KYAMBOGO UNIVERSITY

DIRECTORATE OF RESEARCH AND GRADUATE TRAINING

CALIFORNIA BEARING RATIO FOR COHESIVE SOILS FROM QUASI STATIC CONE TESTS

BY

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APPROVAL

The undersigned approve that i have read and hereby recommend for submission to the Graduate School of Kyambogo University, a dissertational entitled **California Bearing ratio for cohesive soils from Quasi static cone tests** in fulfillment of the requirements for the award of Master of Science in Structural Engineering Degree of Kyambogo University.

Dr. Michael Kyakula

(Supervisor)

Signature.....................

Date: ---01/10/2022---.

Declaration

I Kaweesi Robert, hereby declare that this submission is my own work and that, to the best of my knowledge and belief, it contains no material previously published or written by another person nor material which has been accepted for the award of any other degree of the university or other institute of higher learning, except where due acknowledgement has been made in the text and Reference list.

Signature.....................

Date: ----01/10/2022.----

Dedication

This piece of work is dedicated to my beloved wife Nassolo Juliet ,Children Nakyanzi Trishillar,Namanda Denis Lukyamuzi Paul and Kaweesi Shadrack and my parents; mother Mrs Nassali Mirioth and Father; Mr. Kizza Christopher

Thank you all for inspiring me to be a better person.

Acknowledgement

This Dissertation was made possible with the intellectual guidance of my dear Supervisors, Dr. Michael Kyakula and Dr. Benjamin Kyambadde who passed away as the write up was being completed. The late Dr. Benjamin Kyambadde‘s contribution is greatly acknowledged. Thank you for the cordial environment I enjoyed throughout the supervision process.

Special tribute is paid to Laboratory Technicians who rendered to me assistance in accessing all the relevant equipment needed for the success of this study.

Finally, heartfelt thanks to God who gave me the strength and wisdom to complete this especially in the trying times.

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Abstract

In the majority of nations, the test most frequently employed to identify flexible pavement layers is the California Bearing Ratio (CBR). To get around the test's high cost, inconvenience, and length, Quasi static cone penetrometer machine was fabricated and used to measure the consistency limits (liquid limit-LL, Plastic limit-PL and Plasticity index-PI), which were used to develop an empirical equation to determine CBR. The soil samples were collected in the Districts of Masaka,Kalungu ,Lwengo and Kyotera. A total of fifty soil samples were gathered and taken to the lab for analysis. To ascertain the CBR in the dry state, soil tests were conducted, Plastic limit,Liquid limit and plasticity index . Quasi static penetration forces at 20mm depth of penetration were determined for all soil samples at moisture content equivalent to plastic and liquid limit. It was found that the a forces of 1020gf and 60gf were achieved at a depth of 20mm when the soil was at state of plastic and liquid limit respectively. The correlation and regression analysis between consistency limits, and the experimental CBR obtained showed relatively good coefficient of determination of R^2 = 0.907 between CBR and all the parameters using multiple linear regression analysis (MLRA). The regression equation developed was used together with the relationship developed between the Quasi static Penetration force at consistency limits and the tested consistency limits using thread rolling and cone penetrometer test to come up with the general empirical equation. The empirical equation was validated and found that the CBR values tested using the Quasi static cone penetrometer machine had average variation of 11% compared with those tested using California Bearing ratio equipment.

CHAPTER ONE: INTRODUCTION

1.0 Back ground

Any long-term growth plan for a country must include the development of its transportation infrastructure. The expansion of a country's road system is viewed as a barometer of its economic, social, and commercial development (Katte, et al., 2018). No region or country can develop without adequate transportation facilities especially the road system. This therefore makes sense that 20.8% of the Uganda’s National Budget are allocated to the transportation sector (National Budget Frame work paper FY2019/20-FY 2023/2024). Therefore, it is crucial that accurate soil characterization be done early on in the planning, design, and construction of a road network. This is done to prevent endangering the planned infrastructure, particularly the roadways.

The most used technique is the California bearing ratio (CBR) test for determining the soil bearing strength of the pavement material and is fundamental to pavement design practice in most countries (Zumrawi, 2014).Soil bearing capacity plays a very important role for the design of highway structure. It determines the design thickness of the pavement. The bearing capacity of the sub grade is mostly influenced by the type of soil, water content and its density (Kian, et al., 2005) cited (Huang, 1993 and Lay, 1990).

In Uganda, it is a common practice to determine the subgrade soil bearing capacity for highway pavement design using CBR test measurement. Representative soil samples are compacted at a predetermined maximum dry density and optimal moisture content for the soil material in order to calculate the CBR. The CBR value is only then determined after 4 days of immersion in water and soil penetration with a plunger (BS 1377 part 9).

Due to the differences in technical characteristics throughout the road, performing this experiment on soil samples gathered from a small number of locations cannot accurately represent the entire length of the road. In order to overcome this, a lot of specimens must be collected for testing, which adds cost, time, and labor to the process (Katte, et al., 2018). Also this test is costly as it involves a high level of technical supervision and quality control assessment (Iqbal, Kumar and Murtaza.,2018). This has led to delay to complete the road projects which were supported by Muzaale and Auriacombe,(2018)who in their research found that 87% of respondents agreed that designs and construction complexity leads to delay works to be completed Further more Muhwezi and Otim, (2014) found that, delay in performing inspections and testing as one of the major causes of delays to complete the projects.

Research on correlation between DCP and CBR value has been performed on clay sand and sand soils. The study aimed at relating the result of DCP to CBR value, which takes into account the soil density (Zumrawi, 2014). However, Ahsan,(2015 found out that DCP has a limitations that it needs to be held vertically, a person using it must lift the hummer carefully so as not to lift the whole instrument and releasing the hammer someone must be careful so that it is not out of plumb.

Other methods are available to determine sub grade bearing capacity such as Plate Bearing test, and Hand Cone Penetrometer (HCP) test, also known as Proving Ring Penetrometer (Nugroho, Yusa, and Satibi, 2016). However all these studies need to be carried out in other types of soils like peat soil. Also the liquid limit (LL) and plastic limit (PL) tests are among the most commonly specified tests in the geotechnical engineering industry and originate from the original research of Atterberg (Zumrawi,2017), which was subsequently standardized for use in civil engineering applications (Casagrande ,1932 & 1958), and adopted for the classification of fine-grained soils. These Atterberg limits have been used for numerous purposes, including the estimation of shear strength, deformation and critical-state soil mechanics parameter values. However, (Hrubesova, Lunackova and Brodzki,(2016) noted that the error in using the Casagrande tool may arise due to the differences in behavior in response to shaking. For plastic limit, the test is also very sensitive to the operator technique. Also, difficulties were reported in using the casagrande apparatus method for soils of low plasticity; for which double edged grooving tools were developed (Hovayni, 1958). There are also difficulties in controlling the rate of penetration during fall-cone tests. This complicates their use over the entire plastic range, particularly close to the plastic limit, where slight variations in moisture content may significantly affect the soil strength (Stone and Kyambadde, 2007).

The quasi-static cone test procedures are essentially the same as for the fall-cone tests and are also described by Stone and Phan (1995); Kyambadde, (2003) and Stone and Kyambadde ,(2005 and 2007).This instrument was essentially used to determine plastic ranges of test soils as opposed to other approaches that concentrated on testing at moisture contents around the atterburg limits.Both these two tests, the California bearing Ratio and the Atterberg tests ,are crucial in determining the soil properties which are helpful in design of civil engineering infrastructures. However, carrying out these tests needs a lot of time and money; this may be one of the reasons why most construction projects in Uganda are delayed to be completed (Muhwezi and Otim, 2014).

There is need therefore to find out the correlation between the California bearing ratio and quasi static cone penetration such that incase one of the above tests is carried out, one can be able to predict the other properties. The relationship developed will reduce on time and cost in carrying out these tests hence reduction in delays to complete road projects.

1.1 Problem statement

In Uganda, it is a common practice to determine the subgrade soil bearing capacity for highway pavement design using California bearing ration (CBR) test measurement. California bearing ratio is calculated by gathering representative soil samples that have been compacted at a predefined maximum dry density and optimal moisture content for the soil composition. The CBR value is only then determined after 4 days of submersion and soil penetration with a plunger (BS 1377 part 9).However, carrying out this exercise for a given number of representative samples is expensive, time consuming and laborious (Katte, et al., 2018).These has lead to many cases to carrying out inadequate tests and at times these tests delay the completion of the project.

Research on correlation between DCP and CBR (Zumrawi, 2014), plate bearing test and Hand cone penetrometer (HCP) (Nugroho,et al, 2016) have been carried out in order to reduce the time and cost. However, it has been found that DCP has limitations in that, it needs to be held vertically, a person using it must lift the hummer carefully so as not to lift the whole instrument and while releasing the hammer one must be careful so that it is not out of plumb. In other methods the research was carried out on inorganic soils only and the correlations need to be modified on other types of soils.

Therefore, there is need to develop a correlation between the California bearing ratio and quasi static cone penetrations such that it is possible to predict CBR and other properties using quasi static cone penetrations.

The developed relationship will reduce on time and cost in carrying out these tests hence improvement on the speed of work and reduction in delay to complete road projects.

1.2 Main Objective

The main objective of this study was to investigate the relationship between California bearing ratio and quasi static cone penetration in cohesive soils.

1.3 Specific Objectives

(i) To determine conventionally derived consistency limits of soils of different plasticity.

(ii) To determine quasi static consistency limits of soils of different plasticity.

(iii) To determine the bearing strength of the soil samples using CBR for given moisture content.

(iv) To develop the empirical relationship between conventionally derived consistency limits and CBR.

(v) To develop the empirical relationship between Quasi static cone penetrations and California bearing ratio.

1.4. Research questions

The study was guided by the following research questions:

- What are the consistence limits of cohesive soils of low to high plasticity determined using conventionally derived methods?
- What are the consistence limits of cohesive soils of low to high plasticity determined using Quasi static cone penetration?
- How is the soil bearing strength of the soil sample at given moisture content determined using CBR?
- What is the relationship between conventionally derived consistency limits and CBR?
- What is the empirical relationship between the quasi-static cone penetration and California bearing ratio?

1.5. Conceptual Framework

Figure below shows the key variables of the study.

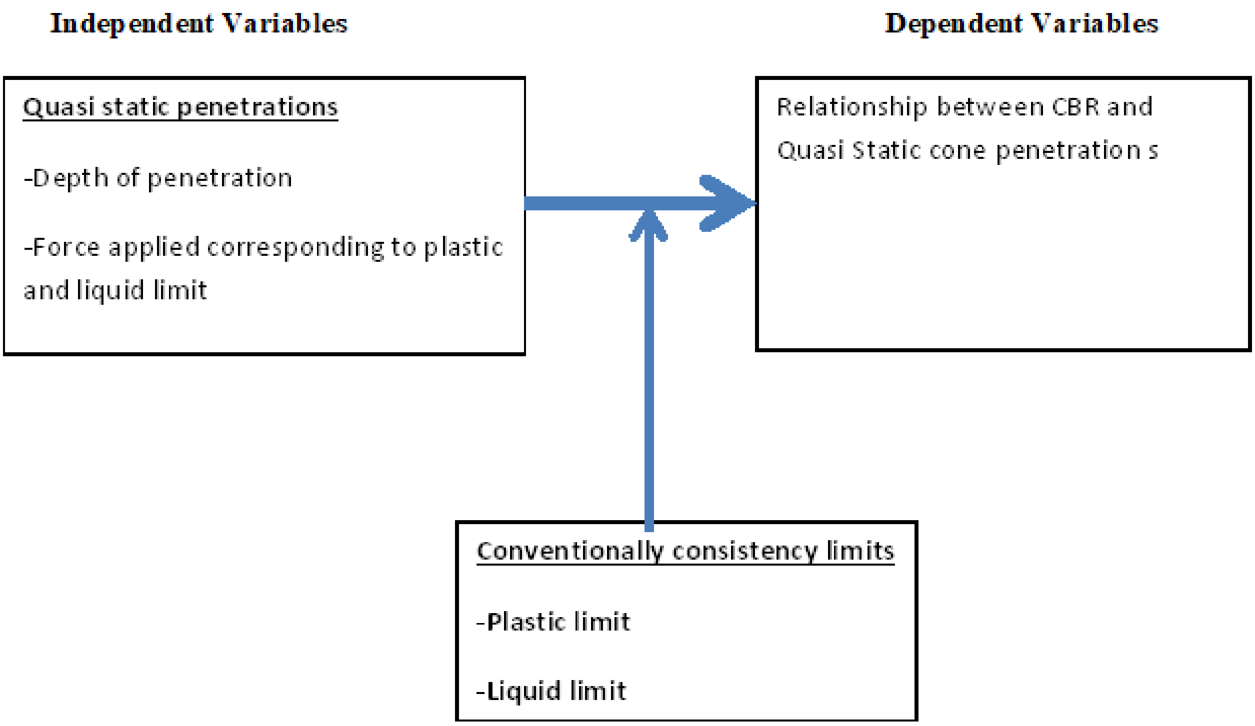

Figure 1.1: Shows the Conceptual framework for Quasi static consistency limits as independent variables and CBR as dependent variable.

The conceptual framework demonstrates the relationship between the independent variables of quasi static penetrations with dependent variable as California bearing ratio. Conventionally derived consistency limits were used as intervening variables. In the study, it was hypothesized that, there was significant effect of quasi static cone penetration forces at consistency limits with CBR.

Quasi static penetration forces at a given depth of penetrations were established at plastic and liquid limit .A relationships between the consistency limits and CBR were established which were used to develop the empirical relationships between quasi static cone penetration forces at consistency limits and CBR.

1.6 Significance of the study

It was found that many engineers, designers and implementers carry out inadequate California bearing tests because this test takes a lot of time and money .So by finding out the relationship between quasi static cone penetration and CBR, engineers will use quasi static cone penetration tests to predict the CBR of the soil. Since quasi static cone test is simple and takes little time and money, therefore many engineers and implementers will use it and this will help in coming up with good decisions for design of pavement layers and this will reduce on the delays to complete the road projects.

Also, by using this simple equipment is expected that the quality control and quality assurance of the road engineering projects will be improved. In the end, it is hoped that the research would produce knowledge and contribute to the body of knowledge already known in the community.

1.7Justification of the study

For the design of roadway buildings, the California Bearing Ratio is crucial. It determines the design thickness of the pavement. However, carrying out this test is expensive, time consuming and laborious (**Katte, et al., 2018**).This means that if there is no simplified and less costly test developed then many road projects will not be completed in time due to delay in design and construction as supported by (Muzaale and Auriacombe, 2018),also they will be a delay in completion of projects due failure in performing inspections and testing of the materials in time (**Muhwezi** and **Otim**, 2014). Basing on the above failures it may also lead shoddy work and low life span of the completed projects.

1.8 Scope

Geographical scope

The study concentrated on soils in Masaka, Kalungu,Lwengo ,and Rakai District where by at least 9 samples were collected in each district.

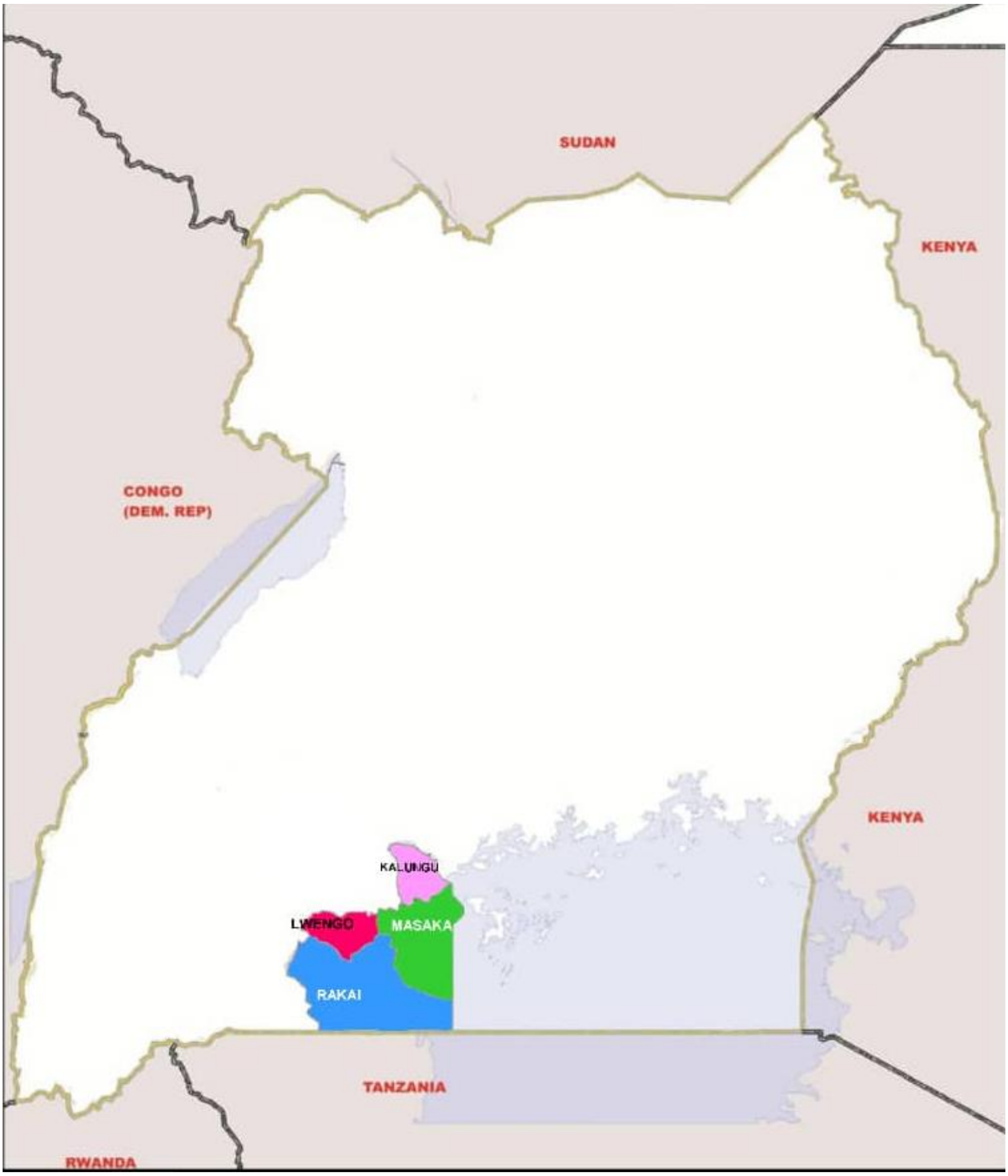

Figure 1.2: Shows map of Uganda indicating Districts where soil samples were collected.

