ADAMA SCIENCE AND TECHNOLOGY UNIVERSITY

SCHOOL OF GRADUATE STUDIES

CIVIL ENGINEERING DEPARTMENT

PREDICTION OF SOAKED CALIFORNIA BEARING RATIO (CBR) OF FINE GRAINED SOILS FROM INDEX PROPERTIES;CASE OF SILTY CLAY SOILS FROM ADDIS ABEBA

A thesis submitted to School of Graduate Studies of Adama Science and Technology University in partial fulfillment of the requirements for the Degree of Master of Science in Civil Engineering under Geotechnical Engineering.

By

Winta Amare Bitew

Advisor: Dr. Addis Allem Zeleke

September 26,2016

ADAMA SCIENCE AND TECHNOLOGY UNIVERSITY

SCHOOL OF GRADUATE STUDIES

CIVIL ENGINEERING DEPARTMENT

Master's Thesis

NAME :WINTA AMARE BITEW

SCHOOL: TECHNOLOGY

DEPARTMENT: CIVIL ENGINEERING

MAJOR: GEOTECHNICAL ENGINEERING

ADVISOR**:** Dr. ADDISALLEM ZELEKE

THESIS TITLE: **PREDICTION OF SOAKED CALIFORNIA BEARING RATIO (CBR) OF FINE GRAINED SOILS FROM INDEX PROPERTIES:CASE OF SILTY CLAY SOILS FROM ADDIS ABEBA**

APPROVAL PAGE

This thesis entitled with **"Prediction of soaked California bearing ratio (CBR)**

Of fine grained soils from index properties; case of silty clay soils" has been approved by the following examiners in partial fulfilment of the requirement for the degree master of science in Civil Engineering (Geotechnical Eng.)

Approved by:

Dr. Addis AllemZeleke

AdvisorSignature Date

Internal examiner Signature Date

External examiner Signature Date

Chairperson Signature Date

Department headSignatureDate

School Dean (SCEA) Signature Date

School of post Graduate Studies Signature Date

Acknowledgment

First of all, I would like to thank almighty God.

I wish to express my deep appreciation and gratitude to my advisor Dr. Addis AllemZeleke for his guidance and insightful comments.

I also like to express my deep appreciation and love to my sisters and to my husband BrhaneKumenit for their motivation, patience and help throughout my education time.

I gratefully acknowledge to Addis Ababa Geotechnical and Highway laboratory staffs for their cooperation and for making the work environment amiable.

Finally, I am thankful to the companies (TCD, AACRA and Best Consulting engineers) for their will to give me data.

TABLE OF CONTENTS

LISTS OF TABLES

LISTS OF FIGURES

SYMBOLS AND ABBREVIATIONS

CBR California Bearing Ratio

BS British Standard

ASTM American Society for testing and Materials

AASHTO American Association of State Highway and Transportation Officials.

SPSS Statistical package for the social science

ML Inorganic low plasticity silts

CL Inorganic low plasticity clay

OL Organic low plasticity silts

MH Inorganic high compressible of silts

CH Inorganic high plasticity clay

OH Organic high plasticity silts

LL Liquid limit

PL Plastic limit

OMC Optimum moisture content for compaction test

MDD Maximum dry density

P_{200} percentage of soil pass through No. 200 sieve

R^2 Correlation Coefficient

ANN Artificial Neural Network

MLRA Multiple Linear Regression Analysis

SLRA Simple Linear Regression Analysis

a_1, a_2, a_3 partial regression coefficients

DCP Dynamic cone penetration

N_C,N_q and N_yTerzaghi's bearing capacity factors

ANOVA Analysis of Variance

Dfdegrees of freedom

Abstract

California Bearing Ratio (CBR) is a test which is currently practiced in the design of pavement to assess the stiffness modulus and shear strength of subgrade material so as to determine the thickness of overlying pavement layers. Though various attempts have been made to predict the CBR value by different researchers from samples of their locality, adopting those developed prediction methods without adjustment leads us to misinterpretation of soil behavior. Therefore, this paper is intended to fill this gap and to minimize the time required to conduct the CBR value test by predicting the CBR value from the index properties of fine grained soils of the study area.

In order to achieve this goal two types of data were collected. The first is laboratory test data which is called primary data as a control point. And the second data are collected from different consulting and construction companies, which are called secondary data. The laboratory tests conducted are, grain size analysis, atterberg limits, compaction test and free swelling on 10 soil samples. These samples were collected from Goro , Bole arabsa and koyefiche sites. And about 114 secondary data were collected from Addis Ababa City Road Authority (AACRA), Transport Construction Design (TCD) and Best Consulting Engineers.

In the analysis part, both MS excel spreadsheet and the SPSS software have been used for the scatter plot, correlation, and regression analysis. Using these tools attempts were made to predict CBR from LL, PL, PI OMC and MDD.

The analysis results show that CBR has strong negative relation with LL and relatively strong positive relation with MDD. From the regression analysis it can be observed that the combination of LL, OMC and MDD gives better prediction of CBR with R^2 value of 0.770. This means 77% variation of CBR depends on the variation of these predictors.

The outcome of this thesis can be applicable for preliminary investigations and on construction of small road projects.

1. INTRODUCTION

1.1 General

Due to the fast change of the world the use of transportation becomes vital with high safety and comfort. To attain this requirement, the strength of the soil beneath the structure must be identified. However, obtaining these properties of soils requires relatively more time, effort as well as money. These difficulties lead engineers to find out the quicker and simple way of testing methods by correlating the required strength of the soil with its index property. On the other hand, investigating the index properties of a soil is much easier than investigating other engineering properties. Therefore, by obtaining the index property, the engineering properties can be predicted satisfactorily from empirical correlations.

The support provided by the subgrade is the most important factor in determining pavement design thickness, composition and performance. The subgrade strength is dependent on the condition at the construction and during service; soil type density and moisture content largely determine subgrade strength. The aim of subgrade evaluation is to estimate a value of subgrade support to use in design. California Bearing Ratio isone of the measures of subgrade support. [11]

California Bearing Ratio (CBR) is a test which is currently practiced in the design of pavement to assess the stiffness modulus and shear strength of subgrade material so as to determine the thickness of overlying pavement layers. The California Bearing Ratio (CBR) test was developed by O. J. Porter for the California Highway Department to evaluate the bearing capacity of pavement materials in laboratory conditions during the early 1920s. Since then, several countries have developed or adopted pavement design methods based on the CBR value of the materials. The CBR is the most widely used strength parameter for fine-grained subgrade soils in flexible pavement design [11].

The design of flexible pavements is much dependent on the CBR of subgrade.CBR depends on the type of soil and its property. And its values can be measured directly in the laboratory test in accordance with BS1377:1990, ASTM D1883-73 and AASHTO T193. A laboratory test generally takes four days to measure the soaked CBR value for each soil sample. The result of the tests is actually an indirect measure, which represents comparison of the strength of subgrade material to the strength of standard crushed rock referred in percentage values.

Many attempts have been made to correlate the CBR value of fine grained soils with their index properties. Among those, Harini HN Sureka Naagesh [9]; Ramasubbarao, G.V.and Siva Sankar[16]; Bao Thach Nguyen, Abbas Mohajerani [3]; and Dilip Kumar Talukdar [5]. And for the case of Addis Ababa area Zelalem Werku [20] and Yared Leliso [19] attempts to correlate and give formula of CBR for general soils from their index properties. However, due to the irregular behavior of soils these predictions and formulas may not be valid for the intended study area.

Therefore, this study believes in filling this gap by correlating CBR with index property of the local area.

1.2 Significance of the study

The study will be helpful in the minimization of time consumption and effort for engineers in the field of pavement design and construction. By this way, it will help the engineer to choose the best material for their project and improve the quality of highway and pavement structure.

1.3 Statement of the problem

As the Engineering behavior of soils vary from place to place and even with time, accurate prediction of parameters that properly characterize it depends on how much representative samples in terms of both space and time are gathered. Though various attempts have been made to predict the CBR value by different researchers from samples of their locality, adopting those developed prediction methods without adjustment leads us to misinterpretation of soil behavior. On the other hand, the disadvantage of an empirical method is that it can be applied only to a given set of environmental, material, and loading conditions. If these conditions are changed, the design is no longer valid, and a new method must be developed through trial and error to be conformant to the new conditions. Therefore, this paper is intended to fill this gap and to minimize the time required to conduct the CBR value test by predicting the CBR value from the index properties of fine grained soils of the study area.

1.4 Scope of the study

The study will cover the prediction of soaked CBR value for fine grained soils for pavement design in selected areas in Addis Ababa.CBR test is carried out in the laboratory as per the procedure outlined in AASHTO T193-63 or ASTM D1883-73 and the other index properties according to their respective standards of AASHTO or ASTM. The required correlation is carried out byapplying a single linear regression model and multiple linear regression models with the aid of SPSS Software.

2. OBJECTIVES

2.1 General objective

The main objective of the study is to come up with applicable results of predicting the California bearing ratio from index properties of fine grained soils (especially on silty clay soils) by conducting laboratory tests and collecting data from different organizations.

2.2 Specific objective

1. To establish relevant relationships betweenCalifornia bearing ratio and index properties of fine grained soils, and to develop appropriate empirical correlations among the correspondingsoil parameters.
2. To examine the validity of the correlations, and to draw appropriate conclusions from the relationships of each empirical equation.

3. LITERATURE REVIEW

3.1 Definition of CBR

California Bearing Ratio (CBR)defined as the penetration resistance of a subgrade soil relative to a standard crushed rock. The CBR method of design was studied extensively by the U.S. Corps of Engineers during World War II and became a very popular method after the war. The CBR penetration test is in astandard piston, having an area of 1935 mm^2, is used to penetrate the soil at a standard rate of 1.3 mm per minute. The pressure at each 2.5-mm penetration up to 12 .7 mm is recorded and its ratio to the bearing value of astandard crushed rock is termed as the CBR. The standard values of a high-qualitycrushed rock are as follows:[16].

Table 3.1 Penetration corresponding to standard unit load applied to standard gravel**.** [16]

Penetration	Pressure
2 .5 mm	6 .9 MPa
5 .0 mm	10 .4 MPa
7 .6 mm	13 .1 MPa
10 .2 mm	15 .9 MPa
12.7 mm	17 .9 MPa

In most cases, CBR decreases as the penetration increases, so the ratio at the 2.5mm penetrations is used as the CBR. However, if the CBR value at penetration of 5mm is larger, the test should be redone. If a second test yields also a larger CBR number at 5.0mm penetration, the CBR for 5.0mm should be used.[16]. Using the equation form of;

$$CBR=\frac{\text{test unit load}}{standard\ unit\ load} *100 \qquad (3.1)$$

3.2 Methods of estimating CBR

Basically there are two modes of testing available for the estimation of subgrade CBR. These are field testing and laboratory testing.

3.2.1 Field test of CBR

Field testing is only applicable where it is proposed that subgrade support values are to be extrapolated from an existing pavement and the subgrade soil conditions are similar to those of the proposed pavement. This procedure may be used to determine the design CBR where soils similar to those of the subgrade of the road being designed, have existed under a sealed pavement for at least two years and are at density and moisture conditions similar to those likely to occur in service. [16] A number of field tests may be used to estimate subgrade CBR, e.g. Insitu CBR test and Cone Penetrometer. If field CBR is to be used directly for evaluation or design without consideration for variation due to change in water content, the test should be conducted less than one of the conditions stated in ASTM D 4429-93.

3.2.2 Laboratory test of CBR

Laboratory testing is applicable both where a suitable existing pavement for extrapolation exists, or from first principles. This procedure may be used to determine design CBR or modulus where sufficient samples of the sub grade material for the new pavement can be obtained for detailed laboratory investigations and where a reasonable estimate can be made of likely subgrade density and moisture conditions in service. The method is particularly useful when a close similarity in density, moisture content and materials is not available between the proposed pavement and any existing road. [16]

Laboratory CBR test is carried out as per the procedure outlined in AASHTO T 193-63 or ASTM D 1883-73. This test method provides the determination of the CBR of material at optimum water content or a range of water content from a specified compaction test and a specified dry unit weight. The dry unit weight is usually given as a percentage of maximum dry density from the compaction tests ofeither standard proctor test (ASTM D 698) or modified proctor test (ASTM D 1557). The specimens are made using compaction energy shown below.

