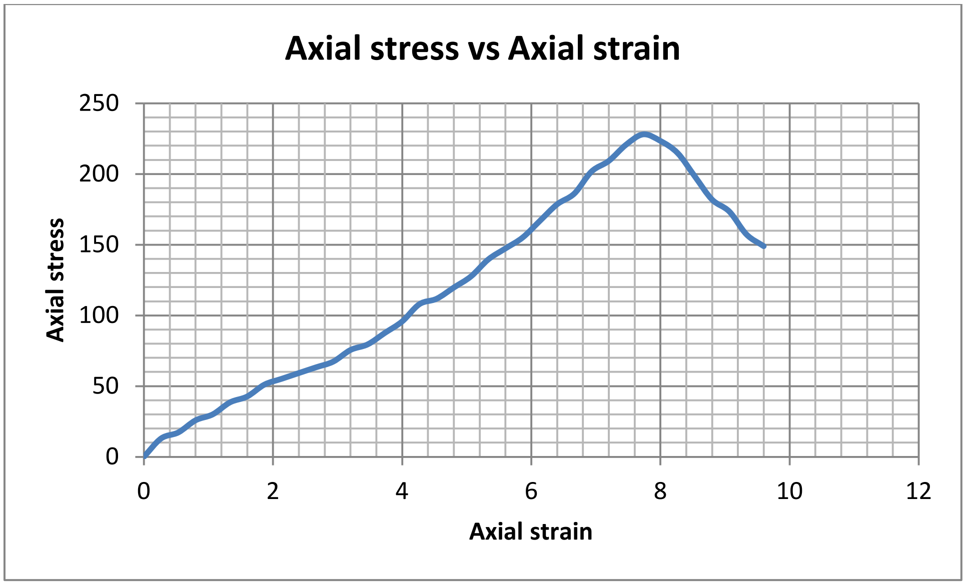


Figure B - 7.11 Unconfined Compression Strength for Main Campus at 1,5m depth