1.8.2. Content scope

The study concentrated on determining the atterberg limits, quasi static cone penetrations forces at atterberg limits and California bearing ratio of soil samples and finding their empirical relationship. 50 samples of soils were tests and compared to find the relationship between these tests.

1.8.3. Time Scope

The study took a period of one year starting from the proposal writing, collection of samples, testing, analyzing data and report writing.

CHAPTER TWO

LITERATURE REVIEW

2.1. Introduction

The chapter presents a review of related literature as guided by study objectives and issues indicated in the conceptual framework about estimating California bearing ratio using quasi static cone penetration tests. The main emphasis were to determine Quasi static penetrations and which were used to estimate California bearing ratio; Different literature sources were used to get ideas and concepts for what other scholars and institutions have done in the same field and the gaps in the literature identified the prediction of CBR using quasi static cone penetrations. Below is a summary of the literature review.

2.2 Basic Definitions

Cone Penetrometer test: is an alternative technique to the Casagrande technique for determining the soil sample's liquid limit.

Atterbarg limits; It refers to shrinkage, plastic and Liquid limits.

California bearing ration: According to BS 1377 part 9, it is the difference between the force per unit area needed to penetrate a soil mass at a rate of 1 mm/min and the force needed to do the same with a standard material.

Bearing capacity: is a measurement of the soil's ability to withstand applied loads.

2.3. Consistency Limits

According to Arthur Cassagrande (1958), fine-grained soils, such as clays and silts display significant changes in behavior and strength depending on the water content. Swedish soil scientist Albert Atterberg created a series of restrictions in relation to the water content of different kinds of soils to measure these behavioral changes. They were later refined by Arthur Casagrande in 1958.

According to the Atterberg Limits System, these fine-grained soils can exist in four different states depending on their water content: solid, semisolid, plastic, and liquid. The weight of the water in the soil sample (Ww) is divided by the weight of the soil sample without any water (Ws), and the result is multiplied by 100 to give the water content (w), which is then expressed as a percentage (Brownjor,2016).

ie $w = \frac{W_w}{W_s}$ ……………………………….. ……………………………….. Equation 2.1

The atterberg limits can be explained as follows.

2.3.1. Shrinkage limit SL.

The water content at which the soil stops changing in volume despite further moisture content reduction is known as the shrinkage limit. The minimum amount of water necessary for the soil to be totally saturated. The soil will become partially wet at any water content below the shrinkage limit. At this moment, soil will transition from a semi-solid to a solid form.

2.3.2. Plastic Limit PL.

The soil's plastic limit is the amount of water at which it transitions from a plastic to a semi-solid form. Any form change will result in the soil showing obvious cracks when rolled in a thread of 3 mm diameter; dirt can no longer act as a plastic material.

2.3.3. Liquid Limit, LL

The soil's liquid limit is the amount of water at which the soil grains are just sufficiently separated from one another for the mass to lose its shear strength. A slight excess of this water content tends to make the soil flow like a viscous fluid, while a slight deficiency makes the soil act more like plastic.

2.3.4. Plasticity index

An indicator of a soil's plasticity is the plasticity index (PI). The range of water contents at which the soil exhibits plasticity is measured by the plasticity index. The PI is the difference between the liquid limit and the plastic limit (PI = LL-PL) (Das, 2006).

Soils with a high PI tend to be clay, those with a lower PI tend to be silt, and those with a PI of 0 (non-plastic) tend to have little or no clay.

Soil descriptions based on PI (Das, 2006);

- 0 -Non plastic.
- 1–5 Slightly plastic.
- 5–10 Low plasticity.
- 10–20 Medium plasticity.
- 20–40 High plasticity.
- >40 Very high plasticity .

2.3.5 Liquidity index

The natural water content of a soil sample is scaled to the maximum levels using the liquidity index (LI). It can be determined as a ratio of the difference between the liquid limit, the plastic limit, and the natural water content:

$$LI=\frac{(W-PL)}{(LL-PL)} \qquad \text{...Equation 2.2}$$

Where W is the natural water content (Das, 2006).

From the plasticity chart (BS 5930: 1999), soil plasticity can be determined over a range of liquid limits and divided into five categories as; low (for liquid limits less than 35 %), intermediate (for liquid limits between 35 % - 50 %), high (for liquid limits between 50 % - 70 %), very high (for liquid limits between 70 % - 90 %) and extremely high (for liquid limits greater than 90 %). The five categories are further classified as silts or clays, based on where they lie in relation to the ‘’A-line;’’ thereby generating a total of ten plasticity classes (Kyambadde, 2010).

2.3.6 Consistency index

The consistency index (CI) measures a soil's firmness and consistency. It is determined as;

$$CI=\frac{(LL-W)}{(LL-PL)} \qquad \text{Equation 2.3.}$$

W represents the current water content. A consistency score of 0 indicates liquid soil, whereas a consistency index of 1 indicates soil at the plastic limit and CI is negative if W>LL. That indicates that the soil is liquid.

2.4. Application of Test

Numerous engineering behaviors, including compressibility, permeability, workability, shrink-swell, and shear strength, are correlated with soil parameters, including the plasticity index, liquid limit, and plastic limit (STP, 2000). And the liquidity index also finds application in the estimation of the sensitivity of natural soils (Terzaghi et.al. 1996).

The fall cone can also be used to determine the soil sample's undrained shear strength according to Hansbo's findings (1957). He stated that the undrained shear strength of the soil is directly proportion to weight of the cone divide by the depth of penetration squared.

I.e. $\tau_f = KQ/h^2$.

Where τ_f is the undrained shear strength ,K is the cone factor, Q is the weight of the cone and h is the depth of penetration.

2.4.1 Plastic Limit Determination

According to standard Test procedure manual 2000, BS 1377-2 1990, the plastic limit can be determined by mixing about 20g of soil with water and roll the ball of soil by hand on the rolling surface with just enough pressure to form an elongated thread if the soil can be rolled to a thread of 3mm thick without crumbling, a portion of soil is taken to determine the moisture content which is the plastic limit.

2.4.1.1. Calculations for Plastic Limit:

A part of the soil sample is weighed and dried to estimate the dry weight after it has been rolled to a thread thickness of 3mm without disintegrating. The weight of the moisture is then calculated by calculating the difference between the dry and wet weights. The plastic limit PL is calculated by multiplying by 100 and dividing the "weight of moisture" by the "dry weight of sample," as stated in the equation below.

Plastic Limit (PL) = $\frac{Weight - of - moisture}{Dry - weight - of - the - sample} \times 100$ …………………..Equation 2.4.

However, the thread rolling method has been found to yield consistent results by the same operator, and more so with experienced operators (Kyambadde,2010) cited by (Prakash et.al,2009) but consistency is not often obtained in cases where results of tests have to been validated across operators.

Also the thread rolling test is direct contact between bare fingers and the soil sample when applying the rolling pressure, in case of contaminated soil samples. The use of any protective glove by the operator would result in a change in the boundary conditions of the test and may also influence the results (Kyambadde, 2010).

2.4.2. Liquid Limit Determination

For determining the liquid limit of soil fractions smaller than 0.425 mm, two methods are typically employed: the Casagrande apparatus method and the Fall Cone Penetrometer Method (BS 1377-2:1990).

2.4.3. Casagrande Apparatus Method

The casagrande apparatus comprises a standard cup that is lifted by a cam and dropped onto a standard metal base. The liquid limit is then defined as the moisture content of the soil at which a standard pre-cut grove will close after 25 blows (clauses 4.5 and 4.6 of BS 1377-2: 1990). This method was found to produce liquid limitations that were depending on the apparatus's base's hardness (Norman, 1958). Difficulties were reported in using the casagrande apparatus method for soils of low plasticity; for which double edged grooving tools were developed (Hovayni, 1958).

According to Hrubesova et al. (2016), the inaccuracy in utilizing the aforementioned instrument may result from variations in how people react to shaking, and the test is also quite sensitive to the operator approach.

2.4.4. Determination of Liquid limit using fall cone Penetrometer

Liquid limit testing can produce more precise results using the fall cone method. In this procedure, a cone with an 80 gram mass and a 30 degree apex angle is suspended above, with the pointed part coming into contact with the soil sample at the very least. For five seconds, the cone is allowed to fall naturally under its own weight. The liquid limit of the soil is the amount of water that permits the cone to pierce for 20 mm during this time.

But Koumoto and Houlsby (2001) conducted study on the theory and application of the fall cone test and came to the following conclusion: Undrained shear strength can be measured quickly and easily using the fall cone test, which can be understood in terms of basic mechanics. The angle of the cone tip, the roughness of the cone's surface, and the rate of shear strain during penetration measurement are all factors that affect the fall cone penetration. Houlsby, (1982), who discovered that the self-weight of a typical soil will contribute little to the cone resistance (usually 1.5% for a 30°, 80 g cone, and much less for 60° cones), also backed up this claim. Controlling the rate of penetration during fall-cone experiments is challenging, though. This makes their application across the whole plastic range challenging, especially near the plastic limit where even little changes in moisture content can have a big impact on soil strength (Stone and Kyambadde, 2007).

2.4.5. Quasi static cone Penetration

The limitations of using the fall cone apparatus may be eliminated if the penetration rate can be controlled such that rate effects are more effectively accounted for. Kyambadde,(2010) in his research of "soil strength and consistency limits from quasi-static cone tests" proved that Quasi-static cone systems (in which displacement control occurs at slow rates, ideally at 1 mm/s) may provide a better alternative in overcoming limitations of fall-cone penetration approaches, such as rate effects. This was also supported by Stone and Phan,(1995) who reported non-significant variations in quasi-static cone responses at penetration rates ranging between 1mm/s and 5 mm/s. The quasi-static cone test procedures are essentially the same as for the fall-cone tests and are also described by Stone and Phan ,(1995); Kyambadde ,(2003) and Stone and Kyambadde ,(2005). It is, however, worth noting that quasi-static cone tests in this research will generally be taken over the plastic ranges of test soils as opposed to earlier approaches that concentrated on testing at moisture contents around the plastic limit using same procedure as for Kyambadde, (2010). However, in this study, a fabricated quasi cone penetrometer was used.

2.5 Maximum Dry Density (MDD):

Compaction is the process of packing soil particles more closely together while decreasing the amount of air in the soil to increase the density of the soil. The degree of compaction of a soil is measured in terms of dry density, i.e. the mass of solids only per unit volume of soil. If the bulk density of the soil is ρ and the water content w, then it is apparent that the dry density ρ_d is given by;

$$\rho_d = \frac{\rho}{1+w} \quad \text{.......Equation 2.5.}$$

The dry density of a given soil after compaction depends on the water content and the energy supplied by the compaction equipment (referred to as the compactive effort).The compaction characteristics of a soil can be assessed by means of standard laboratory tests. The soil is compacted in a cylindrical mould using a standard compactive effort. In BS 1377 (Part 4) [2] three compaction procedures are detailed.

(i) The Proctor Test: In the Proctor test the mould of 1000ml and the soil (with all particles larger than 20mm removed) is compacted by a rammer consisting of a 2.5-kg mass falling freely through 300 mm: the soil is compacted in three equal layers, each layer receiving 27 blows with the rammer.

(ii) Modified AASTO test :In the modified AASHTO test, the mould is the same as the one used in the proctor test but the rammer consists of a 4.5-kg mass falling 450 mm: the soil (with all particles larger than 20mm removed) is compacted in five layers, each layer receiving 27 blows with the rammer. If the sample contains a limited proportion of particles up to 37.5mm in size, a 2250ml mould should be used, each layer receiving 62 blows with either the 2.5- or 4.5-kg rammer.

(iii)Vibrating hammer test: In the vibrating hammer test, the soil(with all particles larger than 37.5mm removed) is compacted in three layers in a 2123ml mould, using a circular tamper fitted in the vibrating hammer, each layer being compacted for a period of 60 s.

After compaction using one of the three standard methods, the bulk density and water content of the soil are determined and the dry density calculated. For a given soil the process is repeated at least five times, the water content of the sample being increased each time. Dry density is plotted against water content and a curve of the form shown in Figure 2 is obtained. This curve shows that for a particular method of compaction (i.e. a particular compactive effort) there is a particular value of water content, known as the optimum water content (w_{opt}), at which a maximum value of dry density (MDD) is obtained.

Plot of dry Unit Weight v/s Moisture Content (Compaction Curve)

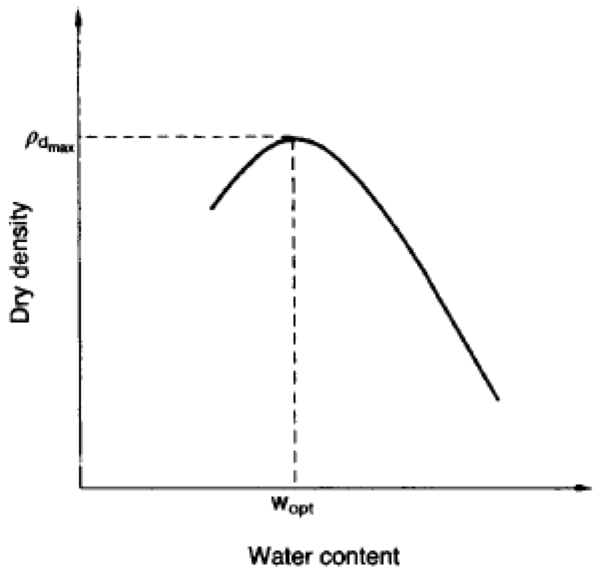

Figure 2.1: Graph of Dry Density against Water Content

However in this research only a proctor test was used to determine MDD of the different soils

2.6. California Bearing Ratio (CBR).

CBR is the ratio expressed in percentage of force per unit area required to penetrate a soil mass with a standard circular plunger of 50 mm diameter at the rate of 1.27 mm/min to that required for corresponding penetration in a standard material.

The ratio is usually determined for penetration of 2.5 and 5mm. When the ratio at 5 mm is consistently higher than that at 2.5 mm, the ratio at 5 mm is used. There are two types of methods in compacting soil specimen in the CBR moulds.

i). Static Compaction method.

ii). Dynamic Compaction method.

Both these two methods the tests are done as per **BS 1377 part 9.**

Table 2.1 gives the standard loads adopted for different penetrations for the standard material with a C.B.R. value of 100%.

Table 2.1: Standard Load Values at Penetration

Penetration of Plunger (mm)	**Standard Load(KN)**
2.5	13.2
5.0	20

The California bearing ratio test is a penetration test used to measure the subgrade tensile strength of pavements and roadways. The findings of these tests are combined with empirical curves to calculate the thickness of the layers that make up the pavement. The most popular technique for creating flexible pavement is this one. The most common metric for sizing flexible pavements in tropical areas is the California Bearing Ratio (CBR). A regression analysis (single and multiple) between the soil's index properties (liquid limit, plastic limit, and plasticity index) using quasi-static cone penetrations must be taken into account because this test is expensive, time-consuming, and labor-intensive.

2.6 Correlate semi-empirical relationships between Quasi static cone Penetrometer and California bearing ratio

The researchers have created the following empirical connections between CBR and other soil characteristics so far:

Zumrawi,(2014) presented the relationship between the CBR and Dynamic Cone Penetrometer (DCP) and came up with the following relationship.

$$CBR = (DCP)[\,Log\,(PI * C)^{26.51} + 8.89]$$
$$[F_i - \frac{0.0269}{(PI * C)^2} + \frac{0.541}{PI * C} - 5.72]$$

…………………Equation 2.6

where: *Fi* : is the initial state factor, *DCP* is the dynamic cone penetration (mm/blow), *PI*: is the plasticity index, *C*: is the clay content. The relationship shown above demonstrates that as the plasticity index grows, the CBR decreases, and as the clay content increases, the CBR decreases as well.

However, Ahsan ,(2015) found out that DCP has a limitation in that it needs to be held vertically, a person using it must lift the hummer carefully so as not to lift the whole instrument and releasing the hammer someone must be careful so as it is not out of plumb

Taha et al,(2015) conducted a research to find the relationship between CBR, Grading Modulus (GM), Maximum dry density (MDD) and came up with the most reliable and accurate model with minimal bias. They came up with best formula as follows:

CBR = 21.21 MDD - 4.34 GM + 0.69 R#10 - 6.013……………………………..Equation 2.7

where, CBR = soaked California Bearing Ratio, %; MDD = maximum dry density according to modified Proctor test, (t/m3); GM = grading modulus, *GM = (P10 + P40 + P200)/100*, R#10 = percentage retained on sieve No.10; P10, P40, P200 = percentage passing U.S. sieve No. 10, 40, and 200, respectively.