Table 3.2 the number of blows and corresponding number of layers for different grade of soils [18]

Method	Blows	Layers	Hammer weight (N)
D698:2(fine grained soil)	56	3	24.5
D698:4(coarse grained soil)	56	3	24.5
D1557:2(fine grained soil)	56	5	24.5
D1557:4(coarse grained soil)	56	5	24.5

3.3 CBR influencing factors

A range of factors influences the CBR of a particular material. Among these soil type, density, moisture content and method of sample preparation plays an important role. Other than material properties the moisture condition of the area is also the factor. The moisture condition at which the material is to be used varies according to the climate of the region.

According to Lawrance and Toole the strength of the hard particles and the plasticity of the fines have higher influence on the CBR value, they also state that the dry density serves as a good indication of the material quality, the general indication being that a higher dry density relates to a better quality material. Kleyn also states the other influential factor for the value of CBR results from its testing procedure. The moisture content of a sample may show significant variance when remolding. This in turn affects the CBR value. In examining the influence of the moisture on the CBR value Porte explains that soaking the material induces swell and moisture absorption, and ultimately reveals a loss in material strength. Hence it can be said that the soaked CBR represents the worst case, as the soaked strength is lower than the strength at the field equilibrium moisture content. [9]

3.4 Application of CBR

CBR is use for the determination of the natural subgrade ground strength and its suitability for pavement design. The CBR values are applicable in deciding the thickness of the pavement. If the CBR value of sugared is high, it means that the sugared is strong and as a result, the design of pavement thickness can be reduced in conjunction with the stronger subgrade. Conversely, if the sugared soil has low CBR value it indicates that the thickness of pavement shall be increased in order to spread the traffic load over a greater area of the weak sugared or alternatively, the sugared soil shall be subjected to treatment or stabilization.

Subgrade materials are typically characterized by (1) their resistance to deformation under load, in other words, their stiffness or (2) their bearing capacity, in other words, their strength. In general, the more resistant to deformation a subgrade is, the more load it can support before reaching a critical deformation value. Application of CBR for different subgrade soils and other materials can be shown below. [7]

Table 3.3: Typical CBR Values for Various Materials

General Soil Type	**USC Soil Type**	**CBR Range**
Coarse-grained soils	GW	40 – 80
	GP	30 – 60
	GM	20 – 60
	GC	20 – 40
	SW	20 – 40
	SP	10 – 40
	SM	10 – 40
	SC	5 – 20
Fine-grained soils	ML	15 or less
	CL LL < 50%	15 or less
	OL	5 or less
	MH	10 or less
	CH LL > 50%	15 or less
	OH	5 or less

3.5 Properties of fine grained soils

The engineering properties of fine-grained soils are controlled by mineralogical factors rather than grain size like that of coarse grained soils. Thin layers of fine-grained soils, even within thick deposits of coarse-grained soils, have been responsible for many geotechnical failures and therefore it is essential to pay special attention to fine-grained soils.

Clays are composed of three main types of minerals kaolinite, illite, and montmorillonite. The clay minerals consist of silica and alumina sheets that are combined to form layers. The bonds between layers play a very importantrole in the mechanical behavior of clays. The bond between the layersin montmorillonite is very weak compared with kaolinite and illite.Water can easily enter between the layersin montmorillonite, causingswelling.

A thin layer of water is bonded to the mineral surfaces of soils andsignificantly influences the physical and mechanical characteristics of fine-grained soils.

Fine-grained soils have much larger surface areas than coarse-grained soils and are responsible for the major physical and mechanical differences between coarse-grained andfine-grained soils. [12]

3.5.1. Swelling properties

Swell Potential of a soil specimen is the ratio of the increase in thickness to the original thickness of a soil specimen compacted at OMC in a consolidation ring, soaked under a surcharge load of 7KPa and is expressed as a percentage. Fine grained soil shows recurrent volume changes with the changes of moisture content, causing problems to the civil engineering structures such as road pavements resting on them. Flexible Pavements constructed on these soil shows signs of damage continuously during the service life of the pavement causes an increase in the maintenance costs. [6]

Generally, properties of fine grained soils can include:

- ✓ Generally low shear strength
- ✓ Plastic and compressible
- ✓ Can lose part of shear strength upon wetting anddisturbance
- ✓ Can shrink upon drying and expand on wetting
- ✓ Generally, very poor material for backfill

- ✓ Generally poor material for embankments
- ✓ Can be practically impervious
- ✓ Clay slopes are prone to landslides
- ✓ Relatively low shear strengths
- ✓ High capillarity and frost susceptibility
- ✓ Relatively low permeability
- ✓ Frost heave susceptibility
- ✓ Difficult to compact

3.6 Classification of fine grained soils

As there is a wide variety of soils covering earth, it is desirable to systemize or classify the soils into broad groups of similar behavior. American Association of State Highway and Transportation Official (AASHTO) classification system is useful for classifying soils for highways. Based on the particle size analysis and the plasticity characteristics AASHTO classifies the soils into coarse grained and fine grained.

Fine grained soils are **silts** and **clays**. Further they can be divided into;

(1) Soilsoflow compressibility(L)if the liquid limit is 50%or less. These are given the symbols ML, CLand OL.

(2) Soils of high compressibility (H)if the liquid limit is more than 50%. These are given the symbols of MH, CH and OH. The exact type of the soil is determined from the plasticity chart and the chart is shown below in the figure 3.1. [1]

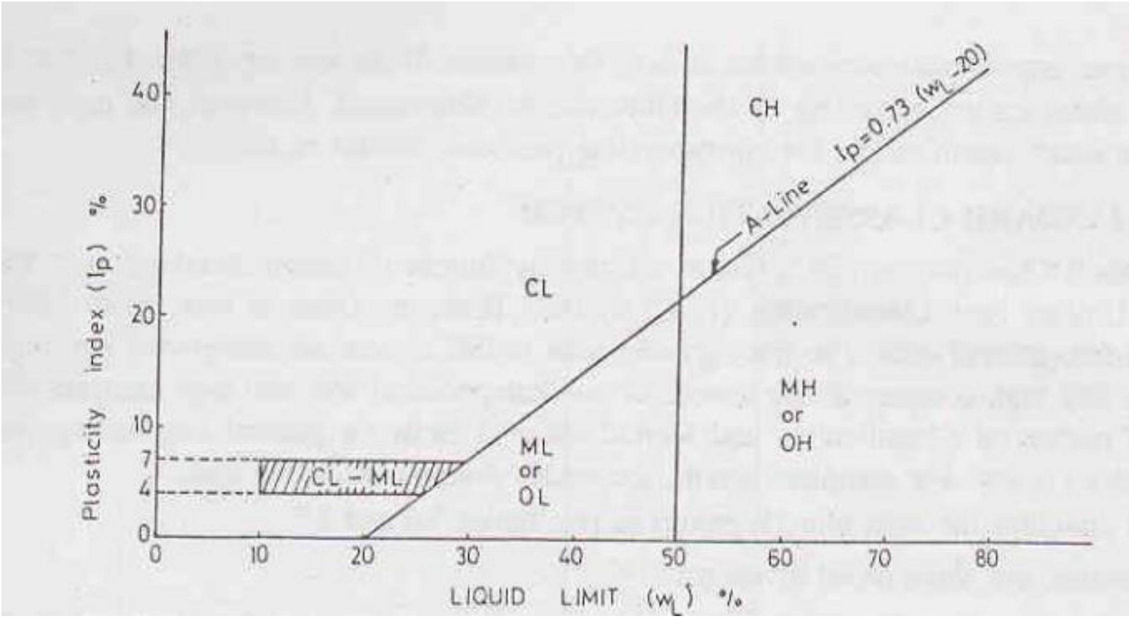


Fig.3.1 plasticity chart (AASHTO system)

Fine-grained soils exhibit considerable changes in physical properties with change of water content. Dry clay may be suitable as a foundation for heavy loads as long as it remains dry, but may turn into swamp when wet. Many of the fine soils shrink on drying and expand on wetting, which may adversely affect structures founded on them. Even when moisture content does not change, the properties of fine grained soils may vary considerably between their natural condition in the ground and their state after being disturbed. [2]

i) **Silts:** have relatively low plasticity compared with clays. In terms of the classification chart they plot below the `A' line. They are unstable in the presence of water and have a tendency to become quick when saturated. When dry, silt can be pulverized easily under finger pressure and will have a smooth feel between the fingers in contrast to the grittiness of fine sand. For similar conditions of previous loading, the higher liquid limit of silt the more compressible it is. [2]

ii) **Clays:** are plastic and virtually impervious. They have low resistance to deformation when wet, but become hard cohesive masses when they dry. They are difficult to compact when wet, and impossible to drain by ordinary means. Large expansion and contraction with changes in water content are characteristics of clays. The higher the liquid limit and thus the plasticity index the more cohesive is the clay.

iii) **Organic soils:** are made of decomposed vegetation and are called peats. Organic soils are dark grey or black in color, and usually have a characteristic odor of decay. Organic clays feel spongy in the plastic range as compared to inorganic clays. [2]

3.7 Silty clay soils

Silty clay soil contains mainly particles of clay, but some silt particles are present. In such types of soils, the principal component of a soil is taken as a noun and the less prominent component as an adjective. The primary soil type with respect to behavior is not necessarily the soil type that constitutes the largest part of the sample. For example, the general character of a mixed soil is determined by clay fraction if it exceeds 30 %. [1]

3.8 Atterberg limits

As it indicates the name atteberg limits comes after the name of Swedish soil scientist Albert Atterberg who originally defined six 'Limits of consistency' to classify fine-grained soils, but in current engineering practice only three of the limits, i.e. liquid, plastic and shrinkage limits are used. He also defined the plasticity index, which is the range of water content where the soil is plastic, and he was the first to suggest it for soil classification. Later in the late 1920's Terzaghi and Casagrande working for the U.S. Bureau of public Roads, standardized the Atterberg limits so that they could be readily used for soil classification purposes. In present geotechnical engineering practice one usually uses the liquid limit (LL), the plastic limit (PL), and the shrinkage limit (SL). [2]

3.8.1 Liquid limit

The liquid limit is the water content at which the soil changes from the liquid state to the plastic state. At this limit the clay is practically like a liquid, but possesses a small shearing strength. The liquid limit of soil depends upon the clay mineral present. The stronger the surface charge and the thinner the particle the greater will be the amount of adsorbed water and therefore, the higher will be the liquid limit.

Liquid limits can be determined by two methods: casagrande'sapparatus and cone penetration method.

Liquid limit is the water content at which the soil is sufficiently fluid to flow when the device is given 25 blows. But since it is difficult to get exactly 25 blows the test is conducted at different water contents so as to get blows in the range of 10 to 40.A plot is made between the water content as ordinate and the number of blows on log scale as abscissa.

3.8.2 Plastic Limit

Plastic limit is the water content below which the soil stops behaving as a plastic material. At this stage the soil begins to crumble when rolled into a thread of soil of 3mm diameter, it loses its plasticity and passes to semi-solid state. About 10 gm of the plastic soil is taken in one hand and a ball is formed. The ball is rolled with fingers on a glass plate to forma soil thread of uniform diameter. And this procedure repeated till the thread crumbles. [1]

3.8.3 Plasticity index

Plasticity index is the range of water content over which the soil remains in the plastic state. [1]

Plasticity is represented by plasticity index PI which is numerically equal to the difference between the liquid limit and the plastic limit water contents of the soil. Plasticity index is used in the classification of fine-grained soils, through the plasticity chart. The plasticity chart is widely used to differentiate between clays and Silts and further, to subgroup them according to the degree of their compressibility. The plasticity index is used in a good number of correlations with many engineering properties such as the compression index, the coefficient of consolidation, swelling potential, the friction, the coefficient of earth pressure at rest, the un drained shear strength. [2]

3.9 Compaction

3.9.1 General

In the construction of engineering structures such as highway embankments or earth dams,loose fills are required to be compacted to increase the soil density and improve their strength characteristics as well as for the durability and stability of structures like foundation, roads and airfield.