Roksana, et al. (2018) investigated the link between CBR and the soil index features of soil samples from Bangladesh. They investigated the relationship between unsoaked CBR and MDD at various levels of compaction (blows), as shown below;

CBR=0.1545MDD +24495 with R^2=0.0689: …………………………………..Equation 2.8

MDD was determined using 10 blows of compaction.

CBR=-0.5028MDD +69.388 with R^2=0.118: …………………………………..Equation 2.9

MDD was determined using 25 blows of compaction.

CBR=-1.228MDD +154.06 with R^2=0.118:…………………………………Equation 2.10.

MDD was determined using 25 blows of compaction.

All the above relationships indicate that MDD alone may not be a good determinant of CBR.

Nugroho,et.al (2016), carried out a research to get a relationship between CBR and Hand cone penetrometer and came up with the following relation ship

$$\text{Field CBR}_{\text{prediction}} = C_0 + C_1\,\gamma + 0.025\ \text{HCP}$$ …………………………………..Equation 2.11.

Where C_0 and C_1 are coefficients depending on the type of soil. HCP is the value of Hand Cone Penetrometer test.

For peat soils, the value of C_0,C_1, and C_2 significantly influenced by fiber peat. The value of C_0, C_1, C_2 is -1.250, 0.085, and 0.005 respectively. However, they discovered in their study that these constants require further testing because it was challenging to establish appropriate values for constants in peat soils with fiber content, which may have had an impact.

Olumuyiwa and Ajibola ,(2017) in their study on the correlation of California bearing ratio value of clays with soil index and compaction characteristics and came up with the following correlations;

UCBR = 32.638 -0.570PI; with R2 = 0.517……………………………………Equation 2.12

$UCBR = 50.013 - 0.113PI - 1.357OMC + 0.003PI^2 + 0.001OMC^2$; With R^2 = 0.938...Equation 2.13

$UCBR = 63.575 + 0.018MDD - 2.727OMC - 9.113 \times 10^{-6}MDD^2 + 0.024OMC^2$;

With $R^2 = 0.940$……………………………………………………………Equation 2.14

Where UCBR=Unsoaked California bearing ratio,

PI=plasiticty index,

OMC =Optimum moisture content

MDD =Maximum dry density

According to the aforementioned correlations, the California bearing ratio falls with increasing plasticity index and ideal moisture content. Additionally, maximum dry density grows along with UCBR, which may be accurate given that strength is a function of density, thus as density rises, we anticipate a rise in UCBR as well. However, their research was carried out on soils with Unsoaked CBR ranging 8%-35%. There is a need to consider also the soils with unsoaked CBR above 35%.

Igbal et.al, (2018), carried out a research on Co-relationship between California bearing ratio and index properties of Jamshoro soils and came up with the following results:

(i) CBRs=0.2807(CBRu) + 5.0352; R=0.718……………………………Equation 2.15.

(ii) CBRu=293.4964 + 25.4466(LL)-59.5422(PI):R=0.691 ………………Equation 2.16

(iii)CBRu=392.0103-2.7748(PI)-154.0842(MDD) :R^2=0.720 ………………Equation2.17

Where CBRs =California bearing ratio for soaked soil samples

CBRu =California bearing ratio for unsoaked soil samples

LL =Liquid limit

PI= Plastic limit

MDD=Maximum dry density

It was observed that CBR values decrease with increase in plasticity index and increase with increase in liquid limit. Also unsoaked CBR was largely dependent on Liquid limit and plastic limit. However, their research was carried out on soils with Unsoaked CBR ranging 65-85. There is a need to consider also the soils with unsoaked CBR below 65.Although Equation (i) can be used to determine soaked CBR ranging from 65-85.

In their 2016 study, "Prediction of CBR using DCP for Local Subgrade Materials," Feleke and Araya came to the following conclusions;

(i) $\log_{10}SCBR=2.015-0.906\log_{10}DCPI$: R= 0.930 Strong relationship

(ii) $\log_{10}UCBR=1.6677-0.895\log_{10}DCPI$: R= 0.902 Strong relationship

(iii) $\log_{10}SCBR=0.397+0.917\log_{10}UCBR$ R= 0.847 Strong relationship

where;

SCBR =California bearing ratio for soaked soil samples

UCBR =California bearing ratio for unsoaked soil samples

DCPI =Dynamic Cone penetrometer carried on insitu soils.

Relationship (i) and (ii) show that as the values DCPI increase it means the soil strength are low and therefore the SCBR and UCBR decreases. And relationship (iii) shows that as UCBR increases also SCBR increases.

In the event that the Unsoaked CBR or DCPI are tested, the above relationship, which was conducted on fine-grained soils, can be utilized to estimate the soaked CBR.

2.7. Summary of the Literature Review

Literature by Zumrawi(2014), Ahsan (2015) and Nugroho et.al (2016) on finding empirical relationship with CBR and other soil properties had some limitations as mentioned above. Literature of Houlsby (1982) and Koumoto and Houlsby (2001) can be based on incase one wants to fabricate his or her own penetration cone basing on the tip and surface roughness.

Also the literature of Stone and Phan (1995) can be used to determine the penetration rate of the quasi static cone penetrometer .

Previous Studies by Feleke and Araya (2016) and Igbal et.al (2018) can be used to determine soaked CBR ounce the unsoaked CBR and DCPI on soil samples are determined.

Therefore, research was necessary to establish the empirical relationship between the CBR and Quasi static cone penetration so as to enable the Engineers and other researchers to predict the CBR using the penetration values from quasi static penetrometer.

CHAPTER THREE

RESEACH METHODOLOGY

3.1. Introduction

This chapter presents the experimental materials, methods or procedures and apparatuses used in this study. The equipments used and recording mechanisms used are presented. All figures and tables of the results are presented at the end of this section.

3.2 Materials for experiments

3.2.1 Fine Soils

Experimental materials comprised of 50 samples of cohesive soils collected from different sites in Masaka,Kalungu,Lwengo and Kyotera District; The choice to employ 50 samples was made in accordance with Abrain's (2014) assertion that the n=30 rule of thumb, where n is the sample size, is a widely used experimental design strategy that uses the central limit theorem. Moreover, the quantity of samples gathered by other researchers in their study on the correlation between soil attributes. Some of the researchers were; Kyambadde, (2010) who tested 83 but of which 37 soil samples from Uganda when he was investigating the relationship between soil strength and consistency limits from quasi-static cone, Nugroho et al (2016) tested 40 samples of soil in Pekanbaru (Indonesia) when they were investigating on estimation of value of CBR from hand cone penetrometer and Katte et.al (2018) collected 33 samples on sangmelima-Mengong road project located in North western edge of Cango craton-Cameroon when they are investing the Correlation of California Bearing Ratio (CBR) value with soil properties of road subgrade soil. Basing on the above researchers with the samples they collected, 50 samples were reasonable.

The targeted samples were 50 samples of different plasticity. The samples were kept in the polythene bags to avoid loss of the moisture content in the soil. The samples were taken to

Kyambogo University laboratory, tested and the results were recorded as presented in chapter four.

3.2.2. Materials

Materials tested were cohesive soils of different plasticity where the particles in the soil can bond to one another. These soils were tested in remolded state after sieving out particles larger than 0.425 mm diameter mixed with distilled water. This was done by remolding the samples using palette knives during mixing. Particular care was taken to breakdown aggregated particles by hand powdering. Remolding was done by kneading with palette knives as recommended in BS 1377-2: 1990.Since this research was only concerned with correlation of CBR with penetrations, other properties of the fine cohesive soils (such as clay mineralogy and composition) were not investigated for soil characterization. This was based on the assumption of any effects of soil mineralogy and composition on plasticity being well reflected in the consistency limits derived. (Kyambadde,2010 cited Mitchell and Soga , 2005).

3.3. Determination of conventionally derived Consistency limits of cohesive soils of low to high plasticity

3.3.1 Thread Rolling Plastic Limit Tests Procedures

BS 1377-2: 1990's clause 5's procedures were followed for conducting the thread rolling plastic limit tests. For the soils which were tested, at least 20 grammes of soil are required at moisture contents sufficient for thread rolling. The sub-samples for plastic limit testing were then subdivided into 2 sub-specimens and each specimen was divided into 4 components; each of which was rolled into threads down to plastic limit moisture contents on visible crumbling during rolling. For each of the sub-specimens, initially rolled into threads of about 6 - 10 mm diameter, uniform light finger pressure was applied during rolling and each of the sub-specimens rolled down to diameters of about 3 mm. Figure 3.1 shows the thread rolling plastic limit test procedure in which the final thread diameter (3 mm) guide rod is included**.**

Figure 3.1: shows thread rolling procedure

3.3.2 Determination of liquid limit using Fall-cone Tests

The BS fall-cone method was used since it is preferred over the casagrande apparatus method (see section 2.4.3) for the determination of the liquid limit (BS 1377-2: 1990). The fall-cone penetrometer device used in this study is shown together with a 30 degree cone and a cone point condition test gauge in Figure 3.2.

Figure 3.2: Fall cone penetrometer

The fall-cone apparatus is fitted with a solenoid release mechanism which releases the cone for self-weight penetration of the 80gf 30 degree cone which occurs over a standardized period of 5 seconds. Additionally, it has a dial gauge with 0.01 mm accuracy. Liquid limit determination with the BS fall-cone is presented in section 2.4.4 and the fall cone test procedures employed in the experimental programme were in accordance with those set out in clause 4.3 of BS 1377-2: 1990. Fall-cone tests were generally conducted over plastic ranges of test soils at penetrations of between 1-30 mm for different soil consistencies. Specimens were prepared as for liquid limits tests except for the fall-cone penetration tests at low moisture contents in which specimen preparation and cup filling takes some time to remove air spaces in the sample. In all cases cups were filled with soil specimens using palette knives starting at the bottom centre of the cup and proceeding outwards to also avoid entrapping air.

3.3.3 Quasi-static Cone Tests

According to Kyambadde (2010), four kinds of apparatuses may be used for quasi-static cone penetration tests during the experimental testing programme and are described in the following sub-sections. Three of these are motorised namely; the soil mini-penetrometer (SMP), a modified triaxial rig, and an adapted universal testing machine. The fourth apparatus is a pocket cone penetrometer (PCP) which is a hand held mini-penetrometer modified from a standard pocket penetrometer.

In this study a fabricated motorized driving system which moves at a speed of 1.33mm/s was used. The machine was made in such a way that it drove the soil sample to be penetrated by the static cone as shown in the Figure. 3.3(a),3.3(b) and 3.3(c).

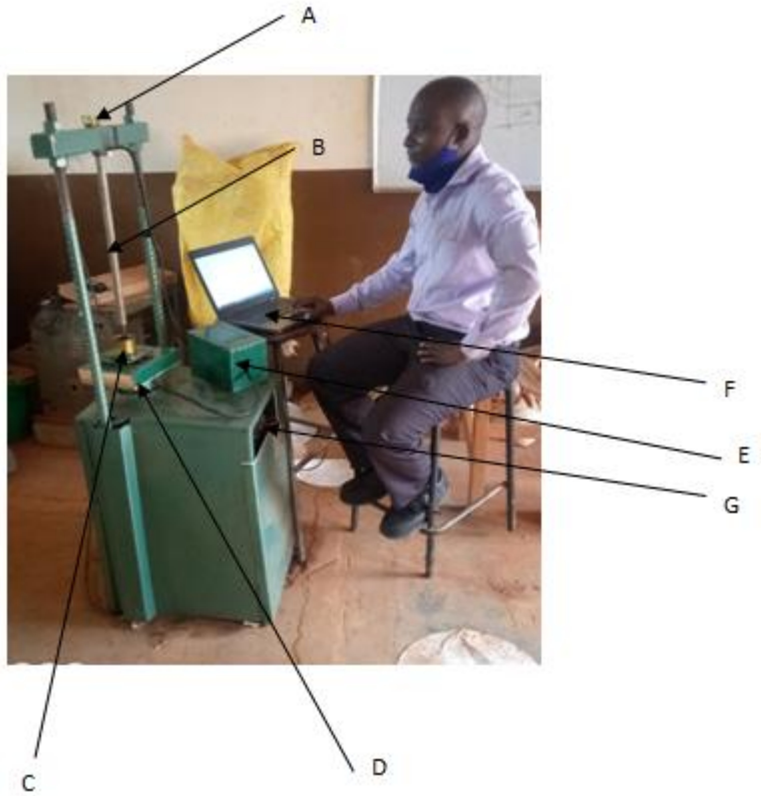

Figure 3.3(a): Quasi-static penetrometer

A is split bubble help us to check whether the horizontal bar is on level.

B is the vertical bar of 20mm diameter with a 30° cornial shapped end used to penetrate the soil sample.

C is a cup of 50mm diameter 40mm height to be filled with the soil sample.

D is a load cell placed on top of the piston used to measure force exerted on the soil sample.

E is the aurdino mother board encased in the metal case used to transfer the data to computer.

F is a computer used to display and store the reading.

G is the switch board where there is the operating switches used to switch on and off the machine and used to operate the machine to move up and down.