Compaction is the densification of soil by application of mechanical energy. It is the process of rearranging the soil grains more closely. By this process the air void in the soils reduced and the density of the soil increased. The degree of compaction of a soil is measured in terms of dry unit weight that means it is the amount of soil solids that can be packed in a unit volume of the soil. [7]

.**3.9.2 Purpose of compaction**

Compaction is the most common soil improvement technique.Generally, it is used as practical means of achieving the following characteristics of soils. [7]

- ✓ To increase shear strength
- ✓ To improve stability
- ✓ To improve bearing capacity
- ✓ To reduce the compressibility and permeability
- ✓ To prevent detrimental settlement
- ✓ To control un desirable volume changes

3.9.3 Compaction methods

There are two methods of compaction. Laboratory and field tests

3.9.3.1 Laboratory compaction test

Laboratory compaction tests are usually utilized to specify the compacted dry unit weight to be attained in the field. Since the control in the field cannot be strict as that of in laboratory, the specifications usually required attainment of 90 to 95 percent of dry unit weight attained in the laboratory.

A. Standard Proctor Compaction Test

Proctor developed this test in connection with the construction of earth fill dams in California. A soil at aselected water content is placed in layers in to a mold of given dimensions, with each layer compacted by 25 or 56 blows of a 2.5kg hammer dropped from a height of 305mm, subjecting the soil to a total compaction effort of about 600kNm/m3.The procedure is repeated for a sufficient number ofwatercontentstoestablisharelationshipbetweenthedryunitweightandthewatercontentofthesoil.

The plot of this data representsacurvilinearrelationshipknownasthecompactioncurveor moisture densitycurve.

Thenthevaluesofwatercontentandstandardmaximumdryunitweightaredeterminedfromthecompactioncurve as shown on figure 3.2 below.

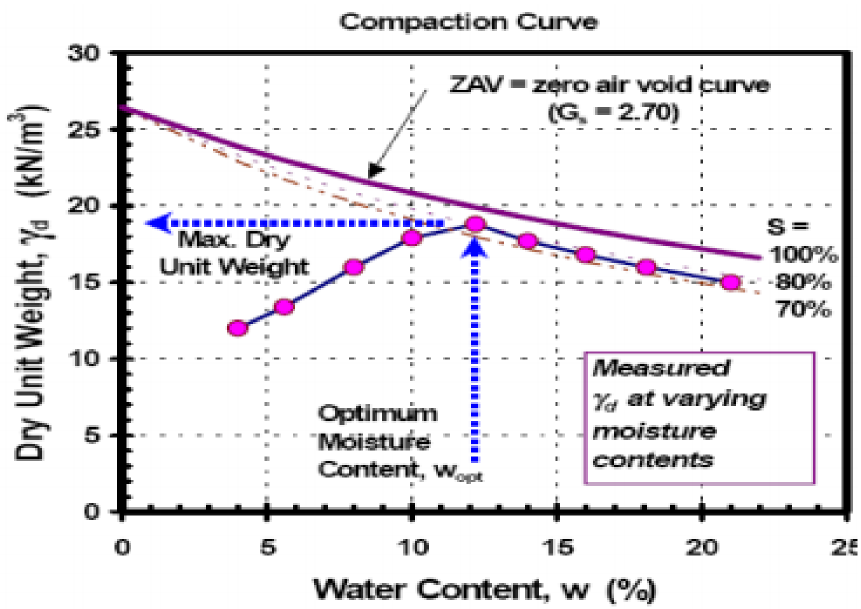


Fig.3.2 Representative moisture content-dry density relation curve**. [15]**

B. ModifiedProctorCompactionTest

Thistestmethodcoverslaboratorycompactionproceduresusedtodeterminetherelationshipbetweenwatercontentanddryunitweightofsoils compactedina4in.or6in.diametermoldwitha 5kg hammerdroppedfromaheightof 457mm producingacompactioneffortof2,700kNm/m3.A soil at a selected water content is placed in five layers in to a mold of given dimensions, withEach layer compacted by 25 or 56 blows of a 4.54kg hammer dropped from a height of 457mm subjecting the soil to a total compactioneffort ofabout 2700kN-m/m3. when this data plotted it representsacurvilinearrelationshipknownasthecompactioncurveormoisture-dryunitweightcurve. Then the values oftheoptimumwatercontentandmaximumdryunitweightaredeterminedfromthecompactioncurve. [13]

3.9.3.2 Field compaction tests

The compaction of soil in the field must obtain the desired unit weight at the optimum moisture content. Therefore, the field engineer must do periodic checks to see whether the Compaction is giving desired results. [13]. The process of checking involves:

1. Measurement of the dry unit weight, and
2. Measurement of the moisture content.

For compaction of soils in the field several methods are used. The choice of these methods depends up on the soil type, the maximum dry density required and economic considerations. The four major types of compaction processes currently in useby modern construction equipment are:

a) Impact

b) Manipulation

c) Pressure

d) Vibration

The necessary compaction of subgrades ofroads,earthfills,and embankments may be obtained by mechanical means. The equipment thatarenormallyusedforcompactionconsistsof:

1. Smooth wheelrollers
2. Rubber tiredrollers
3. Sheep’sfootrollers
4. Vibratory rollers

In cohesive soils, densities of the order of 95 percent of standard Proctor can be obtained withpractically any of the rollers and tampers;however,vibrators are not effective in cohesivesoils. Where high densities are required in cohesivesoils in the order of 95percent of modified Proctor, rubber tired rollers with tireloadsintheorderof100kNandtirepressureintheorderof600kN/m^2 are effective. [13]

3.10 Previously Established Correlation Equations of CBR

As CBR test is time consuming test, a good indication of CBR values from index tests arevery beneficial. Due to this attempts have been made to relate index test parameters to CBR values. Many of these attempts were made in other countries where the soil type and climatic regions are very different.

3.10.1 Correlation of CBR with DCP

The dynamic cone penetrometer (DCP) has been widely used for estimating the strength of soils. Also, the California bearing ratio (CBR) test is the test most widely used in highway pavement design all over the world. Attempts have been made tofind a relationship between DCP and CBR values for fine grained soils by different researchers.

Among these, Harison tries to correlate CBR of granular and cohesive soils with DCP by the form of:[15]

$$Log(CBR)=2.55-1.14log(DCPI) \quad (3.2)$$

Webter et al by 1992 correlate CBR various soils with DCP and gives the following formula:

$$Log(CBR)=2.46-1.12log(DCPI) \quad (3.3)$$

3.10.2 Correlation of CBR with Shear strength.

These are correlations which are tried to correlate CBR values with bearing capacity of soils.

Terzaghi‘s bearing capacity equation for circular foundation is:

$$q_u=1.3cNc + p_o + Nq + 0.3\gamma BN_\gamma \quad (3.4)$$

C=is the cohesion of the soil

γ=the bulk density of the soil

p_o=the overburden pressure at the base of the plunger

B=the diameter of the plunger

And Nc,Nq,and N_γ are terzaghi's bearing capacity factors.

Since angle of shearing resistance for saturated clay in undrained condition, is zero the equation reduced to the form:

$$q_u = 6.68c \quad (3.5)$$

This agrees with experience that the number of surcharge weights used affect the CBR value of sands, for which Nq is much greater but not for clays. Using SI units, the CBR value is 100% for plunger pressure of 6900KN/m2at penetration of 2.5mm, giving

$$CBR= q_u *100/6900 =0.097c \quad (3.6)$$

3.10.3 Correlation of CBR with index properties of fine grained soils

Many attempts have been made to obtain the correlation equations soils in obtaining CRB values of fine grained soils in relating their locality. Among these Zelalem Worku[20] tried to predict CBR value from granular and fine grained soils separately. For Addis Ababa fine grained soils, he gets 0.564 maximumvalue coefficient of determination. The properties considered were LL, PL, PI, OMC, Percent passing 0.075mm sieve no, MDD.The correlation was not strong as granular.

With the formula of:

$$CBR=41.175—0.029* LL—0.009 * P200 \text{ with} R^2=0.564 \quad \ldots\ldots (3.7)$$

According to Harini and Naagesh[9], CBR value is predicted using ANN and MLR and gives 0.94 and 0.86 coefficient of correlationrespectively.With the formula of:

CBR= 4.86- (0.07WL) + (0.01percentfines) with R^2=0.86 ……………………………… (3.8)

Nguyen and Mohajerani [15] correlates CBR from the experimental results, withmoisture content (MC), plasticity index (PI) and maximum dry density (MDD) was found to be strong for the samples tested at OMC, wet side of OMC and soaked conditions.

Log (CBR) = 4.767 + 0.843(MC) + 0.020(PI) - 1.522(MDD) with =R^2= 0.75 …………….(3.9)

Talukdar [5] a multiple linear regression model developed by using Linex function of Microsoft Excel software for the soil type of sandy silt (ML) and correlates CBR as follow**.**[4]

CBR (soaked) = 0.127(LL) + 0.00 (PL) – 0.1598(PI)+1.405(MDD) -0.259(OMC) + 4.618..(3.10)

Yared Leliso [19] also tried prediction of CBR value from index properties and develops different models.Among these, the model with strong coefficient of determination is :

CBR = -21.522 - 0.141*LL + 0.137*PI + 20.244*MDD, with R^2 = 0.629 (3.11)

The importance of this study is to give more emphasis on fine grained soils which are not much focused by other studies in Ethiopia. This means the study will play an important role in filling the gap of the previous studies. Most of the fine grained soils used in the previous studies have more than 50% liquid limit which are most probably excavated from the proposed site due to their weak bearing capacity. But this study focuses on fine grained soils having liquid limit of less than 50% with better bearing strength.

4. FIELD INVESTIGATION AND SOIL SAMPLING

4.1 Study area and sampling of soils

For the thesis work two types of data are involved. These are primary and secondary data.

The primary data was obtained from different areas in Addis Ababa. According to previous studies Addis Ababa is dominantly covered by red fine grained soils in northern and western part and with black expansive soils in the eastern and southern parts.

Silty clay soils are mostly found in the eastern and southern part of Addis Ababa. Because of this three sites were selected and ten disturbed soil samples collected from Goro, Koyefiche(kality) and Bole Arabsa sites. These sites are selected as they are representative places for the case of silty clay soils and it is believed the result of the study will play big role in expansion of Addis Ababa. Location of sampling areas are shown in figure 4.1 below.