DETAIL OF THE SWITCH BOARD

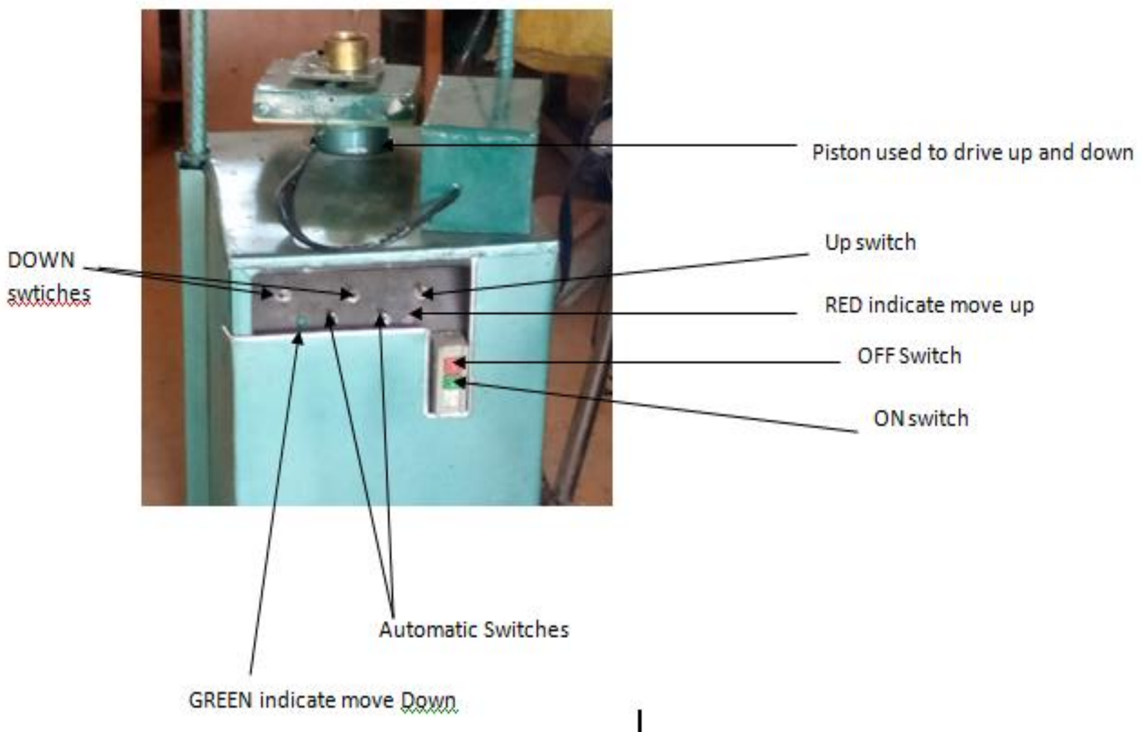

Figure 3.4(b): Detail of switch board for quasi static cone penetrometer

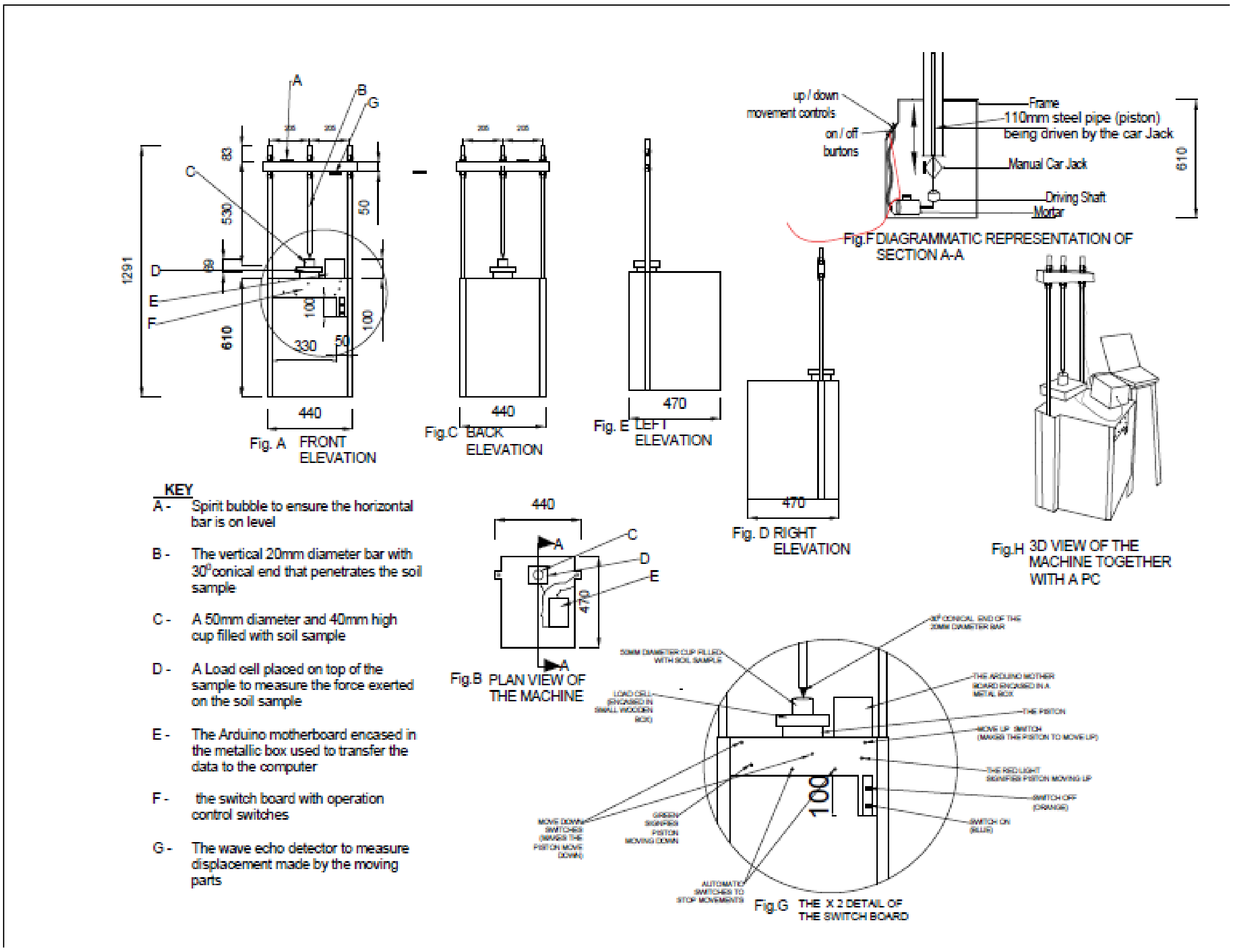

Figure 3.3 (c) Systematic drawing for Quasi static machine

Similar to the fall-cone tests, the quasi-static cone test protocols are also detailed by Stone and Phan (1995), Kyambadde (2003), and Stone and Kyambadde (2005 and 2007).

Since it was a new fabricated machine, the research was generally concentrated on testing at moisture contents around the plastic limit and Liquid limit.

3.3.3.1. Calibrating the Quasi static Machine

Calibration of the machine started after the load cell, HX711 amplifier and the Arduino board are connected and the software (HX711 library) installed. Care was taken in setting up the scale (two plates were wired on opposite ends on the load cell). The calibration factor was then determined using an object whose mass was known. The code was uploaded from the HX711 library while taking into account the recommendations for calibrating the load cell that were included in the library documentation. After uploading, the reset button on the Arduino board was pressed while the serial monitor was opened at a baud rate of 57600. When there was no load on the scale, instructions were followed on the serial display until the machine tare automatically (constant reading). Then, a known-weight object (500g) was placed on the scale, and you waited until you got a steady reading. The reading in this instance was 600g. Then the formula was used to get the calibration factor as indicated below;

Calibrating factor = reading weight/known weight.

In this case the calibrating factor =600/500=1.2

This factor was served because we may need it in the future. After calibrating the load cell, then the machine was ready to be used.

3.4: To determine quasi static consistency limits of soils of low to high plasticity

The quasi static cone penetration consisted of the 20mm diameter bar cone shaped similar to that of fall cone penetrometer attached to the frame similar to the frame of triaxial machine as shown in Figure 3.3. The frame was made in such a way that it moves the soil sample in a cup to be penetrated by a cone at constant penetration of 1.33mm/s. The penetration rate of 1.33mm/s was used because it has been proved that any penetration rate between 1mm/s-5mm/s there is no signification variation in penetration force (Stone and Phan, 1995). A load cell that was positioned at the base of the cylindrical cup holding the specimen was used to measure the force applied. The penetration was determined using Ultra sound sensor attached at the bottom of the frame. The load cell and ultra sound sensor were both connected to Urduino mother board which acts as data logger. The results were read directly from the computer using urduino mother board so as to improve in the accuracy of reading the penetration and force at the same time.

The approach was similar to that used by Kyambadde (2010), the only difference was that he used a data logger and Linear Voltage displacement traducers (LVDT) to read the results on the computer. Quasi penetration tests were carried out on each sample at plastic limit and liquid limit determined by conventionally using thread rolling and fall cone method respectively. A graph of force versus depth of penetration and Force versus penetration squared were plotted to get curves of different moisture content. Soil samples were mixed with moisture content equivalent to plastic and liquid limit respectively which were determined using thread rolling and fall cone penetrometer method. Forces corresponds to a 20mm depth of penetration were determined for both at plastic and liquid limit and these were the quasi consistency limits. A depth of 20mm was chosen basing on the assumption that it is in the middle of the cup used ,expect homogeneity of the sample in middle of the sample and also basing on fall cone penetration test as they consider 20mm depth of penetration. This was proved by the results where the forces at 20mm depth were almost the same compared to forces at depth of 10mm and 30mm as discussed in chapter four.

3.5. Determination Maximum Dry Density (MDD)

The MDD was determined by compaction the soil sample in a cylindrical mould using a standard compaction effort as BS 1377 (Part 4) [2) using a proctor test.

All soil particles larger than 20 mm were removed from dry soil using a sieve. A portion of the soil sample was then mixed with water and compacted in a mould measuring 1000 ml using a rammer made of a 2.5 kg mass that fell freely through a 300 mm opening. The soil was compacted in three equal layers, with each layer receiving 27 blows from the hammer.

Following compaction, the soil's bulk density and moisture content were assessed, and the dry density was then computed using the formula below.

$\rho_d = \dfrac{\rho}{1+w}$ Where ρ_d is the dry density, ρ =Bulky density and w = water content

The procedure was carried out at least five times for a specific soil sample, with the sample's moisture content rising each time. A curve of the kind depicted in Figure 3.1 was created by plotting a graph of dry density against moisture content. This curve was used to compute the maximum dry density (MDD), also known as the optimal moisture content (OMC).

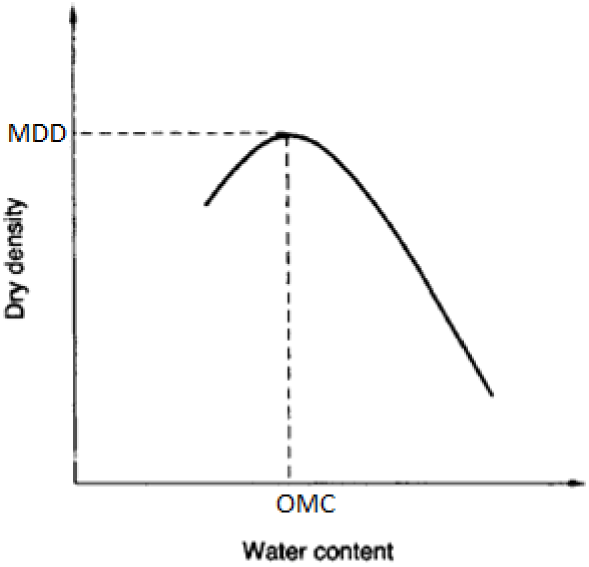

Figure 3.5: Show determination of maximum dry density at optimum moisture content

The aim of determining the optimum moisture content was to use the same amount of water when determining the California bearing ratio (CBR).

3.6. Determination the bearing Strength of the soil samples using CBR for given moisture content

3.6.1. CBR test procedures

CBR is the ratio of the force per unit area needed to penetrate a soil mass at a rate of 1.27 mm/min using a standard circular plunger with a 50 mm diameter to that needed to do the same in a standard material.

At penetration of 2.5 and 5mm, the ratio was calculated. The ratio at 5 mm is utilized when it consistently exceeds the ratio at 2.5 mm. There are two ways to compact soil samples in the CBR moulds:

i. Static Compaction method.
ii. Dynamic Compaction method.

However, CBR was determined using the static compaction method in accordance with BS 1377 part 9 at the maximum moisture content, which was also noted. The outcomes of the measured CBR are shown in Chapter 4, Table 4.4.

Figure 3.4 demonstrates the CBR device that was used to evaluate the soil samples' bearing strength.

Figure 3.6: California Bearing Ratio (CBR) Machine

3.7. Measurement of Variables

3.7.1: To develop the empirical relationship between conventionally derived consistency limits and CBR

Table 3.1 below shows that each sample, the laboratory testing yielded the plastic limit, liquid limit, optimum moisture content, maximum dry density, and CBR values. From the results a regression analysis was run and the empirical relationship between consistency limits and CBR was developed.

Table 3.1: Shows Consistency Limits, Optimum moisture content, maximum dry density and unsoaked California bearing ratio (CBR).

SAMPLE	Plasticity Index PI (%)	Plastic Limit PL(%)	Liquid Limit LL (%)	Optimum Moisture content (OMC) (%)	Dry Density (Kg/m^3)	CBR (%)
s2	12.5	15.5	28	11.4	1784	48.4
W7	12.5	25	37.5	10.9	1809	55.6
W4	12.5	27.3	39.8	7.5	1867	63.4
w1	12.5	30.9	43.4	12.8	2006	75.3
s12	12.5	31.5	44	11.3	1842	77.5
W10	12.5	27.3	39.8	8.9	1988	64.8
W13	12.8	15.8	28.6	7.8	1783	45.6
W15	12.9	15.9	28.8	11.2	1765	44.3
s6	13.2	18.5	31.7	38.7	2076	48.7
W17	13.6	20.3	33.9	16.3	2097	46.3
W19	13.7	18.1	31.8	15.8	1893	50.7
W21	13.7	26.4	40.2	19.4	2035	44.8
m5	13.8	20.6	34.4	10.4	1921	50.7
W22	13.9	23.9	37.8	15.2	1876	56.3
W2	13.9	24.9	38.8	15.7	1851	57.8
W5	14.3	25.3	39.6	19.3	1770	57.4
s1	14.4	29.2	43.5	9.5	2010	83.4
W8	14.7	31	45.7	9.3	2052	69.8
W11	14.8	35	49.8	12.7	1974	70.3
W14	16.3	35.5	51.8	12.6	1921	60.8
m7	16.6	28.6	44.85	11.5	1840	73.4
W18	16.7	25.6	42.3	15.5	1777	55.7

SAMPLE	Plasticity Index PI (%)	Plastic Limit PL(%)	Liquid Limit LL (%)	Optimum Moisture content (OMC) (%)	Dry Density (Kg/m^3)	CBR (%)
w16	16.7	33	49.7	20.5	1983	62.3
M1	16.8	22.4	39.2	14.5	1768	50.4
W20	16.8	28.6	45.4	16.5	2010	55.4
W3	16.9	33.4	50.3	17.9	1821	61.4
W6	17.3	28.1	45.4	12	2125	62.5
W9	16.9	33.4	50.3	16.4	1893	60.4
W12	17.6	22.9	40.5	16	1914	48.9
s11	18	25	43	35.5	1462	44.3
S10	18	26.7	44.7	10.7	1701	43.8
Y4	18	16.8	34.8	16.8	1625	33.5
M2	18.2	15.3	33.5	14.4	1746	25.15
Y6	18.3	16.3	34.6	16.6	1928	20.36
s5	18.4	14.4	35.8	15	1745	17.45
Y7	19	11.5	31.5	17.5	1753	16.36
Y2	19.1	10.9	30	17.1	1647	15.32
m10	19.1	12.8	31.9	15	1684	22.67
S7	19.3	22.5	41.8	17.2	1533	40.5
Y5	19.4	30.9	50.3	13.4	1615	46.67
s9	20	25.6	45.6	15.6	1797	48.07
Y11	20	25.6	45.6	11.4	1928	48.37
m6	22.1	24.2	46.3	13.8	1733	40.8
Y9	22.2	11	33.2	27.2	1630	9.3
S4	22.6	13.8	36.4	19.7	1662	14.4
Y3	22.8	22	44.8	19.1	1670	25.8
S3	23.1	19.8	42.9	19	1672	24.51
Y8	23.2	20	43.2	21.3	1596	27.21
S8	24.5	21.5	46	13.6	1627	26.3
Y1	25.8	18.5	44.3	13	1597	15.8

3.7.2 To develop the empirical relationship between Quasi static cone Penetrations and California bearing ratio

Basing on the results got from the quasi static penetrations and the CBR at consistency limits respectively, the empirical relationship was determined using a regression analysis. The results were presented in chapter four.

3.8. Summary of the chapter

The soil samples were collected, laboratory tests for consistency limits ,CBR ,OMC ,MDD and Quasi static consistency limits were conducted according to the required standards .The results were recorded and analyzed as presented in chapter four.

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 Introduction

Results are presented in this chapter were about the conventional BS fall-cone liquid limit and thread rolling plastic limit, consistency limits, California Bearing ratio tests, and quasi-static cone penetration tests. The quasi-static cone penetration tests refer to those conducted with a fabricated mechanically driven device. Tables and Figures of the results are presented with in the chapter. In addition, analysis of the results for experimental testing programme reported in from section 4.1 to 4.4 is presented. This involves; (i) the development of alternative plasticity index parameters based on quasi-static cone penetration tests which provide upper and lower strength indices similar to the conventional BS 1377 liquid limit (LL) and plastic limit (PL), (ii) development of the empirical relationship between conventionally derived consistency limits and CBR and **(iii)** development of the empirical relationship between Quasi static cone Penetrations and California bearing ratio. Tables and figures are also presented.

4.2 California bearing Ratio Test (CBR)

Results of unsoaked CBR tests at optimum moisture content conducted for 50 samples comprised of soil collected from Masaka,Kaungu,Lwengo and Rakai District are presented.

4.3 Consistency Limit tests: Fall-cone and Thread rolling Plastic Limit Tests

4.3.1 Fall-cone Tests

Fall-cone tests were conducted for 50 test soils of different plasticity obtained from Masaka,Kaungu,Lwengo and Rakai District. According to the BS 1377-2: 1990 technique, the fall-cone tests were carried out generally over the complete plastic ranges of the soils in 79 cm3 specimen cups with the cone gently oiled to reduce adhesion effects. For the fall-cone tests conducted at soft consistency moisture contents (typically above plastic limit), results were plotted on graphs of fall-cone penetration depth versus moisture content relationships. From the graph, the fall-cone liquid limit (LL) was derived as the moisture contents at 20 mm penetration depths for the smooth 80g cone. The derived LL values varied between 28 and 50.3 as presented in Table 3.1 along with the thread rolling plastic limits (PL).

As shown in Table 3.1, 12 of the 50 samples (24%) tested soils lay within the low plasticity (liquid limits less than 35 %), 35 of the 50 samples (70%) of the tested soil sample lay with in intermediate (liquid limits between 35 - 50 %) and 3 of 50 tested soil sample (6%) lay within high plasticity (of liquid limits greater than 50 %) soils classification according to BS 14688-2: 2004.

4.3.2 Thread rolling Plastic Limit (PL)

Thread rolling plastic limit tests were undertaken for 50 test soils comprising of soils from Masaka,Lwengo ,Kalungu and Rakai District following the procedure set out in part 2 of BS 1377, and also outlined in section 3.3.1 of chapter 3. For each of the tested soils, at least four tests were carried out and the average taken as the conventional plastic limit, PL. The PL ranged between 11 and 36, and is presented in Table. 3.1

4.4 Quasi-static Cone Tests

The results of the quasi-static cone tests that led to the development of the quasi-static liquid and plastic limits are presented in this section. Also presented are results of preliminary investigations on quasi-static cone penetration load versus depth relationships.

4.4.1 Quasi-static Cone Tests for Fine Soils

This was done using the fabricated mechanically driven cone devices outlined in chapter 3 (see section 3.3.2). Quasi-static cone tests were conducted for various soils of different plastic ranges. The results are presented in Table 4.1 and other graphs plotted to come up with the results in Table 4.1 are shown in Appendix F.