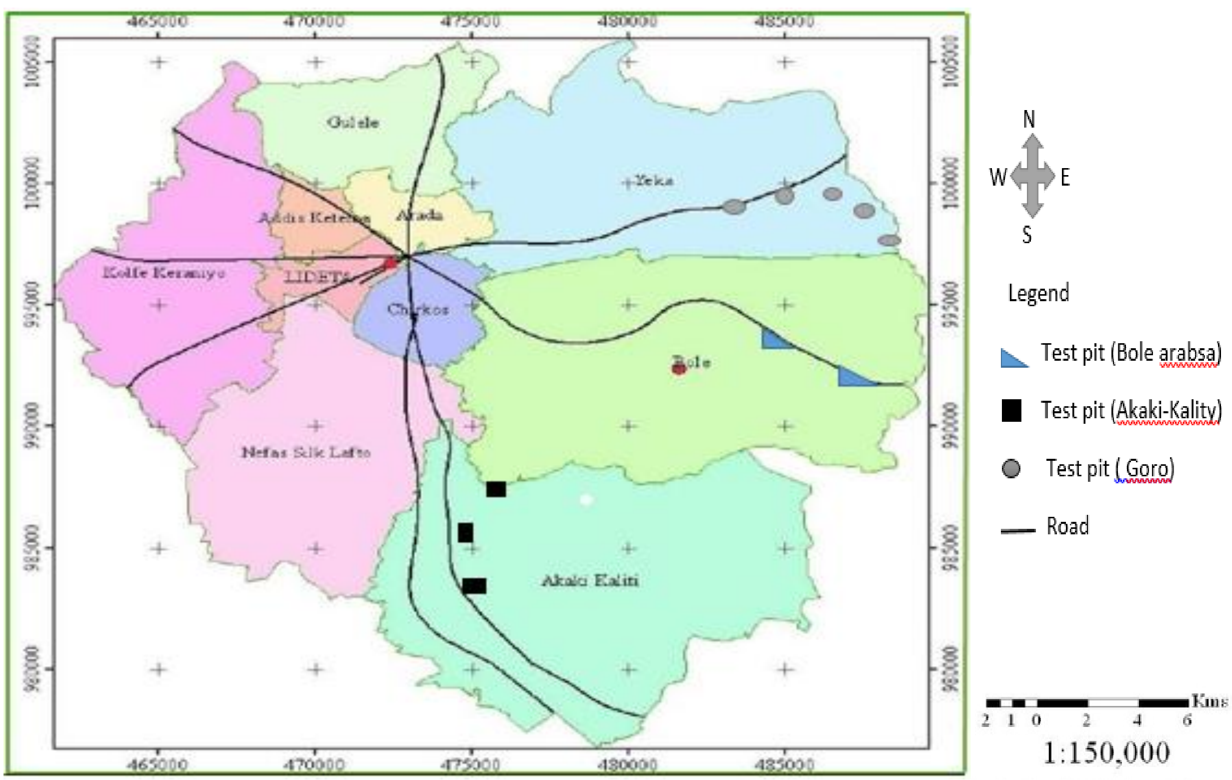


Fig 4.1. Location of sampling areas

The samples were taken from a depth ranging from 2.5m to 3.5m below the ground surface. These are five test pits from Goro site,three test pits from koyefiche and two test pits from Bole arabsa site.

4.2 Visual identification of soils in the field

The identification of soils on the field was made based onprocedures described in ASTM D 2488-00 (Standard practices for Description and identification of soils) standards. The first step used in identifying the type of soil was to determine whether it is fine or coarse grained soil based on the size,texture and distribution of soil particles. After the fine grained soils are identified, the simple and important test methods are used for detail identification of the soil types.These are:

Dry strength test: is a measure of the character and quality of colloidal fraction of soil. If the sample, which is 1/8-inch ball size mold, cannot be broken the soil is highly plastic and samples of low plasticity will break easily.

Dilatancy test: is observing rapidity of water rising to the surface when sample is shaken and when the sample is squeezed between fingers. Speed of appearance during shaking and disappearance assists in identifying the fines in the sample.

Plasticity (toughness) test:is the method of testing the plasticity of the soil. When water is added to the fines portion of the soil sample and the sample is then rolled into 3mm diameter thread.

Fig.4.2 Field investigation and soil sampling

4.3 Sampling methods and sample preservation.

Soil sampling is made from three projects by taking about 50 kg of disturbed soils. For the koyefiche site since the subgrade was covered by selected material, 1m by 1m test pit is excavated using hand tools.

To keep the natural moisture content as well as to avoid the exposition of soils from foreign materials like excessive heat and water, the bags which is used to hold the soil samples made to be water proof like plastic type.

For identification purpose the name of the sampling area, depth of sampling and color of the sample were written on the outside part of the bag.

The transportation of the soil samples was by vehicle to Geotechnical laboratory of Addis Ababauniversity (AAiT). Since it is not such a long distance, the soil sample is not expose to any obstacle.

4.4 Sample size and sample preparation

For correlation purpose the required number of sample size is large. but due to shortage of permeation time for using the laboratory as well as to get accurate results the soil sample size is decided to be ten.

Preparation of soil samples for the required tests starts by air drying. Before go to the next step the air dried soil samples were thoroughly pulverized by fingers and wood hammer.

For particle size distribution test representative 500gram air dried (passing no.4) and 50gram oven dried (passing no.200) sample are prepared for wet sieve and hydrometer analysis respectively according to ASTM D1140-97 (Standard Test Method for Amount of Material in the Soils Finer than the No. 200 (0.075mm) Sieve) procedure.

For atterberg limit tests about 200 gram airs dried (passing no.40) sample taken and soaked for hours by small amount of water which is used to moist the soil grains. In order to get precise results multipoint method is chosen and prepared to test according to ASTM D 4318-00 (Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils) procedures.

For compaction test, method A of compaction is chosen because less than 20 % by mass of material is retained on the 4.75mm (no.4) sieve. About 16kg soil sample is prepared for five trails of each soil sample. The test is then proceeding according to ASTM 698-00 (Standard Test Methods for Laboratory Compaction Characteristics of Soil) procedures.

For California bearing ratio about 6 kg airs dried passing 4.75mm (no.4) sieve soil sample is prepared from each sample in the Highway material laboratory. The test is proceed then according to ASTM D 1883-99 (Standard Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils) procedures or AASHTO T 193-63.

4.5. Data Collection

In this study, besides the 10 primary data 114 secondary data were collected from different organizations namely: Addis Ababa City Road Authority (AACRA), Transport construction design (TCD) and Best consulting engineers (Best). Totally 124 laboratory data were collected and analysis is made to obtain the relationship between index properties and California Bearing Ratio.

The primary data include the laboratory tests such as: Atterberg limits, compaction characteristics, specific gravity, grain size, free swell and CBR. All tests used in this study are performed on fine-grained soils that encompass, Black, White, and Grey soils.

The accuracy of the primary data depends on the precision of the laboratory work. For the analysis part, to bring the primary and secondary data in similar platform the technique used is omitting the lab results which are exceptional like for higher liquid limit higher CBR values and the like from the secondary data.

5. LABORATORY TESTING OF SOILS

5.1 Index property tests

The numerical results which are obtained from tests which carried out in order to classify a soil are called index properties. [15] Therefore index properties are tests which are used to determine and characterized the basic properties of soils. These properties are particle size distribution, specific gravity,moisture content, atterberg limits and moisture-density relationships. So this chapter presents the determination and analysis of index properties and CBR tests with careful follow up of procedures.

5.1.1 Particle size distribution

It is a process of separation of soils in to different fractions based on the particle size. It has two methods sieve analysis for coarse grained soils and sedimentation (and/or wet analysis) for fine grained soils. Since most of soils are found mixed a combined analysis can be used. For the case of this study hydrometer and wet sieve analysis are used. A representative 500gram air dried soil is soaked for 24 hours, washed on No.200 (0.075mm) sieve and record the results. As well as for hydrometer analysis a representative 50gram oven dried soil is soaked for 24 hours and then the hydrometer reading is recorded for 24hours.after all the necessary data is recorded and analyzed, the gradation curve is drowning which represents the distribution of particles of different sizes in the soil mass. The curve plots as the percentage of fines as ordinate (natural scale) and the particle size as abscissa(log scale) as shown below. The test is performed according toASTM D1140-97 procedureor AASHTO T 88.

Table 5.1 Typical steps for plotting particle size distribution.

Sample Location	Goro site, at 3m depth
Sample description	Black, silty clay
Test procedure standards	ASTM D1140-97
Date tested	Sep.25,2015

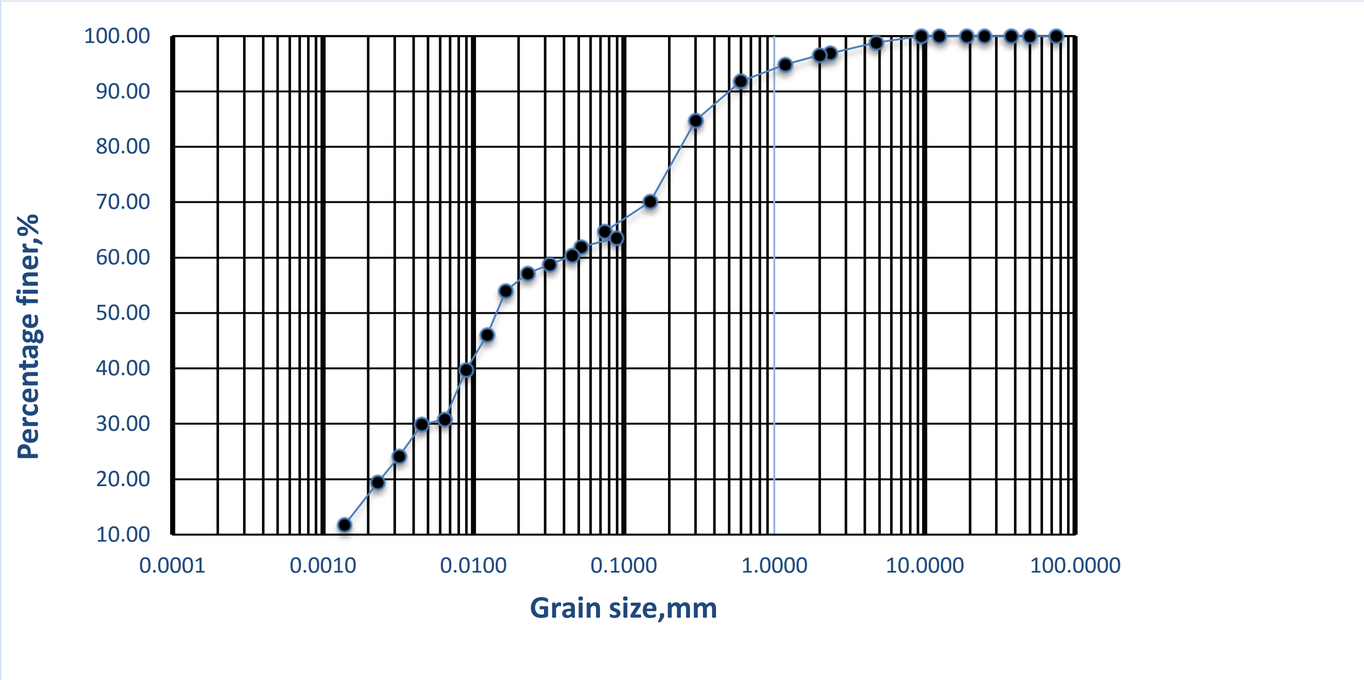


Fig.5.1 particle size analysis

5.1.2 Specific gravity

Specific gravity of a solid particle is defined as the ratio of the mass of a given volume of solids to the mass of an equal volume of water at 4^0c. [1] specific gravity uses to determine particle size distribution.

The test is done by using 250ml and 500ml pycnometer for two trails for each soil sample.About 25gram of oven dried soil specimen is entered in to the pycnometer using funnel. Then water is added until the water level is ½ of the depth of the main body of the pycnometer. It is well mixed until slurry is formed. Using vacuum entrapped air is removed. Water is added to the air free slurry carefully from reentering of air. Then the slurry with the pycnometer is weighted and temperature is measured. The slurry is then removed; cleaned and dry the pycnometer. The pycnometer is again filled with clean water up to the mark of the pycnometer. Similar to the slurry the water with pycnometer is weighted and temperature is measured. Finally, computation is done as shown below. The test is performed according to ASTM D 854-02 procedure.

Table 5.2. Typical steps for computation of specific gravity.

Sample Location	Bole arabsa,at 3m depth
Sample description	White silty clay
Test procedure standards	ASTM D 854-02
Date tested	Sep.24,2015

	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	178.2	178.5
Temperature, T_x(°c)	23	23.1
Weight of pycnometer + water at T_x , W_{pw}(atT_x)	162.50	163.00
Weight of dry soil,w_s (gm)	25	25
Conversion factor, K	0.9993	0.9993
Specific gravity of soil at 20°c.	2.69	2.63
Average specific gravity of soil.	**2.66**	

5.1.3 Atterberg limit

These test methods cover the determination of the liquid limit, plastic limit, and the plasticity index of soils. Taking 200gram of 0.425mm sieve passing air dried soil considerable water is added to moist the soil grains and placed for hours. From this about 20 gram of the mixed soil is set aside for determination of plastic limit. Water contents are determined by separating the blows number in to four parts from 15 up to 35.the soil mix at each blow number are taken and oven dried for 24 hours from which water contents are calculated. The plot is drawn as the moisture content as ordinate (natural scale) and number of blows as abscissa (log scale). From the plot, the liquid limit was determined by locating the watercontent at 25 blows.