Table 4.1: Table showing soil samples and quasi static force at penetration of 30mm, 20mm and 10mm plastic and liquid limits

Sample	PLASTIC LIMIT			LIQUID LIMIT			PI (%)	PL (%)	LL (%)
	force (N) at 10mm (X 0.00981)	Force (N) at 20mm (X 0.00981)	force (N) at 30mm (X 0.00981)	Force (N) at 10mm (X 0.00981)	Force (N) at 20mm (X 0.00981)	Force (N) at 30mm (X 0.00981)			
s12	389.1	1155.6	2433.1	8.8	60.6	176.3	12.5	31.5	44.0
w1	379.4	1159.8	2461.0	9.7	64.2	155.2	12.5	30.9	43.4
W4	326.3	991.7	2100.7	28.6	57.4	105.4	12.5	27.3	39.8
W7	312.1	1009.3	2171.3	24.9	55.8	107.3	12.5	25.0	37.5
s2	327.4	1081.9	2339.4	6.2	63.8	159.7	12.5	15.5	28.0
W10	260.7	994.9	2218.4	24.7	57.4	109.5	12.6	15.6	28.3
W13	297.5	1006.4	2187.9	23.1	58.2	116.7	12.8	15.8	28.6
W15	303.8	990.5	2135.0	25.5	59.7	116.7	12.9	15.9	28.8
s6	179.8	902.5	2299.0	23.4	54.1	105.1	13.2	18.5	31.7
W17	179.8	902.5	2299.0	36.0	59.4	104.8	13.6	20.3	33.9
W19	258.0	1002.9	2244.4	32.5	58.9	102.9	13.7	18.1	31.8
W21	220.0	976.6	2237.6	34.7	59.9	101.9	13.7	26.4	40.2
m5	191.8	999.7	2346.2	5.9	59.9	150.1	13.8	20.6	34.4
W22	267.1	1054.6	2367.1	31.9	63.1	115.1	13.9	23.9	37.8
W2	317.9	997.4	2129.9	31.9	63.1	115.1	13.9	24.9	38.8
W5	226.9	981.1	2238.1	28.4	59.9	112.4	14.3	25.3	39.6
s1	215.3	1176.2	2777.7	36.0	52.5	79.4	14.4	29.2	43.5
W8	184.5	1068.3	2541.3	41.0	61.8	96.3	14.7	31.0	45.7
W11	223.0	984.7	2254.2	37.2	60.0	98.0	14.8	35.0	49.8
W14	219.6	1013.4	2336.4	34.4	60.2	103.2	16.3	35.5	51.8
m7	451.5	1078.5	2123.5	10.6	40.5	90.4	16.6	18.6	34.9
w16	385.4	1003.5	2033.5	27.7	60.1	114.1	16.7	33.0	49.7
W18	388.1	959.9	1912.9	26.2	59.2	114.2	16.7	25.6	42.3

Sample	PLASTIC LIMIT			LIQUID LIMIT			PI (%)	PL (%)	LL (%)
	force (N) at 10mm (X 0.00981)	Force (N) at 20mm (X 0.00981)	force (N) at 30mm (X 0.00981)	Force (N) at 10mm (X 0.00981)	Force (N) at 20mm (X 0.00981)	Force (N) at 30mm (X 0.00981)			
W20	438.4	1012.9	1967.2	30.2	59.3	107.8	16.8	28.6	45.4
M1	194.1	1126.5	2680.5	31.5	63.2	115.9	16.8	22.4	39.2
W3	184.8	996.0	2348.0	34.0	62.2	109.2	16.9	33.4	50.3
W6	135.2	967.1	2353.6	32.3	59.6	105.1	17.3	28.1	45.4
W9	227.9	995.0	2273.5	32.5	60.1	106.1	17.5	25.9	43.3
W12	208.9	1055.2	2465.7	31.4	61.7	112.2	17.6	22.9	40.5
s11	120.8	983.3	2420.8	24.8	61.7	123.6	18.0	25.0	43.0
S10	280.7	1010.3	2226.3	33.3	62.9	112.3	18.0	26.8	44.8
Y4	234.4	997.0	2268.0	32.1	61.2	109.7	18.2	15.3	33.5
M2	60.9	978.3	2507.3	30.5	62.6	116.6	18.3	23.3	41.6
Y6	120.3	931.2	2282.3	27.4	57.7	108.2	18.4	24.4	42.8
s5	56.0	1023.2	2635.2	24.7	55.0	105.4	19.0	19.5	38.5
Y7	149.1	990.6	2393.1	28.2	56.1	102.6	19.1	19.8	38.9
m10	41.1	1038.1	2699.6	32.6	53.2	87.5	19.1	10.9	30.0
Y2	104.0	1002.2	2499.6	31.1	60.8	110.3	19.3	22.5	41.8
S7	106.7	1064.5	2661.1	33.9	59.3	101.7	19.4	19.8	39.2
Y5	148.7	1026.5	2489.5	32.6	59.9	105.4	20.0	18.6	38.6
s9	176.9	1144.4	2756.0	31.2	50.1	81.6	21.6	23.6	45.2
Y11	277.3	1009.0	2228.1	30.2	64.4	121.4	22.1	24.2	46.3
m6	147.7	1138.5	2790.1	24.8	61.7	123.6	22.2	11.0	33.2
Y9	217.1	997.1	2297.1	32.8	63.1	113.6	22.6	13.8	36.4
S4	215.1	982.8	2262.3	24.2	62.3	125.8	22.8	22.0	44.8
Y3	297.2	1009.7	2197.2	30.5	64.1	120.1	23.1	19.8	42.9
S3	299.2	1015.3	2208.8	31.9	60.2	107.4	23.2	20.0	43.2
Y8	293.0	999.8	2177.8	39.6	63.3	102.8	24.5	21.5	46.0
S8	235.0	995.8	2263.8	23.3	60.7	123.0	25.8	24.5	50.3
Y1	309.0	1010.4	2179.4	29.1	63.5	118.7	26.0	23.7	49.7
Average	**234.67**	**1019.85**	**2336.72**	**27.80**	**59.58**	**112.60**			

4.4.2 Preliminary tests

The force at each depth of penetration was recorded and penetration force versus depth curves were plotted for all the soil samples. Atypical plot is shown in Figure 4.1and 4.2.

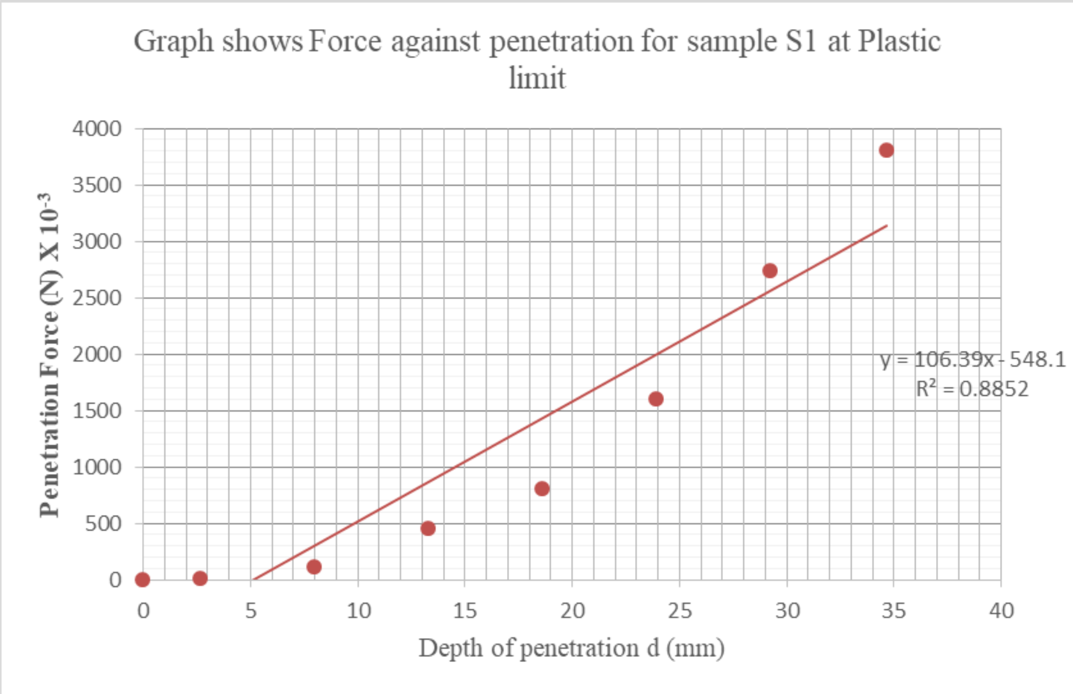

Figure 4.1: Penetration force against depth of penetration at plastic limit

From Figure 4.1 the points seem to trace a parabola, and the best-fit line leaves a number of them. Therefore a new plot of penetration force Verus depth square was made .This is shown in Figure 4.2

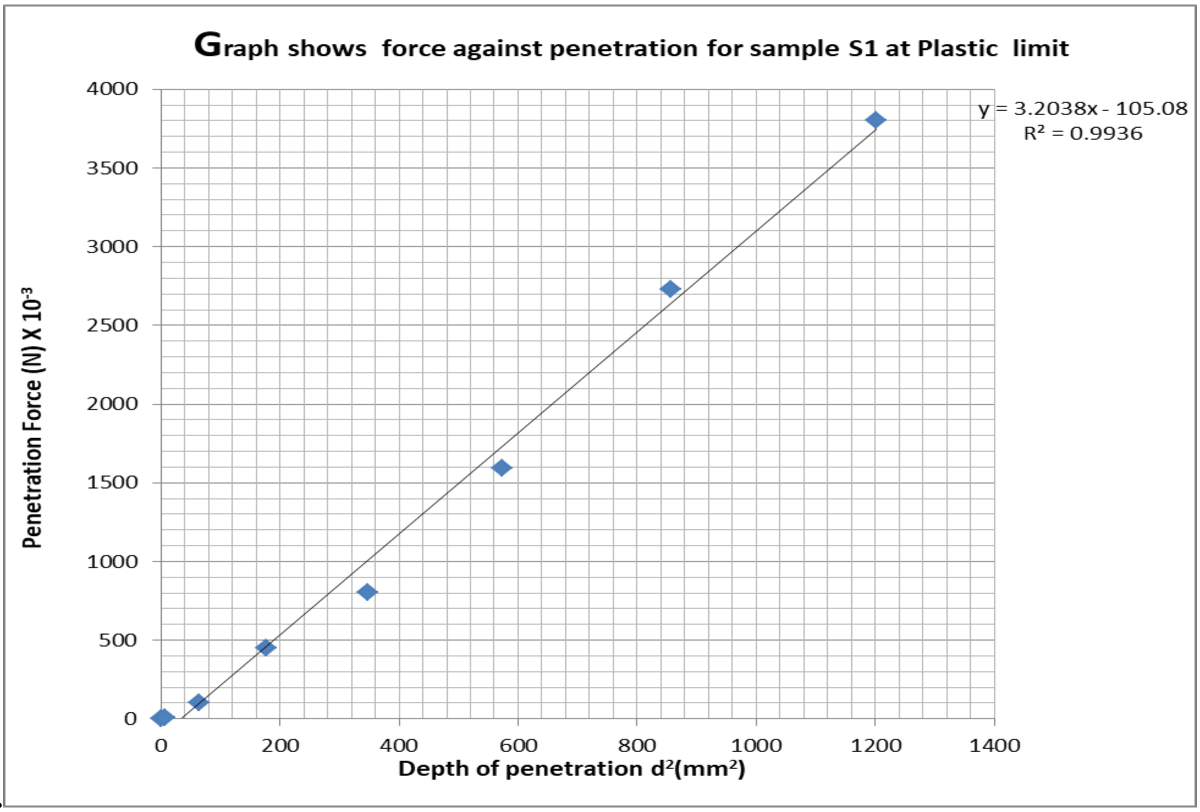

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Figure 4.2: Force against Depeth of penetration square at plastic limit

From Figure 4.2 it is seen that penetration Force versus depth square gives the best plot. This is This is because the cone's vertical force is directly proportional to the square of the depth (Koumoto and Houlsby,2001).

Ie $\frac{Q}{S_u h^2} = f\left(\frac{\gamma h}{s_u}, \beta, \alpha\right)$

where Q is the vertical force exerted on the soil, S_u is the undrained shear strength, β is the apex angle of the cone , γ is the density of the soil and α is the surface properties of the cone

This is also support by Hansbo,(1957) who stated that the shear strength of the soil is directly proportion to weight of the cone divide by the depth of penetration squared.

ie $\tau_f = KQ/h^2$.

Where τ_f is the undrained shear strength, K is the cone factor, Q is the weight of the cone and h is the depth of penetration.

Therefore, the 50 tests for penetration were plotted by Penetration Force versus depth squared.

However, for some of the soils it was observed that some of the loads versus depth squared relationships were not entirely linear. This may be, among others factors, attributed to the load recording mechanism (load-cell) of the quasi-static cone devices. However, even with the curvature in the load versus depth squared relationships, linear regression curves indicated correlation factors (R) generally above 0.92.From the graph of force versus penetration squared, the force corresponding to penetration of 10mm, 20mm and 30mm were determined for all the 50 samples. The aim was to determine which penetration depth matches the drop cone at plastic Limit and Liquid Limit.

Figure 4.3 and 4.4 show a plot of force at penetration depth versus corresponding plastic limit and liquid limit.

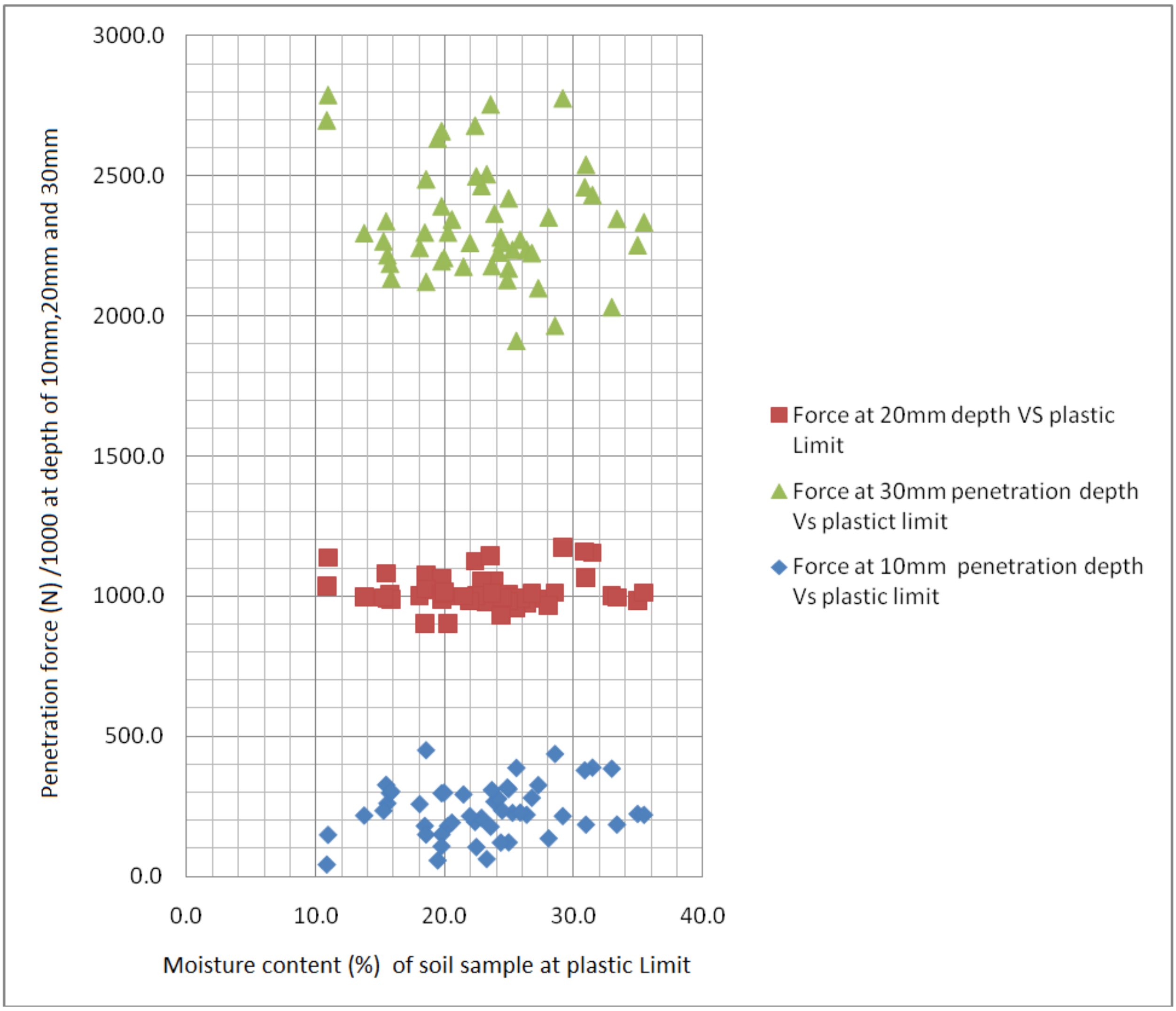

Figure 4.3 Shows penetration force at different depth against moisture content at plastic limit

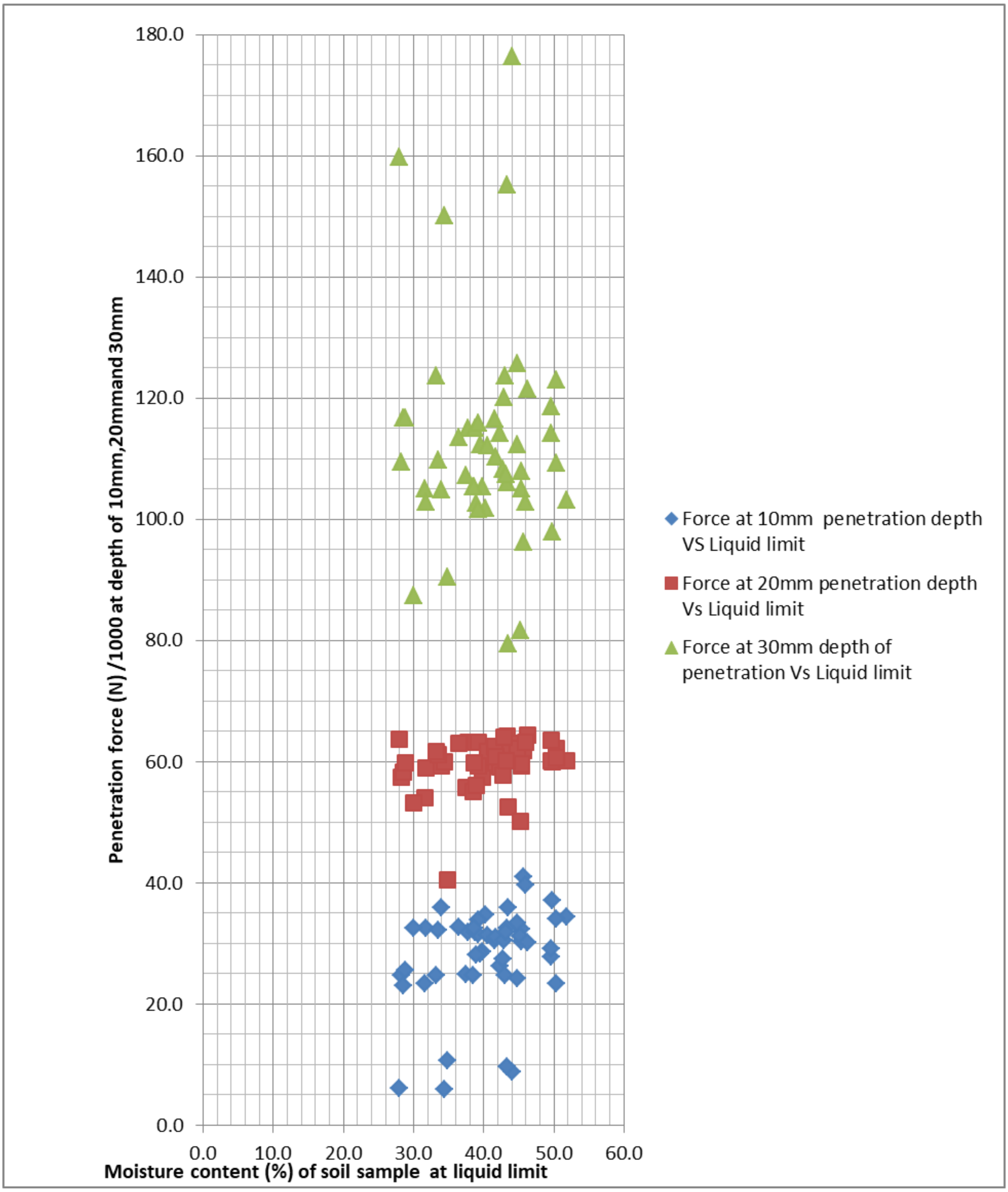

Figure 4.4: Showing a graph of force at penetration depth of 30mm,20mm and 10mm against moisture content of soil samples at liquid limit

From the Figure 4.3 it is seen that the accuracy is most at penetration depth of 20mm .This is because the penetration force over many different plastic limit values are too close with a narrow range ($902.5x10^{-3}$-$1176.2x10^{-3}$N) whereas for 10mm penetration it varies from ($41.1x10^{-3}$-$451.5x10^{-3}$N)and for 30mm ,it varies widely from ($1912.9x10^{-3}$-$2790.1x10^{-3}$N).

Similarly from Figure 4.4 it can also be seen that the accuracy was most at penetration of 20mm.This was because the penetration force over many liquid limit values were too close with a narrow range ($40.5x10^{-3}$-$64.5x10^{-3}$N),f or 10mm penetration it varied from ($8.8x10^{-3}$-$36x10^{-3}$N)and for 30mm ,it varied widely from ($81.6x10^{-3}$-$176.3x10^{-3}$N). Therefore the 20mm values were to be used for determination of quasi static force for both plastic and liquid limit.

4.5 Determination Quasi static consistency limits

This was done using the fabricated mechanically driven cone devices outlined in chapter 3 (see section 3.3.2). Quasi-static cone tests were conducted for various soils of different plastic ranges. Table 4.1 shows the samples used to develop the relationships.

4.5. 20mm depth Quasi-static Cone Penetration load at LL (LL_{qc})

Denoted LL_{qc}, the quasi-static cone load associated with LL is obtained from the load versus penetration depth squared at liquid limit in Appendix F. For 50 test soils, **LL_{qc}** ranged from $40.5x10^{3}$Nto $64.21x10^{-3}$N: overall averaging about $59.58X10^{-3}$ N. The above quasi static force of $59.58x10^{-3}$N is approximately similar to Swedish fall cone of 60g. Therefore the **LL_{qc}** at $60x10^{-3}$N and can be denoted by QL_{60}.

Therefore, QL_{60} (Quasi static liquid limit) can be described as the moisture content in the soil sample at which the quasi static force of 0.06N can penetrate a soil sample up to a depth of 20mm.

4.5.1. 20mm depth Quasi-static Cone Penetration load at PL (PL_{qc})

Denoted PL_{qc}, the quasi-static cone load associated with PL is obtained from the load versus penetration depth squared at Plastic limit in Figure 4.2. For thirty (30) test soils, PL_{qc} ranged from 902.51×10^{-3}N to 1176.2×10^{-3}N: overall averaging about 1019.85×10^{-3}N.Values of PL_{qc} are summarized in Table 4.1.

Therefore the PL_{qc} is $1020. \times 10^{-3}$ N and can be denoted by QP_{1020}.

Therefore, QP_{1020} (Quasi static Plastic limit) can be defined as the moisture content in the soil sample at which the quasi static force of 1N can penetrate a soil sample up to a depth of 20mm.

4.6. Correlations of Conventional Plasticity Index Parameters and California Bearing Ratio

Figure. 4.5 to Figure 4.7 show relationships between the California bearing ratio and moisture contents normalized to different index parameters namely; fall-cone liquid limit (LL); see Figure. 4.5, thread rolling plastic limit (PL); see Figure. 4.6 and plasticity index (PI); see Figure 4.7 and their multi relationships as shown from Table 4.3 to 4.6.

4.6.1 Correlation between CBR and Liquid limit (LL)

After comparing CBR with LL, the regression analysis is presented mathematically 4.1:

CBR= 1.244LL -4.003 with $R^2 = 0.187$. ……………………………………..Equation (4.1)

Therefore, LL can be used to explain 18.7% of the variation in CBR. The statistical output's specifics show that the association between liquid limit and CBR that has been developed is not statistically significant ($\alpha > 0.05$). This suggests that for all soil samples, there is a modest correlation between LL and CBR. According to the calculated Pearson's correlation coefficient (R), A very weak indicator of unsoaked CBR is liquid limit. According to the relationship shown above, CBR slightly rises as the liquid limit does.

Figure 4.5 and Table 4.3 below shows the relationship in equation (4.1)

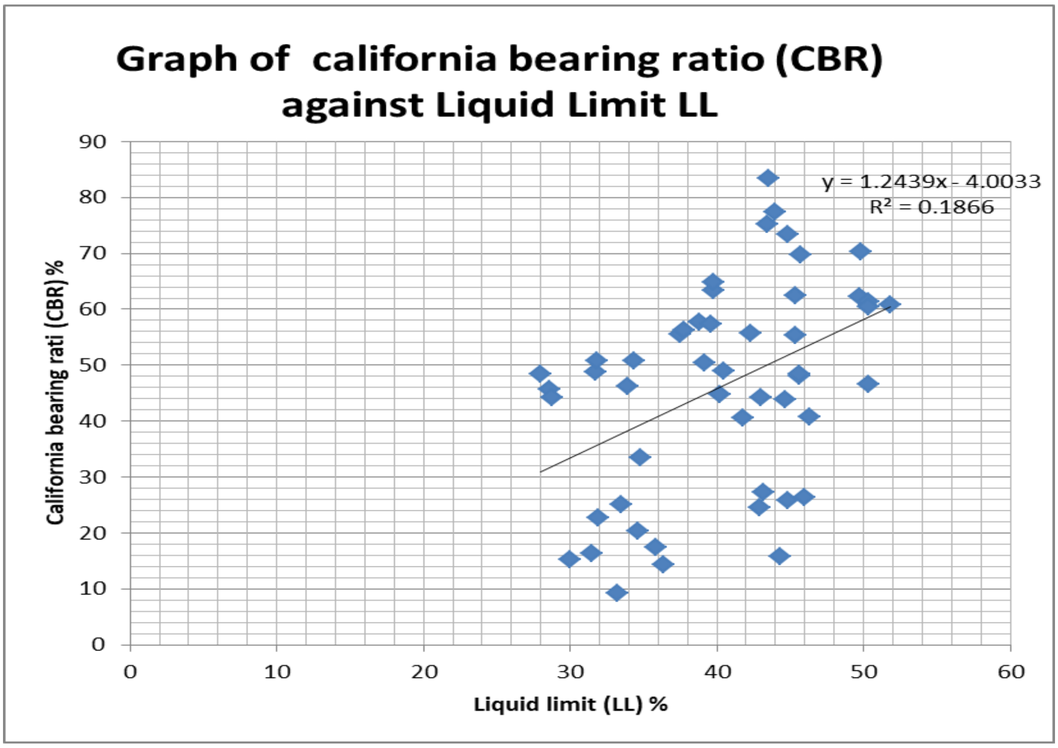

Figure 4.5: Shows relationship between CBR and LL

Table 4.2: Shows regression matrix for CBR against LL

Regression Statistics	
Multiple R	0.4320
R Square	0.1866
Adjusted R Square	0.1661
Standard Error	18.2884
Observations	50.0000

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	3599.97	3599.97	10.7634	0.00193
Residual	48	16054.3	334.466		
Total	49	19654.3			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	-4.003	16.440	-0.478	0.635	-40.918	25.190	-40.918	25.190
LL	1.244	0.402	3.281	0.002	0.511	2.129	0.511	2.129

4.6.2: Model 2: Correlation between CBR and Plastic limit (PL)

Equation 4.2 represents the regression analysis after comparing CBR with PL.

CBR= 2.285PL -6.797. with $R^2 = 0.669$. …………………………………..Equation (4.2)

Therefore, PL can explain 66.9% of the variation in CBR. The statistical output data show that there is a statistically significant association between the plastic limit and CBR ($\alpha > 0.05$). This suggests that the plastic limit and CBR are related. According to the calculated Pearson's correlation coefficient (R), the plastic limit is a predictor for unsoaked CBR. The above relationship indicate shows that as soils with high plastic limit plastic limit had high unsoaked CBR. Figure 4.6 and Table 4.4 below shows the relationship in Equation (4.2)

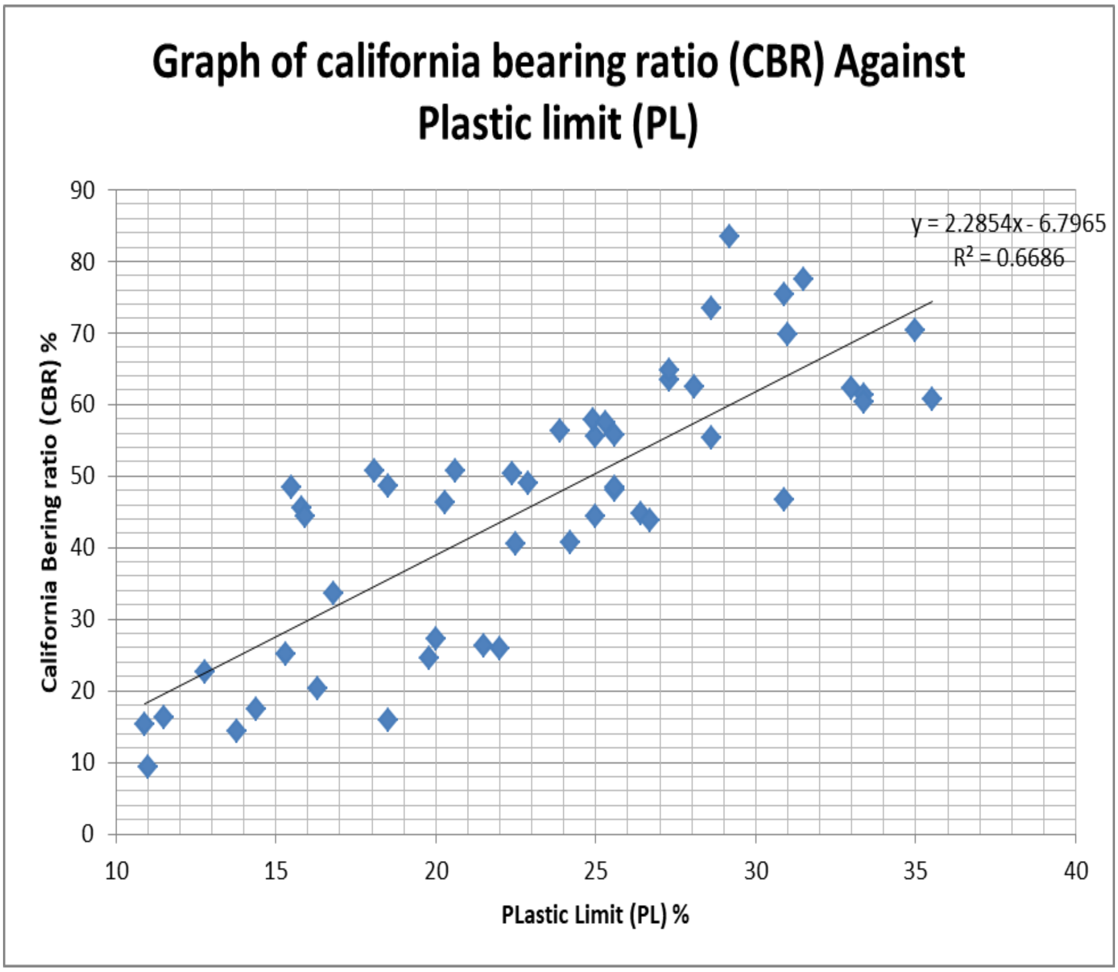

Figure 4.6: Shows relationship between CBR and PL

Table 4.3: shows regression matrix for CBR against PL

Regression Statistics	
Multiple R	0.817
R Square	0.669
Adjusted R Square	0.624
Standard Error	12.282
Observations	50

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	12413.81	12413.81	82.29581	5.56E-12
Residual	48	7240.504	150.8438		
Total	49	19654.32			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	-6.797	6.324	-1.544	0.129	-22.480	2.951	-22.480	2.951
PL	2.285	0.262	9.072	0.000	1.852	2.907	1.852	2.907

4.6.3 Model 3: Correlation between CBR and Plasticity index (PI)

Equation 4.3 represents the regression analysis following the correlation of the CBR with the plasticity index;

CBR= - 3.636PI+ 108.300 with, $R^2 = 0.491$…………………………………..Equation (4.3)

Consequently, the dependent variable is predicted by the PI. The statistical output details show that there is a considerable correlation between the plasticity index and CBR ($\alpha < 0.05$). This suggests a connection between PI and CBR. The plasticity index is a predictor for unsoaked CBR, according to the Pearson's correlation coefficient (R) that was obtained.

The above relationship indicates that as the PI increase the CBR reduces which is in agreement with Zumrawi, (2014), and Igbal, Kumar and Murtaza ,(2018). Figure 4.7 and Table 4.4 below shows the relationship in Equation (4.3)

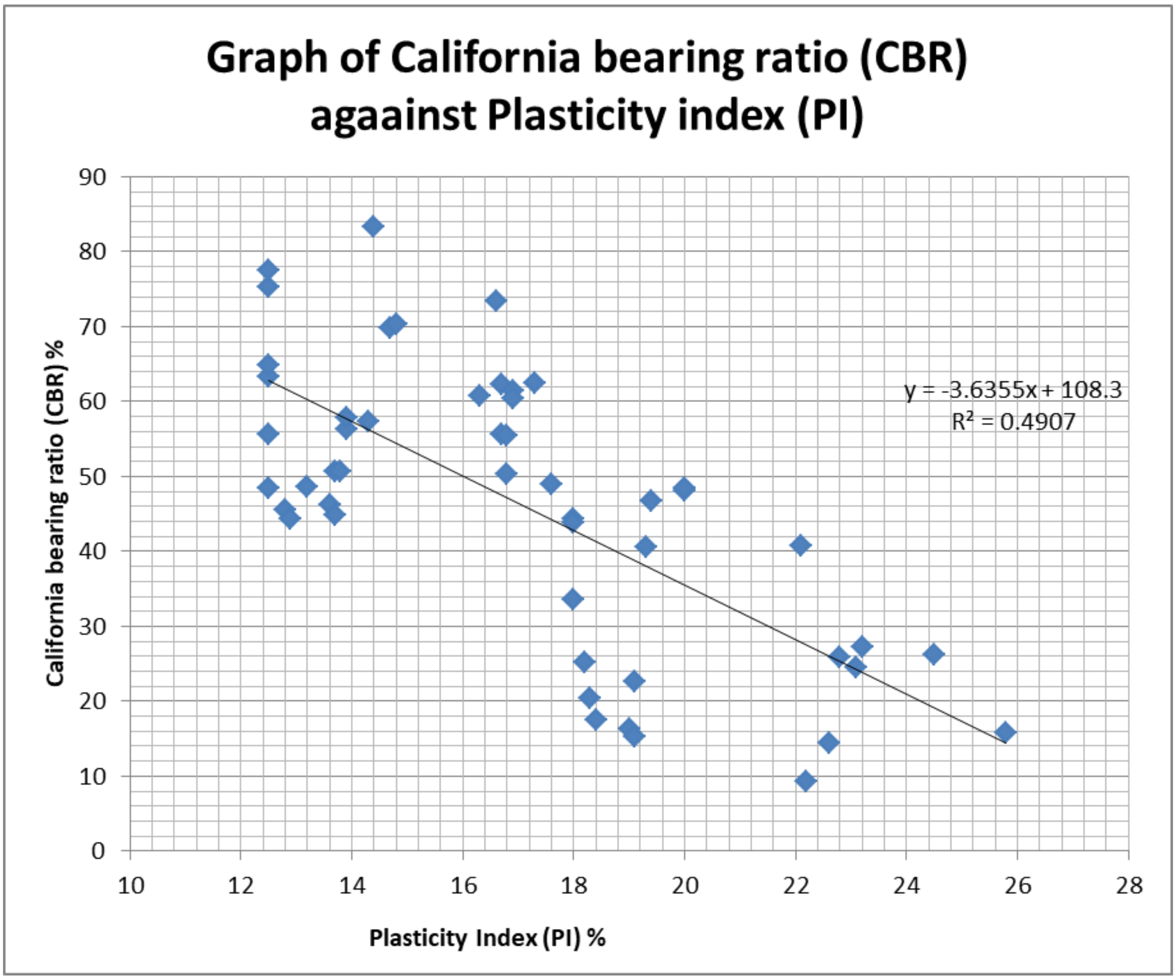

Figure 4.7: Graph of CBR Vs PI

Table 4.4: Shows regression matrix for CBR against PI

Regression Statistics	
Multiple R	0.700
R Square	0.491
Adjusted R Square	0.422
Standard Error	15.221
Observations	50

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	8533.975	8533.975	36.836	0.000
Residual	48	11120.34	231.6738		
Total	49	19654.32			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	108.300	10.538	10.249	0.000	86.820	129.196	86.820	129.196
PI	-3.636	0.604	-6.069	0.000	-4.878	-2.450	-4.878	-2.450

4.6.4 Multiple Linear Regression Analysis

Thirty samples (n = 50) were subjected to a multiple linear regression analysis, and the following results were attained after a variety of potential predictor combinations were tested.