For plastic limit from the 20gram mixed soil two ball like are rolled in 3mm tread on glass plate. The thread is rolled until the soil crumbled when it reached 3mm diameter. At this stage the soil is taken and oven dried for 24 hours and water content is measured. Then the average value of the water contents from two trails taken as plastic limit. The tests are done according to ASTM D 4318-00 procedures or AASHTO T 90.

The plastic index is obtained by subtracting the plastic limit from liquid limit.

Table 5.3 Typical steps for Atterberg limit calculations

Sample Location	Koyefiche site at 3.5m depth
Sample description	Gray silty clay
Test procedure standards	ASTM D 4318-00
Date tested	SEP.29,2015

	Liquid Limit					Plastic Limit	
Trial No	1	2	3	4		1	2
Container No	WT4	WI	T-7	T-6		AO	G7
Mass of container, g	15.8	15.9	15.4	15.7		15.9	15
Mass of container + Wet soil, g	43.20	37.80	45.60	46.90		20.50	23.60
Mass of container + Dry soil, g	35.50	31.30	36.10	36.90		19.70	21.80
Mass of water, g	7.70	6.50	9.50	10.00		0.80	1.80
Mass of dry soil, g	19.70	15.40	20.70	21.20		3.80	6.80
Water content, %	39.09	42.21	45.89	47.17		21.05	26.47
No of blows	33	26	21	16		------	------

Liquid Limit, % =	43	Plastic Limit, % =	24	PI, % =	19

Fig.5.2 Atteberg limit analysis.

5.2 Free swell

Test procedure for free swell was suggested by Holtz and Gibbs to measure the expansive potential of cohesive soils. The test is not standardized by ASTM. The procedure consists of pouring very slowly of 10 cubic centimeters of that part of the dry soil passing No. 40 sieve in to a 100 cubic centimeters graduated measuring cylinder and letting the content stand for approximately twenty-four hours until all the soil completely settles on the bottom of the graduating cylinder. Then the final volume of the soil is noted. [8]

Fig.5.3 Laboratory testing of free swelling

5.3 Compaction

The standard laboratory compaction method is used to determine the relationship between water content and dry unit weight of soils (compaction curve).

A soil at selected water content was placed in three layers into a mold of given dimensions, with each layer compacted by 25 blows of 24.4kN rammer dropped from a distance of 305mm, subjecting the soil to a total compaction effort of about 600 kN/m^2. The resulting dry unit weight is determined. The procedure was repeated for sufficient number of water contents to establish a relationship between the dry unit weight and the water content. The values of optimum water content and maximum dry unit weight were determined from the compaction curve. The test is conducted according to ASTM 698-00 or AASHTO T 99. The procedure of computation is illustrated in Table 5.5 as follows.

Table 5.4 Typical steps for calculations of OMC and MDD

Sample Location	Koyefiche site at 3.5m depth
Sample description	Gray, silty clay
Test procedure standards	ASTM 698-00
Date tested	Oct.5,2015

Trial No.	1	2	3	4	5
Mass of Mold, g	3137.4	3137.4	3137.4	3137.4	3137.4
Mass of mold + Compacted Soil, g	4511	4670.4	4780.6	4810.7	4740
Mass of Compacted soil, g	1373.6	1533	1643.2	1673.3	1602.6
Volume of Mold, cm3	944	944	944	944	944
Bulk density, g/cm3	1.46	1.62	1.74	1.77	1.70
Water Content, %	17.37	20.83	24.23	28.71	31.76
Water Content					
Container No	L-1	T-3	WI	W-P	3-F
Mass of container, g	15	15.2	15.8	15.8	15.8
Mass of container + wet soil, g	40	50	39.9	41.8	54.8
Mass of container + Dry soil, g	36.3	44	35.2	36	45.4

Mass of Water, g	3.7	6	4.7	5.8	9.4
Mass of Dry soil, g	21.3	28.8	19.4	20.2	29.6
Water content, %	17.37	20.83	24.23	28.71	31.76
Dry Unit Weight, g/cm3	1.24	1.34	1.40	1.38	1.29

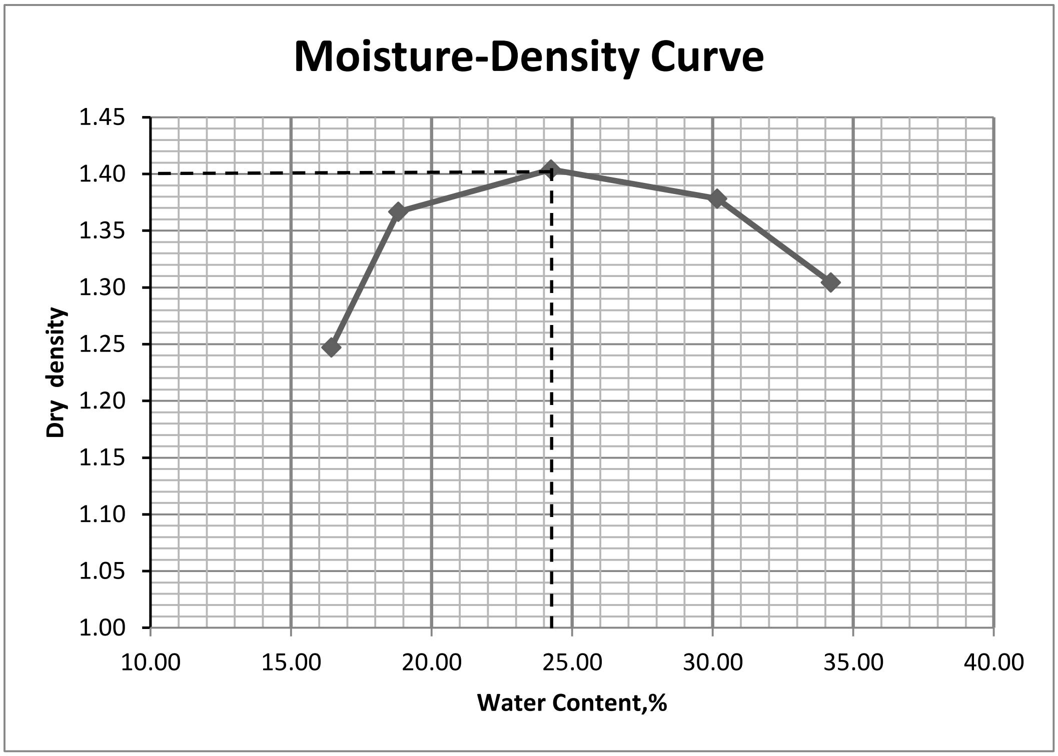


Fig.5.4compaction curve, for determination of MDD and OMC

5.4 California Bearing Ratio

As previously discussed CBR is use for the determination of the natural subgrade ground strength and its suitability for pavement design.

This test used for determination of the CBR of a material at optimum water content from compaction test.

Taking about 6kg of 4.75mm sieve passing air dried soil, mixed it using the optimum water content which is found from compaction test. The soil is compacted by separating into 5 layers using 4.54kg rammerand it compacts 56 blows for each layer. The spacer disk is removed from the mold weight and moisture contents measurement are then taken. The mold is soaked in the water full container for 4 days by recording the initial swelling. After four days the final swelling is recorded and the mold is out from the water and put on the table of loading machine. The loading machine penetrates the specimen at a rate of 1.27mm/min and readings are recorded from initial up to 7.62mm. After all the necessary data is recorded and analyzed the curve is drown.

Fig.5.5 photo of soaked soil mold for CBR test

The CBR is then determined by reading from the curve the load that causes a penetration of 2.54 mm and dividing this value by the standard load (6.9 MPa) required producing the same penetration in the standard crushed stone. Similarly, the CBR at 5.08 mm penetration is obtained by dividing the load causing a penetration of 5.08mm with the load of 10.3MPa required to produce the same penetration in standard crushed stone. The two values are then compared and if the 2.54 mm value is greater than the 5.08mm value, it is the CBR of the material and used for design purposes. If on the other hand the 5.08 mm value is larger, the test is entirely repeated on a fresh specimen. If the new percentage value at 5.08 mm penetration is still greater, then this is taken as the CBR value. The analysis table and load-penetration curve is conducted according to AASHTO T 193-63 orASTM D 1883-99 as illustrate below.

Table 5.5 typical steps for calculation CBR value.

Sample Location	Bole arabsa site at 3mm
Sample description	White, silty clay
Test procedure standards	ASTM D 1883-99
Date tested	Oct,15 2015

Penet.	Ring Reading	load	Stress	Standard stress	CBR
(mm)	(Div.)	N	N/mm2	(N/mm^2)	%
0.00	0.0	0	0		
0.64	22.0	565.554	0.292276		
1.27	35.0	899.745	0.464984		
1.91	45.9	1179.951	0.609794		
2.54	58.9	1514.142	0.782502	6.9	11.34062
5.08	96.5	2480.726	1.282029	10.3	12.44688
6.35	117.0	3007.719	1.554377		
7.62	132.0	3393.324	1.753656		

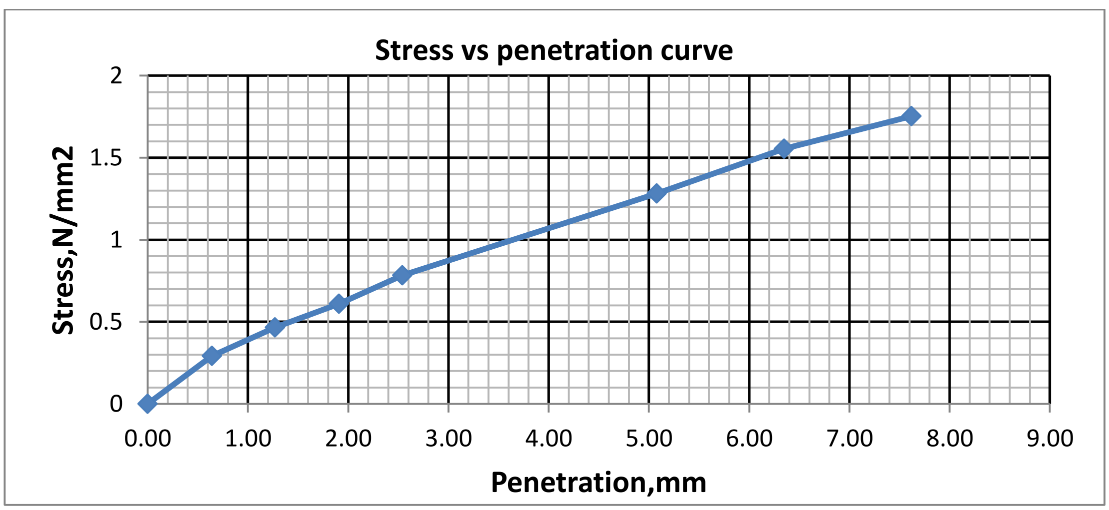


Fig.5.6 Illustration of stress vs penetration curve for determination of CBR.