4.6.5. Model 4: Liquid limit, Plastic index, and CBR Correlation

Equation gives the coefficients for the derived regression model, which is a single linear expression (4.4)

CBR= -2.601PI + 1.868PL + 47.340. R^2 = 0.897. …………………………... Equation (4.4)

As a result, the independent variables can explain 89.7% of the variance in CBR. According to the specifics of the statistical output, there is a statistically significant association between the plasticity index, plastic limit, and CBR ($\alpha < 0.05$). The above relationship indicate that as the plastic limit increases the CBR increase this is because as the plastic limit increases the plasticity index reduces hence the CBR increase as seen in 4.6.6. Table 4.4 explains more of the relationship in Equation (4.4)

Table 4.5: Shows regression matrix for CBR against PI and PL

Regression Statistics	
Multiple R	0.947
R Square	0.897
Adjusted R Square	0.893
Standard Error	6.112
Observations	50.000

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	2	15368.36	7684.182	205.6878	5.7E-24
Residual	47	1755.848	37.35847		
Total	49	17124.21			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	47.340	6.151	7.696	0.000	34.966	59.715	34.966	59.715
PI	-2.601	0.254	-10.243	0.000	-3.112	-2.091	-3.112	-2.091
PL	1.868	0.137	13.655	0.000	1.593	2.143	1.593	2.143

4.6.6. Model 5: Correlation between CBR and Plastic index and Liquid limit

Equation (4.5) below gives the coefficients for the multiple linear expression that makes up the regression model that was obtained;

CBR= 1.960LL – 4.557PI + 44.216. , R^2 = 0.812. …………………………... Equation (4.5)

As a result, the independent variable can explain 81.2% of the variance in CBR. The statistical findings demonstrate that there is a statistically significant relationship between the plasticity index, the liquid limit, and the CBR ($\alpha < 0.05$). The above relationship indicate that as the Liquid limit increases the CBR reduces this is because as the liquid limit increases the plasticity index increase hence the CBR reduces as seen in 4.6.6.The relationship is also similar to that of (Igbal ,Kumar and Murtaza ,2018). (4.5).

Table 4.6: Shows regression matrix for CBR against PI and LL

Regression Statistics	
Multiple R	0.901
R Square	0.812
Adjusted R Square	0.804
Standard Error	8.869
Observations	50

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	2	15957.05	7978.527	101.4239	0.000
Residual	47	3697.264	78.66519		
Total	49	19654.32			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	44.216	8.991	4.918	0.000	26.129	62.303	26.129	62.303
LL	1.960	0.202	9.714	0.000	1.554	2.365	1.554	2.365
PI	-4.557	0.364	-12.533	0.000	-5.288	-3.826	-5.288	-3.826

4.6.7. Model 6: Correlation between CBR and Plastic Limit and Liquid limit

Equation (4.6) below gives the coefficients for the multiple linear expression that makes up the regression model that was obtained;

CBR= 4.566PL-2.650LL+46.705. , R^2 = 0.841. …………… ………………..Equation (4.6)

As a result, the independent variables can explain 84.1% of the variance in CBR. The statistical findings demonstrate that the plastic limit, liquid limit, and CBR are statistically significantly correlated ($\alpha < 0.05$). The above relationship indicate that as the Liquid limit decreases the CBR increase this is because as the liquid limit decreases the plasticity index decrease hence the CBR increases as seen in 4.6.3.Table 4.6 explains more of the above relationship in Equation (4.6)

Table 4.7: Shows regression matrix for CBR against PL and LL

Regression Statistics								
Multiple R								0.917
R Square								0.841
Adjusted R Square								0.835
Standard Error								8.142
Observations								50
ANOVA								
				df	*SS*	*MS*	*F*	*Significance F*
Regression				2	16538.845	8269.402	124.751	0.000
Residual				47	3115.515	66.28755		
Total				49	19654.320			
	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	46.705	8.296	5.630	0.000	30.016	63.394	30.016	63.394
PL	4.556	0.326	13.971	0.000	3.900	5.212	3.900	5.212
LL	-2.650	0.336	-7.889	0.000	-3.326	-1.974	-3.326	-1.974

4.6.8. Model 7: Correlation between CBR and Plasticity Index, Plastic Limit and Liquid limit

The Equation following gives the coefficients for the multiple linear expression that makes up the regression model that was obtained;

CBR= 1.645PI+ 6.040PL – 4.250LL + 49.534 , $R^2 = 0.907$. ……………….Equation (4.7)

As a result, the independent variables can explain 90.7% of the variance in CBR. The statistical output shows that the plasticity index, plastic limit, liquid limit, and CBR significantly correlate with one another ($\alpha > 0.05$).

The above relationship indicate that the plastic limit and liquid limit are the major determinant of CBR which actually true because plasticity index which is another determinant depends on liquid and plastic limits**.** Table 4.7 explains more of the above relationship in equation (4.7).

Table 4. 8: shows regression matrix for CBR against PL, LL and PI

Regression Statistics	
Multiple R	0.952
R Square	0.907
Adjusted R Square	0.901
Standard Error	5.881
Observations	50.000

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	3	15533.46	5177.819	149.7274	0.000
Residual	46	1590.755	34.582		
Total	49	17124.210			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	49.534	6.003	8.252	0.000	37.452	61.617	37.452	61.617
PI	1.645	1.959	0.840	0.405	-2.298	5.588	-2.298	5.588
PL	6.040	1.914	3.156	0.003	2.187	9.892	2.187	9.892
LL	-4.250	1.945	-2.185	0.034	-8.166	-0.335	-8.166	-0.335

4.7 Empirical rrelationship between CBR and Quasi static Consistency limits

Basing on the correlation between CBR and consistency limits, empirical relationships was developed basing on replacing quasi static consistency limits with conventionally derived consistency limits developed in equations 4.1 to4. 7 as follows;

4. 7.1 Empirical Relationship between CBR and Quasi static Liquid limits

From the equation 4.1 ; CBR= 1.244LL -4.003 the empirical relationship between the CBR and quasi static liquid limit was developed as shown in equation (4.8) below

$CBR = 1.244QL_{60} - 4.003$…………………………………… …………….Equation (4.8)

where

CBR= Unsoaked California Bearing ratio.

QL_{60}= Quasi static Liquid limit (Moisture content in soil sample at which the quasi static force of 0.06N penetrate the soil sample up to 20mm)

4. 7.2. Empirical Relationship between CBR and Quasi static Plastic limits

From the equation 4.2; CBR= 2.285PL -6.797. the empirical relationship between the CBR and quasi static plastic limit was developed as shown in equation (4.9) below

$CBR = 2.285QP_{1020} - 6.797$……………………………………………………Equation 4.9

where

CBR= Unsoaked California Bearing ratio.

QP_{1020}= Quasi static plastic limit (Moisture content in soil sample at which the quasi static force of 1N penetrate the soil sample up to 20mm)

4. 7.3.Empirical Relationship between CBR and Quasi static Plastic Index

From the equation 4.3; CBR= - 3.636PI+ 108.300 the empirical relationship between the CBR and quasi static plasticity index was developed as shown in equation (4.10) below

$CBR = -3.636QPI + 108.300$………………………………………………….Equation (4. 10)

where

CBR= Unsoaked California Bearing ration.

QPI = Quasi static plasticity index ($QPI = QL_{60} - QP_{1020}$)

4. 7.4. Empirical Relationship between CBR, Quasi static plastic limit and Quasi static Plastic Index

From the equation 4.4; **CBR=** -2.601**PI +** 1.868**PL +** 47.340 the empirical relationship between the CBR ,QP_{1020} and QPI was developed as shown in equation (4.11) below

CBR= -2.601QPI +1.868QP_{1020} +47.340…………………………………….. Equation (4.11)

4.7.5. Empirical Relationship between CBR, Quasi static Liquid Limit and Quasi static Plastic Index

From the equation 4.5; CBR= 1.966PI-4.557LL + 44.216 the empirical relationship between the CBR ,QP_{1020} and QPI was developed as shown in equation (4.12) below

CBR= 1.966QPI-4.557QL_{60} + 44.216……………………………………... Equation (4.12)

4.8. Validation of the Empirical equation 4.14

Data of soil tests for consistency limits and unsoaked CBR was obtained from R.S.V engineering group, a registered materials laboratory found in Uganda. The results were assumed to be correct results and were used to check the validation of the equation (4.14). Since equation (4.14) was similar to equation (4.7) because it was developed by replacing conventionally derived consistency limits with quasi-static consistency limits; equation 4.7 was used to validate the empirical relationship. Figure 4.8 displays a control graph between experimental and predicted values of CBR that was created using the sample validation Table described in Table 4.8. The intersection of experimental and anticipated CBR values is shown as a straight line. Nearly every point was located nearer the straight line. Only three or four of the points tended to veer from the path. Accordingly, it is possible to characterize the soil's strength at an early stage using the anticipated CBR values. As illustrated in Figure 4.8, a comparison graph was also drawn to confirm the validity of the developed connection. A discrepancy between the two curves was seen for soil samples 8, 10, and 11. This could be ascribed to mistakes made during the execution of the laboratory testing. A difference between the two CBR values may be seen on the graph. Both graphs generally exhibit the same pattern. Equation 4.7 provided the percentage variance for each CBR value in the sample.

Percentage Variation = $\left(\frac{Actual\ CBR - \Pr edicted\ CBR}{Actual}\right) \times 100$ ……………Equation 4.15

Table 4.8 provides the percentage difference between the actual and anticipated CBR using Equation 4.7. A good value demonstrating that the projected values of CBR did not considerably depart from experimental values is the average percentage variation derived from the model, which is 11.6%.

Therefore, if the quasi-static consistency limits were used we should expect to have more accurate values and for that case, equation 4.14 should have been used.

Table 4.9: shows data for soil samples obtained from R.S.V Engineering group and their predicted CBR values

Sample No.	Maximum dry density (MDD)	Optimum moisture content % (OMC)	Plasticity Index % (PI)	Plastic limit % (PL)	Liquid limit % (LL)	California bearing ratio % (CBR)	Predicted CBR % using Equation 4.7 (CBR_p)	Residual % ($CBR - CBR_p$)	$\left[\frac{CBR - CBR_P}{CBR}100\right]^2$
1	1.904	8.8	12.6	27	39.6	64	65.04	1.04	2.65
2	1.889	11.7	12.7	27.2	39.9	63.8	65.14	1.34	4.40
3	1.894	9.6	12.9	15.4	28.3	45	43.50	-1.50	11.18
4	1.849	10.6	13.9	20.1	34	52	49.30	-2.70	26.89
5	1.849	11.6	14.2	24.3	38.5	57	56.04	-0.96	2.84
6	1.907	8.2	14.1	31.2	45.3	69	68.65	-0.35	0.26
7	1.908	8.9	16.7	28.4	45.1	56	56.87	0.87	2.39
8	1.773	10.7	17.9	22.2	40.1	49	42.64	-6.36	168.34
9	1.775	12.6	25.1	18.7	43.8	16	17.62	1.62	102.71
10	1.786	11.2	17.4	16.5	33.9	25	33.74	8.74	1222.76
11	1.803	10.5	19.6	25.4	45	48	43.94	-4.06	71.47
12	1.808	9.7	12.9	15.8	28.7	44	44.21	0.21	0.23
Total $[\Sigma]$									1616.11
Average percentage $\sqrt{\Sigma/12}$									11.60

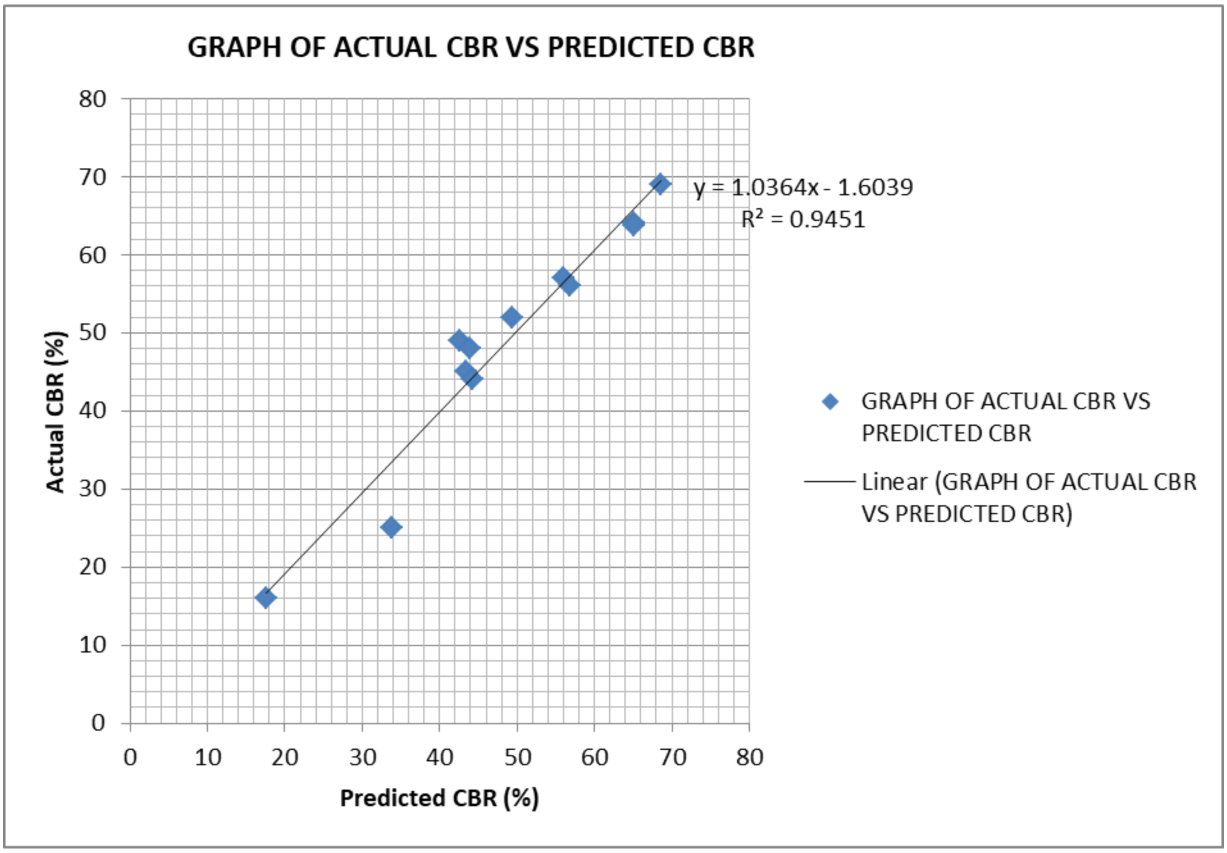

Figure 4.8: shows a graph of Actual CBR against Predicted CBR

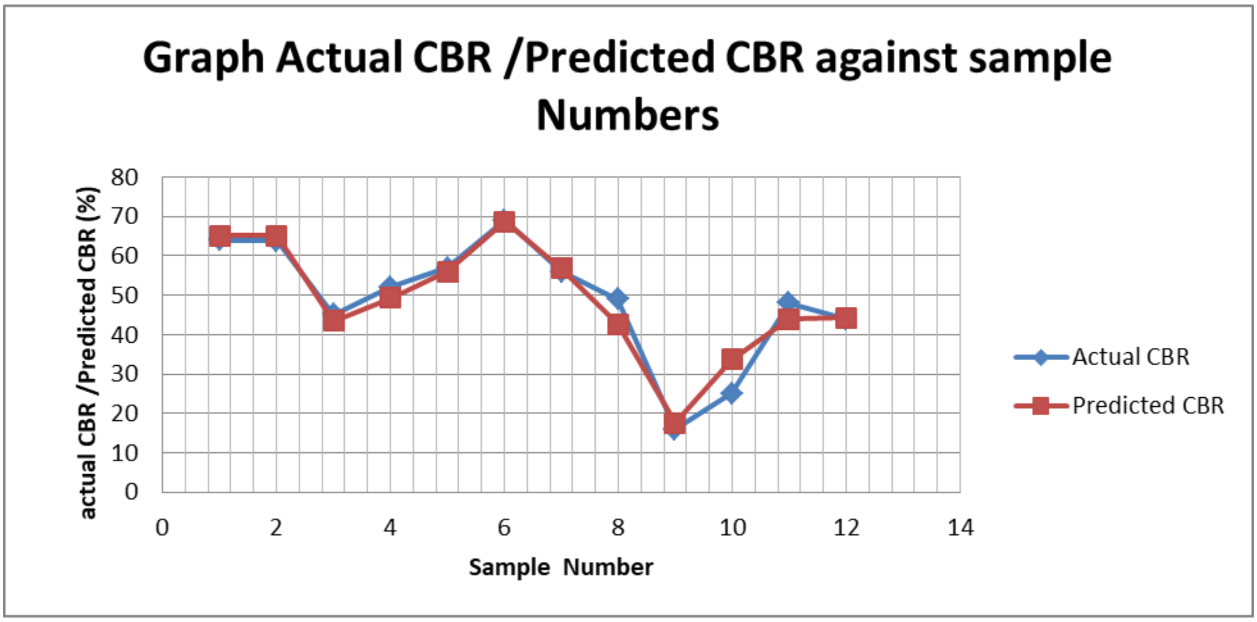

Figure 4.9: Showing comparison in a variation of Actual CBR and Predicted CBR.