Table 5.6 summary of primary test results

Sample code	percent passing, %					atterberg limit			AASHTO Classification	compaction		CBR %	Free swell, %
	Gravel	Sand	Silt	Clay	FC	LL	PL	PI		OMC, %	MDD		
SS1	1.2	11.1	43.2	44.5	87.7	49	32	17	A-7-5	25	1.35	8	28
SS2	0.4	11	32.6	56	88.6	48	30	18	A-7-5	24.5	1.41	9	35
SS3	2.2	22	31.8	44	75.8	42	32	10	A-7-5	20.5	1.42	14	25
SS4	1.3	11.4	38.3	49	87.3	47	33	14	A-7-5	24	1.4	11	30
SS5	0.8	17.8	26.4	55	81.4	46	28	18	A-7-6	22.5	1.45	12	38
SS6	1.2	34.1	24.7	40	64.7	41	29	12	A-7-5	19	1.44	18	25
SS7	0.7	17.5	36.8	45	81.8	49	35	14	A-7-5	27.5	1.38	10	30
SS8	1.1	33.6	19.3	46	65.3	45	30	15	A-7-5	33	1.44	13	30
SS9	4.9	26.6	23.5	45	68.5	44	29	15	A-7-6	21.5	1.45	17	25
SS10	5.2	27	29	38.8	67.8	43	24	19	A-7-6	24.5	1.43	16	25

6. DATA ANALYSIS, RESULTS AND DISCUSSIONS

6.1 Introduction

The purpose of correlation analysis is to measure and interpret the strength of a linear or nonlinear relationship between two continuous variables.Before obtaining the relations between two or more variables, valid data must be collected that may be obtaining from experiment (primary data) or may be collected from known sources (secondary data). In this thesis both primary data (10 samples) and secondary data (114 samples) are collected. In this Chapter analysis have been done to develop possible relationships among the parameters.

To obtain a relation between two or more variables many methods can be used. But for this thesis two common methods are used. Namely, Scatter plot and Regression Analysis. Detail discussion and analysis of each method is presented below.

6.2 Scatter plot and Best-Fit curve

Scatter plots of the variables are done by using MS excel spreadsheet. This software is chosen because it is found to be the most powerful and manageable tool for scatter plot analysis and determination of correlation between two variables.

According to aim of the thesis the scatter plots are done to express the degree of correlation strength of the dependent variable CBR by each independent variables of LL, PL, PI,OMC and MDDfor both primary and all collected data. The figures of scatter plot are shown below.

6.2.1 Scatter plot for primary data

The scatter plots of the primary data are done by collecting the results of the laboratory controlled tests of 10 soil samples. These are presented on fig 6.1-6.6 as shown below.

A. Relation between LL and CBR

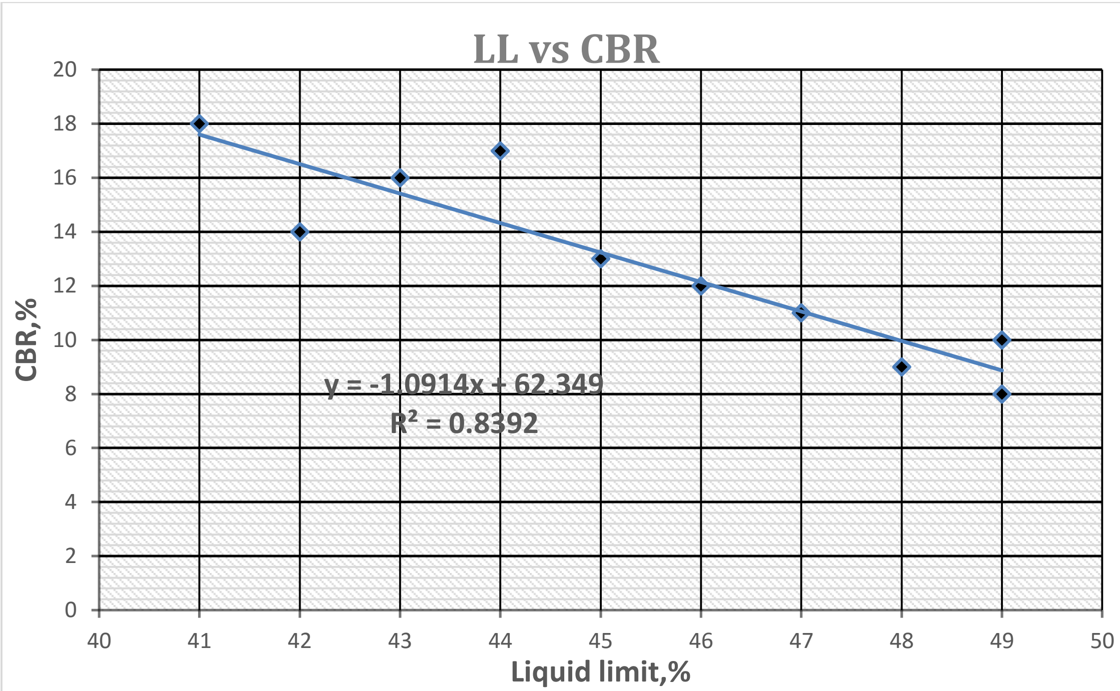


Fig 6.1 Scatter plot and best-fit curve of liquid limit and CBR

B. Relation between PL and CBR

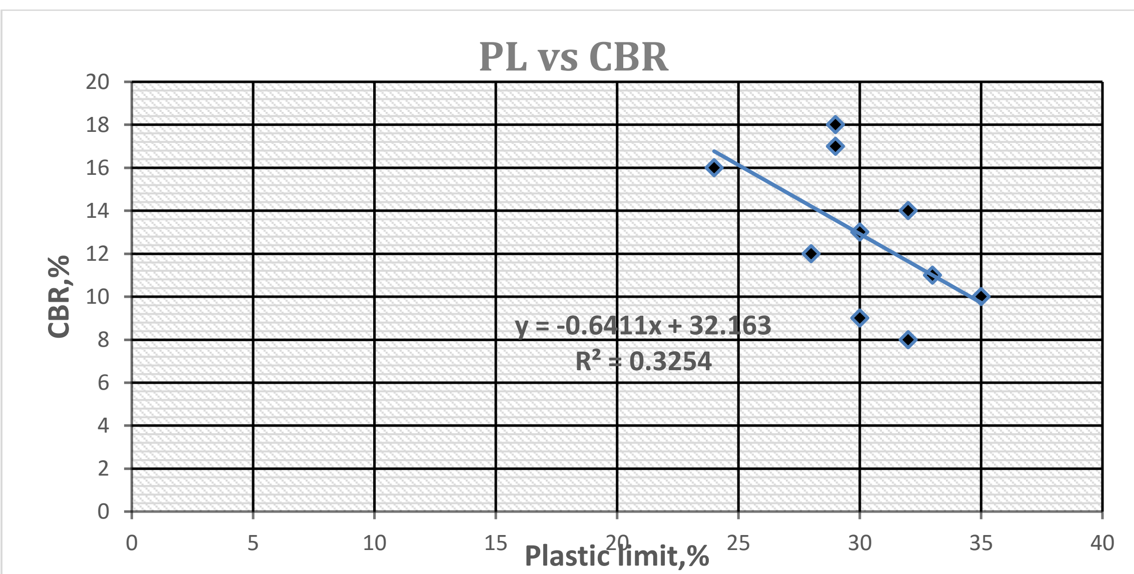


Fig 6.2 Scatter plot and best-fit curve of Plastic limit and CBR

C. Relation between PI and CBR

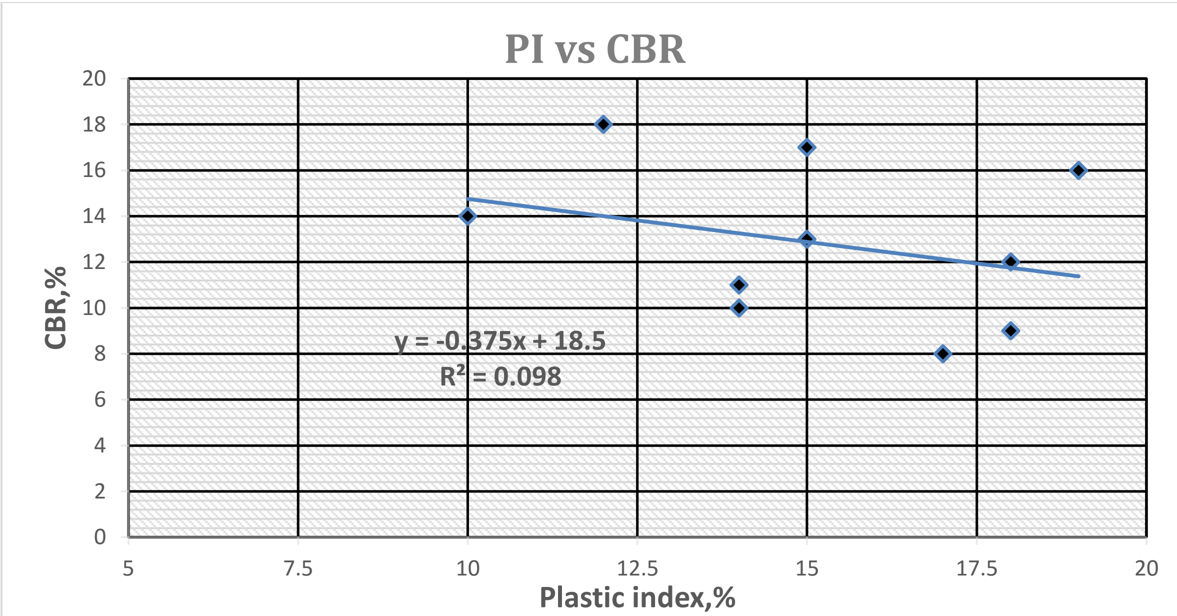


Fig 6.3 Scatter plot and best-fit curve of Plastic index and CBR

D. Relation between OMC and CBR

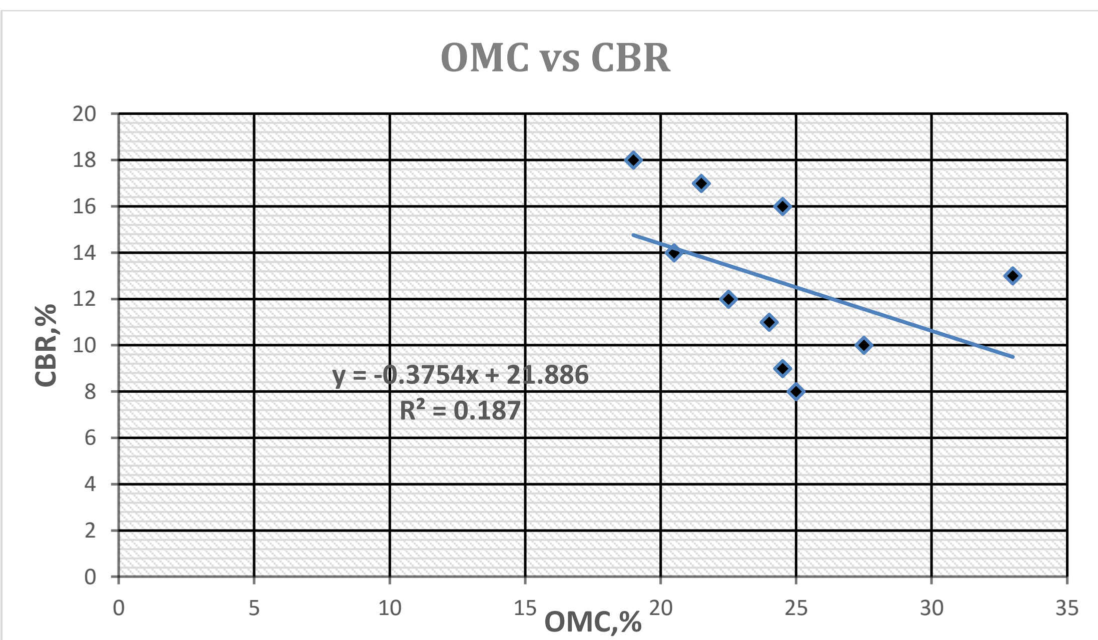


Fig 6.4 Scatter plot and best-fit curve of Optimum moisture content and CBR

E. Relation between MDD and CBR

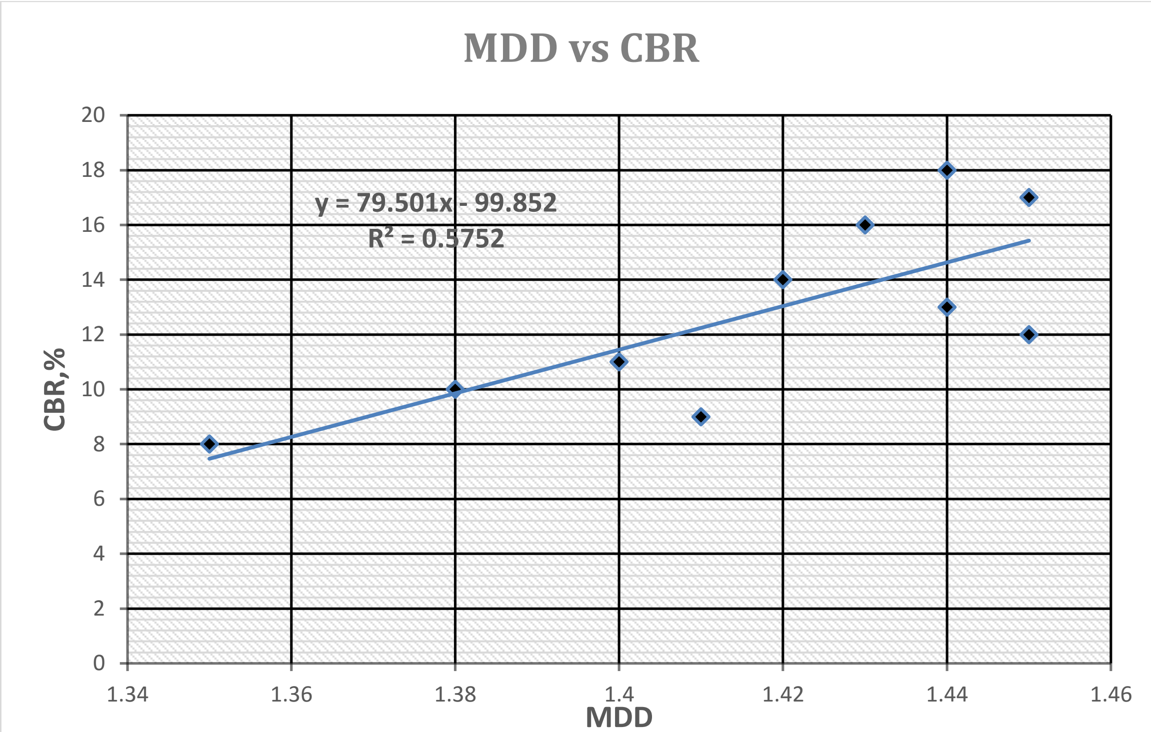


Fig 6.5 Scatter plot and best-fit curve of Maximum Dry Density and CBR

As shown above, California bearing ratio(CBR) is dependent variable and LL, PL, PI, OMC and MDD are independent variables. From the figures it can be observe that CBR has strong negative correlation with LL and on fig.6.1 and weakest negative correlation with plastic index as shown on fig.6.3. On the other hand, CBR has relatively strong positive correlation with MDD as on fig.6.5 shown. The plots also show the type of correlation among variables, in this case the relation seems non leaner.

In general, it can be conclude that for preliminary soil investigation the CBR of fine grained soils (especially silty clay) can be predicted from liquid limit, LL without significant errors.

6.2.2 Scatter plot for combined (primary & secondary) data

These are the scatter plots of the combined data. 10laboratory experimented samples and 114 from known consulting companies. Figures 6.6-6.11 are shown below.