4.8.1 Comparison of the developed empirical equation with previous equations

The following empirical formulae were developed as a result of study by Olumuyiwa and Ajibola (2017) on the association between the California bearing ratio value of clays and soil index and compaction characteristics; $UCBR = 32.638 - 0.570PI$; with $R^2 = 0.517$,from Equation (2.12) and $UCBR = 63.575 + 0.018MDD - 2.727OMC - 9.113 \times 10^{-6}MDD^2 + 0.024OMC^2$;With $R^2 = 0.940$,Equation (2.14) .Also Igbal et.al, (2018), carried out a research on correlation between California bearing ratio and index properties of Jamshoro soils and developed the relationship as ;

$CBRu=392.0103-2.7748(PI)-154.0842(MDD)$:$R^2=0.720$,from Equation(2.17)

The above equations were compared with the developed equation (4.7) as shown in Table 4.9 by using the tested results got from RSV engineering group. Based on the analysis of the results in Table 4.9, the following observations were made;

It was observed that the current empirical equation developed (4.7) was more accurate with an 11.6% average variation in results compared to other equations ie equation (2.12) had an average variation of 50.7%, equation (2.14) with an average variation of 38.2% and equation (2.17) had average variation of 85.3%.

Equation (2.12) showed smaller variations of 12.1% in CBR values below 35% this was because the above equation was developed with the CBR values between 8% and 35%. Therefore the above equation cannot be used to predict CBR values above 35% . The weakest of the above equation was indicated in section 2.6.

Equation (2.17) showed smaller variations of 8.2% in CBR values above 60% this was because the above equation was developed with the CBR values ranging between 65%-85%. Therefore the above equation can only be applied on soils with CBR above 60%. The weakest of this equation was also indicated in section 2.6.

Therefore the current equation 4.6 was a better equation than other developed equations since it can be applied to all ranges of CBR Values.

Table 4.10: shows comparison of developed empirical equations for prediction of unsoaked CBR.

Sample No.	Maximum dry density (MDD)	Optimum moisture content % (OMC)	Plasticity Index % (PI)	Plastic Limit % (PL)	Liquid limit % (LL)	California bearing ratio % (CBR) (a)	predicted CBR_p % using equation (2.12) for 2017 (b)	Residue (R_o) %	$\left[\frac{(a-b)}{a}\times100\right]^2$	predicted CBR % using equation (2.14) for 2017(c)	Residue (R1) %	$\left[\frac{(a-c)}{a}\times100\right]^2$	predicted CBR % using equation (2.17) for 2018 (d)	Residue (R2) %	$\left[\frac{(a-d)}{a}\times100\right]^2$	predicted CBR % using equation (4.7) (e)	Residual(R3) %	$\left[\frac{(a-e)}{a}\times100\right]^2$
1	1.904	8.8	12.6	27	39.6	64	25.456	38.544	3627.051	41.47	22.53	1356.40	64.05	-0.05	0.01	65.04	1.04	2.65
2	1.889	11.7	12.7	27.2	39.9	63.8	25.399	38.401	3622.795	34.99	28.81	2233.24	66.09	-2.29	12.84	65.14	1.34	4.40
3	1.894	9.6	12.9	15.4	28.3	45	25.285	19.715	1919.413	39.64	5.36	73.34	64.77	-19.77	1929.55	43.50	-1.50	11.18
4	1.849	10.6	13.9	20.1	34	52	24.715	27.285	2753.222	37.40	14.60	524.13	68.96	-16.96	1063.26	49.30	-2.70	26.89
5	1.849	11.6	14.2	24.3	38.5	57	24.544	32.456	3242.204	35.20	21.80	1336.16	68.13	-11.13	381.45	56.04	-0.96	2.84
6	1.907	8.2	14.1	31.2	45.3	69	24.601	44.399	4140.456	42.86	26.14	1397.01	59.47	9.53	190.75	68.65	-0.35	0.26
7	1.908	8.9	16.7	28.4	45.1	56	23.119	32.881	3447.577	41.24	14.76	778.65	52.18	3.82	46.54	56.87	0.87	2.39
8	1.773	10.7	17.9	22.2	40.1	49	22.435	26.565	2939.189	37.18	11.82	124.47	69.69	-20.69	1782.42	42.64	-6.36	168.34
9	1.775	12.6	25.1	18.7	43.8	16	18.331	-2.331	212.2485	33.06	-17.06	9306.80	49.62	-33.62	44143.11	17.62	1.62	102.71
10	1.786	11.2	17.4	16.5	33.9	25	22.72	2.28	83.1744	36.08	-11.08	87.11	69.06	-44.06	31055.54	33.74	8.74	1222.76
11	1.803	10.5	19.6	25.4	45	48	21.466	26.534	3055.786	37.62	10.38	173.47	60.40	-12.40	667.20	43.94	-4.06	71.47
12	1.808	9.7	12.9	15.8	28.7	44	25.285	18.715	1809.149	39.41	4.59	118.89	78.02	-34.02	5977.47	44.21	0.21	0.23
Total $[\Sigma]$									30852.26			17509.67			87250.14			1616.11
$\sqrt{\Sigma/12}$								Average percentage	50.7%			38.20%			85.3%			11.6%

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Introduction

Conclusions and recommendations are the two components of this chapter.

The section on conclusions provides general findings of the research in a concise manner basing on objectives and gives succinct answer to each specific objective. The conclusions are given for each specific objective.

The specific objectives were to:

i) Determine cohesive soils with low to high plasticity's traditionally derived consistency limits:

ii) Determine quasi static consistency limits of soils of low to high plasticity.

iii) Determine the bearing Strength of the soil samples using CBR for given moisture content.

iv) Develop the empirical relationship between conventionally derived consistency limits and CBR.

v) Develop the empirical relationship between Quasi static cone Penetrations and California bearing ratio.

From laboratory experiments, the following conclusions and recommendations have been made

5.2 Conclusions

1. An alternative quasi-static cone plastic limit (QPL_{1020}) was proposed and is defined as the moisture content corresponding to a 20mm quasi-static cone penetration depth for a penetration load of 1N .From the results it correlates closely with conventional thread rolling plastic limit for the 30 soils samples used.

2. Further, the quasi-static cone liquid limit (QLL_{60}) was proposed and is defined as the moisture content corresponding to a 20 mm quasi-static cone penetration depth for a penetration load of 0.06N. From the results obtained ,it correlates closely with conventional fall cone liquid limit for the 30 soils samples used.

3. Semi-empirical expressions were proposed for derivation of California bearing ratio and consistency limits. These semi-empirical relationships, presented in Figures. 4.5, 4.6, and 4.7, were based on correlations of California bearing ratio with liquid limits, Plastic limit and Plasticity index respectively,

4. Semi-empirical expressions were proposed for derivation of California bearing ratio and Quasi static consistency limits. The semi-empirical relationships presented in equations 4.8-4.14 was based on relationship quasi static: liquid limits, Plastic limit and Plasticity index and California bearing ratio,

5. Quasi-static cone tests provide a simple and straightforward method of determining California bearing ratio relationships, on applying appropriate consistency limits.

6. The plasticity index can be entirely directly determined with quasi-static cone tests, by determining the difference in moisture content between quasi static liquid limit (QL_{60}) and quasi static plastic limit (QP_{1020})

7. Consistency limits of mixtures directly obtained from quasi-static cone tests through derivation of their respective QL_{60}, QPL_{1020} and QPI are in reasonable agreement with suggested approaches used to modify the conventionally derived index parameters.

8. The California bearing ratio of cohesive soils may be estimated directly from quasi static cone tests using quasi static consistency limits. This approach is satisfactory with 90.7 % variance.

5.3. Recommendations

The following proposals are made for further development of the quasi-static cone approaches and consistency limits testing, for cohesive fine and mixed soils.

1. Undertaking extensive experimental programmes for soils of varied geology and plasticity, which may provide improvements of the alternative quasi-static cone approaches developed in this research.

2.Using the derived relationship between Unsoaked CBR and Soaked CBR for (Feleke and Araya,2016) and(Igbal ,Kumar and Murtaza ,2018) one can determine soaked CBR ounce the unsoaked CBR is known.

3. It is essential to evaluate consistency limits based on soaked CBR testing for consistency evaluations.

4. Further investigation should be undertaken on this fabricated Quasi static cone penetrometer to establish whether it can test the CBR directly without using correlations with consistency limit approach.

Reference

Braja.M.Das ,(2006),Principles of Geotechnical Engineering 7th Edition published by Cengage Learning Customer & Sales Support pp79-84.

Brownjr Thomas. H,(2016), Highway Engineering,2016.

BSI: Methods of Soil Testing for Civil Engineering Applications, Part 2 (BS 1377-2). British Standards Institution, London, 1990.

Casagrande, A, (1958), Notes on the design of the liquid limit device. Géotechnique **8**,pp 84-91.

Ehsan Kianirad,(2011),Development and Testing of a Portable In-Situ Near-Surface Soil Characterization System pp, 82 A Dissertation Presented to the Department of Civil and Environmental Engineering in partial fulfillment of the requirements for the degree of Doctor of Philosophy in the field of Geotechnical and Geo Environmental Engineering.

Eva Hrubesova, Barbara Lunackova and Ondrej Brodzki (2016), Comparison of Liquid Limit of Soils Resulted from CasagrandeTest and Modificated Cone Penetrometer Methodology, published by Elsevier ltd pp, 1,365

Feleke Gebremariam. G. and Alemgena A. Araya ,(2016),Prediction of CBR using DCP for local subgrade materials.

Gill, K.S. Jha, J.N. and Choudhary, A.K.[1],2010, CBR Value Estimation Using Dynamic Cone Penetrometer. Indian Geotechnical Conference – 2010, p,1.

Hansbo Sven,1957 A new approach for determination of shear strength of clay by fall cone test.Royal Swedith Geotechnical institute proceedings no.14. pp19.

Houlsby Guy Tinmouth,1982,Theoritical analysis of the fall cone test. University of Oxford, Geotechnique Volume 32;issue 2;page 111-118

Hovayni, P. 1958, A new grooving tool. *Géotechnique* **8**, 79.

IS 2720(Part 16):1987 Methods of test for soils: Laboratory determination of CBR (second revision).

Iqbal Faisal,Aned Kumar and Ali Murtaza ,(2018),Co-Relationship between California bearing ratio and index properties of Jamshoro soils. Published by Mehran University research journal of engineering and technology pp.185-190

Kaniz Roksana, Tabassum Islam and Abdul Muqtadir, 2018, Relationship between CBR and Soil Index Properties of Bangladesh Soil Samples**:** Journal of Modern Science and Technology Vol. 6. No. 2. June 2018 Issue. Pp.1-9

Katte Valentine Yato, Mfoyet Souleyman Moupe, Wouatong Armand Sylvain Ludovic , Manefouet Bertille and Bezeng Lawrence Aleh ,2018 ,Correlation of California Bearing Ratio (CBR) Value. Article in Geotechnical and Geological Engineering · June 2018. pp1-2

Kian Teck Teh, Salihuddin Hassim, Ratnasamy Muniandy, Husaini Omar and Azmi Hassan,2005, The development of road construction material selection system (RC-mss),published by Journal of the Eastern Asia Society for Transportation Studies, Vol. 6, pp. 1313 - 1328

Koumoto, T. & Houlsby, G. T. (2001). Theory and practice of the fall cone test,pp 701-712

Kyambadde Benjamin Senyonga,(2010) Soil strength and consistency limits from quasi-static cone tests, A thesis submitted in partial fulfillment of the requirements of the University of Brighton for the degree of Doctor of Philosophy pp.31-44.

Mahedi Hasan, Abu Sayed, M. A Hossain, Mosharaf Hossain and Mehedi Hashan Sohel,2017. Determination of Consistency Limits of Different Agricultural Soils, International Invention Journal of Agricultural and Soil Science (ISSN: 2408-7254) Vol. 5(1) pp. 1-7.

Mohajerani, A. A suggested calibration for the cone penetrometer liquid limit. Australian Geomechanics.Vol.34(4).pp.71-76.

Mugenda, O.M. and Mugenda A.G (2003). Research methods: quantitative and qualitative approaches. Nairobi: Acts Press pp, 250

Muhwezi.L., Acai.J.and , Otim .G,2014, An Assessment of the Factors Causing Delays on Building Construction Projects in Uganda published by International Journal of Construction Engineering and Management 2014, 3(1): 13-23.

Muzaale .T and Auriacombe .C. J, 2018. Policy challenge of road infrastructure projects performance trends, issues and concerns in Uganda Volume 10 (3) 18 publish20ed by African journal of public Affairs.p.16

National Budget frame work paper financial year 2019/20-financial year 2023/2024 issued in December 2018 pg xxxi

Nagaraj, T.S. and Srinivasa Murthy, B.R, 1987. Consistency limits of soils- principles and potentials. In Cheremisinoff et al (eds), Civil Engineering Practice **3**, Geotech/Ocean Engng. Lancaster, USA: Technomic Publishing Company.

Olumuyiwa Samson Aderinola and Ajibola Ibrahim Quadri,2017. Clay bearing ratio values in California are correlated with soil index and compaction features, according to a study published in the International Journal of Scientific Research and Innovative Technology. ISSN: 2313-3759, Vol. 4 No. 4; April 2017, pp17-21.

Reaffirmed- Dec 2016.AASHTO: Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Part II Methods of Sampling and Testing. 25th Edition. American Association of State Highway and Transportation Officials, Washington 2005.

Standard Test procedure (STP ,2000) BS 1377-2:1990, Methods of test for soils for civil engineering purposes. Classification tests

Sherwood P.T.,Ryley, M.D. An Investigation for a Cone Penetrometer Method for the Determination of Liquid Limit. Geotechnique.Vol.20(2).pp.203-208.

Soewignjo Agus Nugroho, Muhammad Yusa and Syawal Satibi ,2016: Value Estimation of California Bearing Ratio from Hand Cone Penetrometer Test for Pekanbaru Soils pp,30.

Stone, K.J.L. & Phan, K.D. (1995). Cone penetration tests near the plastic limit. *Géotechnique* **45**, No. 1, 155-158.

Stone K.J.L. and Kyambadde, B.S, 2007. The determination of strength and index Properties of fine grained soils using a soil mini-penetrometer. J.Geotech & GeoEnv.Engineering 133, No. 6, 667-673.

Sultan Taraweh and Mohmd Sarieh,(2013),Causes of cracks and Deterioration of pavement on high ways in Jordan from contractor's perspective volume 3,No.(10) published by civil and environmental research.

Taha. S, El-Badawy.S, Gabr. A,and Azam.A.M, 2015: Modeling of California Bearing Ratio using Basic Engineering Properties. Conference Paper · November 2015. p,6

Terzaghi. K, Mesri, G. & Peck, R.B, 1996. Soil mechanics in engineering practice, 3rd edition. New York: Wiley-Intersicence.

Zumrawi.M. M. E ,2014 Prediction of In-situ CBR of Subgrade Cohesive Soils from Dynamic Cone Penetrometer and Soil Properties, Conference Paper,October 2014. pp 439-442.

Appendix i: Results for CBR Test

Appendix ii: Results for Plastic and Liquid Limit

Appendix iii: Shows Graphs of Force against Depth of Penetration at Liquid and Plastic Limit

Appendix iv: Lab results from RSV materials laboratory