A. Relation between LL and CBR

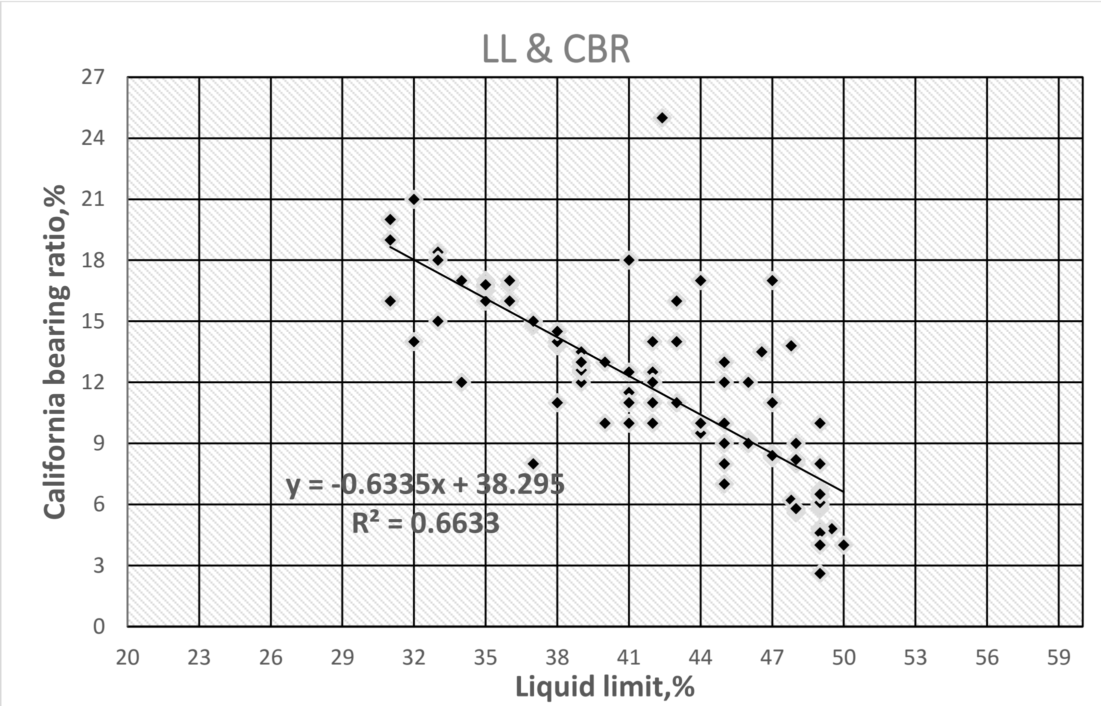


Fig 6.7 Scatter plot and best-fit curve of liquid limit and CBR

B. Relation between PL and CBR

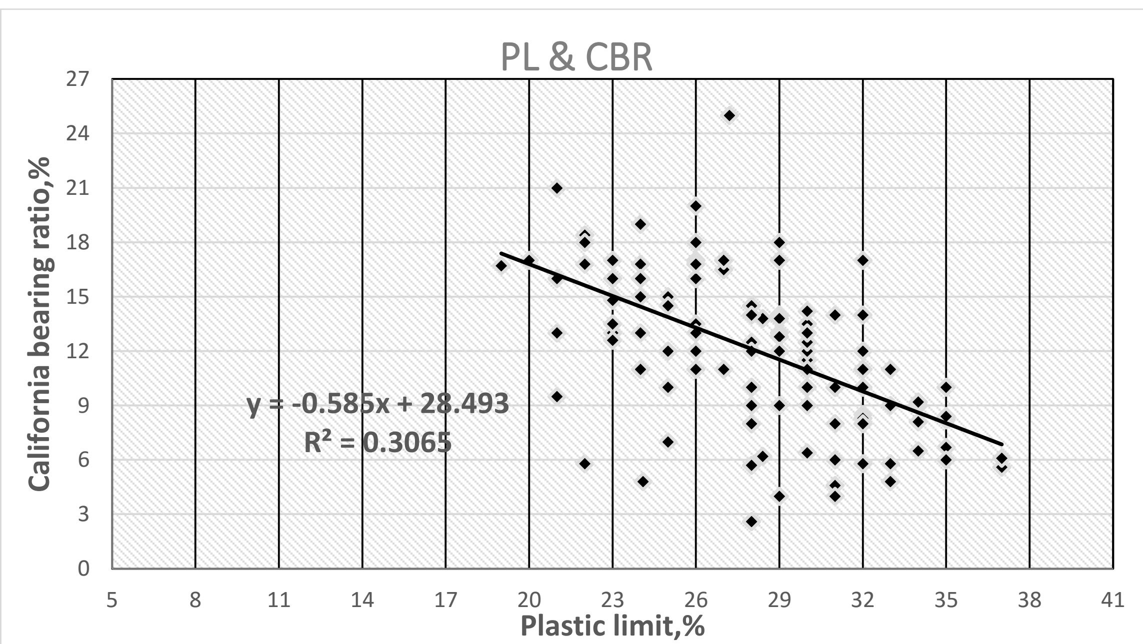


Fig 6.8 Scatter plot and best-fit curve of Plastic limit and CBR

C. Relation between PI and CBR

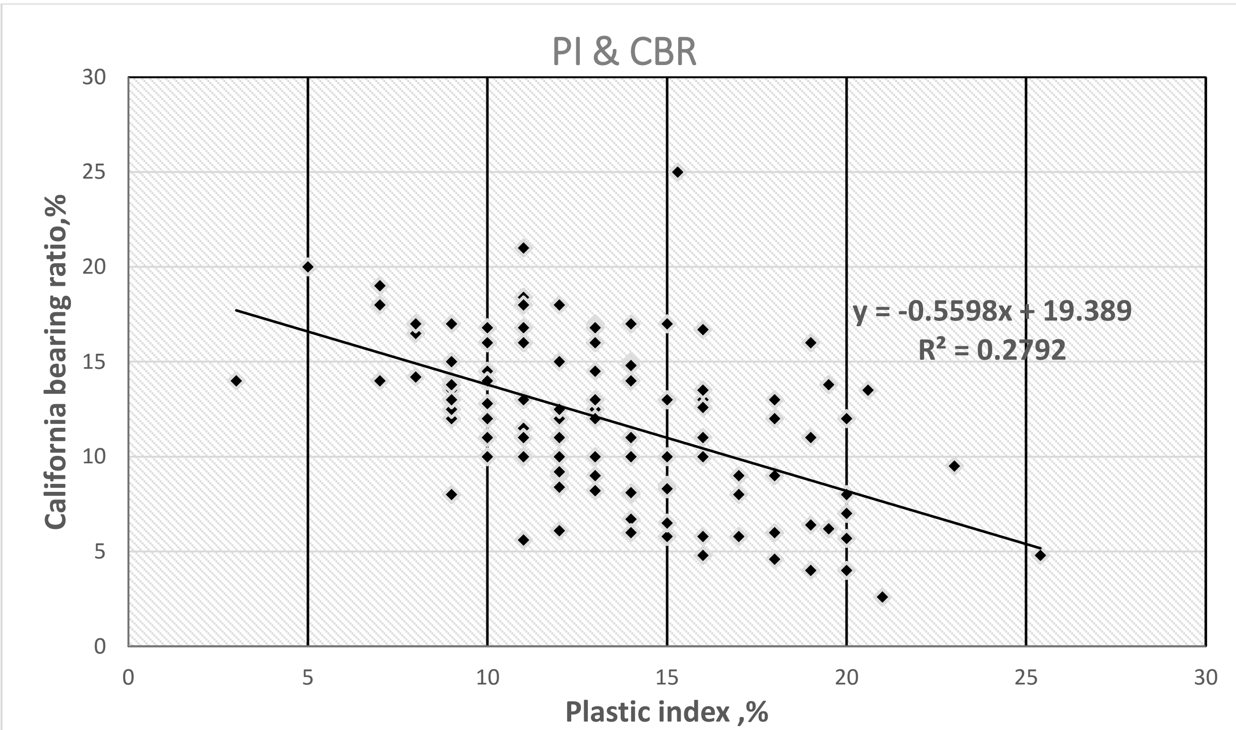


Fig 6.9 Scatter plot and best-fit curve of Plastic index and CBR

D. Relation between OMC and CBR

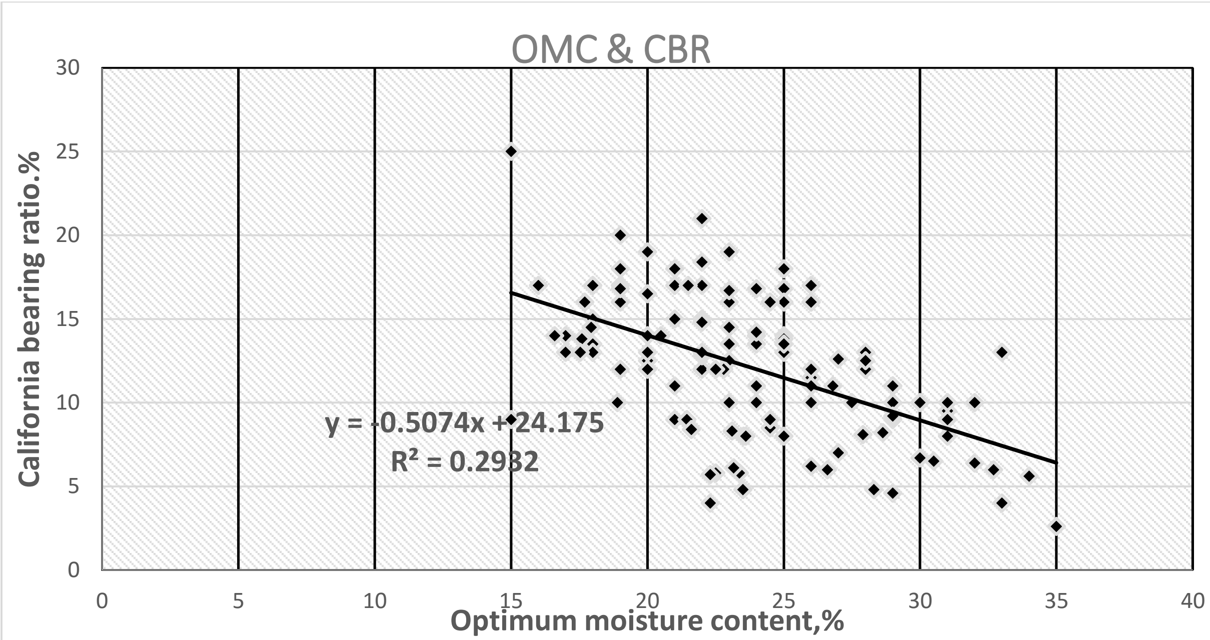


Fig 6.10 Scatter plot and best-fit curve of Optimum moisture content and CBR

E. Relation between MDD and CBR

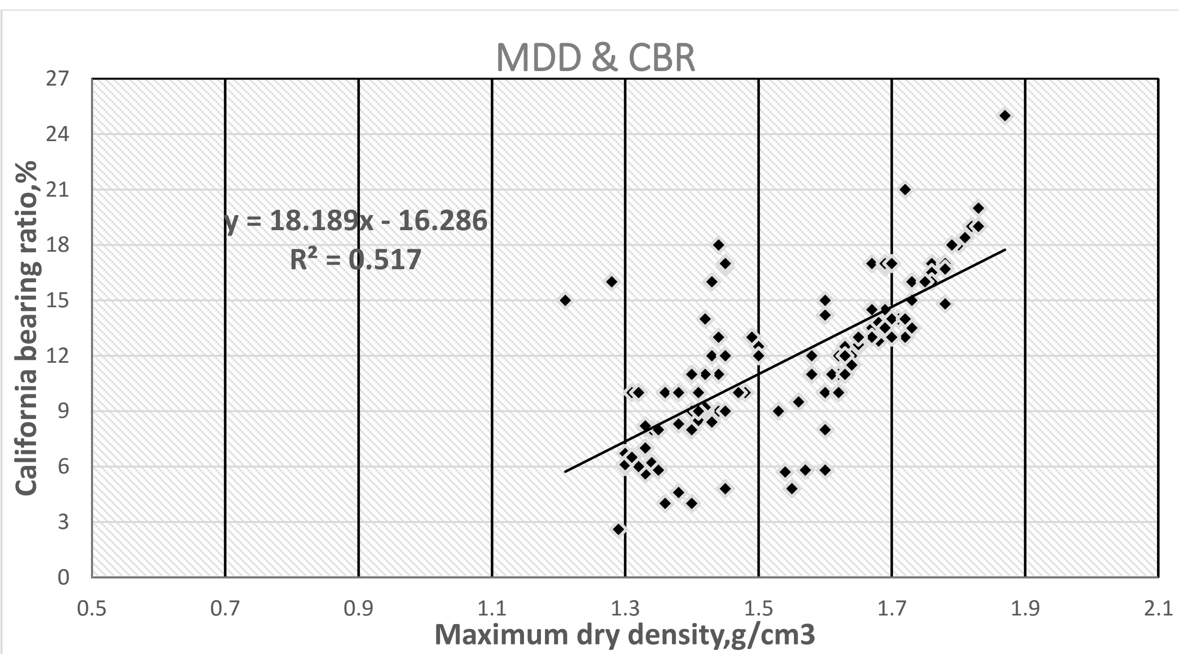


Fig 6.11 Scatter plot and best-fit curve of Maximum Dry Density and CBR

Similar to the primary data, the plots of the combined data shows strong negative correlation of CBR with LL, weakest correlation with P_{200} and relatively strong positive correlation with MDD.

6.3 Correlation and Regression analysis

Correlation and regression are statistical methods that are commonly used in the applied science to compare two or more variables. But they are quite different.

6.3.1 Correlation

Correlation measures the association between two variables and quantities the strength of their relationship. Correlation evaluates only the existing data.Correlation provides a numerical measure of the linear relationship between two continuous variables X and Y. The resulting correlation coefficient, “r value” is known as the Pearson product moment correlation coefficientafter the mathematician who first described it. X is known as the independent(explanatory) variablewhile Y is known as the dependent(response) variable. A significant advantage of thecorrelation coefficient is that it does not depend on the units of X and Y and can therefore be used to compare any two variables regardless of their units. [14]

An essential first step in calculating a correlation coefficient is to plot the observations in a “scatter gram” or “scatter plot” to visually evaluate the data for a potential relationship or the presence of outlying values. These scatter plots are shown on above figures 6.1-6.11.

The calculation of the correlation coefficient is mathematically complex, but readily performed by most computer statistics programs which is called Statistical Package for Social Science (SPSS). Correlation and correlation coefficient of the laboratory experimented samples and secondary data are present on tables 6.1 & 6.2 as shown below.

Table 6.1 Correlation coefficients (R) between variables for primary data

	Correlations						
		LL	PL	PI	OMC	MDD	CBR
LL	Pearson Correlation	1	0.535	0.435	0.482	-.707*	-.916**
	Sig. (2-tailed)		0	0	0.001	0.002	0
	N	10	10	10	10	10	10
PL	Pearson Correlation	0.535	1	-0.528	0.181	-0.629	-0.57
	Sig. (2-tailed)	0		0	0.003	0.004	0
	N	10	10	10	10	10	10
PI	Pearson Correlation	0.435	-0.528	1	0.291	-0.04	-0.313
	Sig. (2-tailed)	0	0		0	0.013	0
	N	10	10	10	10	10	10
OMC	Pearson Correlation	0.482	0.181	0.291	1	-0.21	-0.432
	Sig. (2-tailed)	0.001	0.003	0		0.002	0.014
	N	10	10	10	10	10	10
MDD	Pearson Correlation	-.707*	-0.629	-0.04	-0.21	1	.758*
	Sig. (2-tailed)	0.002	0.004	0.013	0.002		0.011
	N	10	10	10	10	10	10
	Sig. (2-tailed)	0.005	0.012	0.001	0	0.028	0.001
	N	10	10	10	10	10	10
CBR	Pearson Correlation	-.916**	-0.57	-0.313	-0.432	.758*	1
	Sig. (2-tailed)	0	0	0	0.014	0.011	
	N	10	10	10	10	10	10
*. Correlation is significant at the 0.05 level (2-tailed).							
**. Correlation is significant at the 0.01 level (2-tailed).							
Where LL ,PL, PI, OMC and CBR in % and MDD in g/cm³							

Table 6.2 Correlation coefficients (R) between variables for all data.

Correlations							
		LL	PL	PI	OMC	MDD	CBR
LL	Pearson Correlation	1	.648**	.680**	.353**	-.674**	-.814**
	Sig. (2-tailed)		0	0	0	0	0
	N	124	124	124	124	124	124
PL	Pearson Correlation	.648**	1	-0.092	.336**	-.506**	-.554**
	Sig. (2-tailed)	0		0.014	0	0	0
	N	124	124	124	124	124	124
PI	Pearson Correlation	.680**	-0.092	1	0.157	-.415**	-.528**
	Sig. (2-tailed)	0	0.014		0.031	0	0
	N	124	124	124	124	124	124
OMC	Pearson Correlation	.353**	.336**	0.157	1	-.470**	-.542**
	Sig. (2-tailed)	0	0	0.031		0	0
	N	124	124	124	124	124	124
MDD	Pearson Correlation	-.674**	-.506**	-.415**	-.470**	1	.719**
	Sig. (2-tailed)	0	0	0	0		0
	N	124	124	124	124	124	124
	Sig. (2-tailed)	0.003	0.004	0.003	0.005	0.002	0.451
	N	124	124	124	124	124	124
CBR	Pearson Correlation	-.814**	-.554**	-.528**	-.542**	.719**	1
	Sig. (2-tailed)	0	0	0	0	0	
	N	124	124	124	124	124	124
**. Correlation is significant at the 0.01 level (2-tailed).							
Where LL, PL, PI, OMC, P200 and CBR in percent (%) and MDD in g/cm3							

The result of 'r' value is influenced by different statically methods, such as the mean and standard deviation as well as markedly influence by a single outlying value. The square of 'r'value known as the coefficient of determination or R^2. It describes the proportion of change in the dependent variable CBR which is explained by each independent variable LL,PL,PI,OMC and MDD. For example, as it seen on the above table the value of r is 0.814 between the dependent CBR and independent variable LL. Therefore, coefficient of determination R^2 is 0. 663.This means about 66.3% of the change in CBR can be explained by a change in LL. And this is the same for all variables shown on the table. The larger the correlation coefficient, the larger the coefficient of determination, and the more influence changes in the independent variable have on the dependent variable.

6.3.2 Regression analysis

Regression uses the existing data to define a mathematical equation which can be used to predict the value of one variable based on the value of one or more other variables. The regression equation can therefore be used to predict the outcome of observations not previously seen or tested. [15]Regression analysismathematically describes the dependence of the Y variable on the X variable and constructs an equation which can be used to predict any value of Y for any value of X. Regression analysis can be two types. These are Simpleregression and multipleregression analysis.

A. SimpleRegression

Simple linear regression considers a single independent or predictor x and a dependent or response variableY.the dependent can be described by the model:

$$Y= b+ax \text{ for linear (6.1)}$$

Where Y is the dependent variable, X is the single independent variable, a is the Y-intercept of the regression line, and b is the slope of the line (also known as the regression coefficient).[16]

B. Multiple Regression analysis

A multiple regression is a model that contains more than one independent variable to describe the dependent one. The dependent can be described by the model:

$Y=a_0 + a_1x_1 + a_2x_2 + a_3x_3$…………for linear(6.2) $Y= a_0 + a_1x_1+ a_2x_2 + a_3x_3 + a_4x_1^2 + a_5x_2^2 + a_6x_3^2 \ldots + a_{n-2}x_1^n + a_n +x_3^n$ for nonlinear(6.3)

Where, a_1,a_2,a_3……..a_n= coefficients of the independent variables

X_1,x_2,x_3…….x_n= the independent variables

Y= the response (dependent variable)

In addition to deriving the regression equation, regression analysis also draws a line of best fit through the data points of the scatter gram. These "regression lines" may be linear, in which case the relationship between the variables fits a straight line, or nonlinear, in which case a polynomial equation is used to describe the relationship.

6.3.2.1 Regression analysis between CBR and LL

Table 6.3 summary model of Regression analysis between CBR and LL

Model Summary

R	R Square	Adjusted R Square	Std. Error of the Estimate
.814	.663	.661	2.447

The independent variable is LL.

Coefficients

	Unstandardized Coefficients		Standardized Coefficients	t	Sig.
	B	Std. Error	Beta		
LL	-.634	.041	-.814	-15.502	.000
(Constant)	38.295	1.711		22.379	.000

The model equation is then:

$CBR= 38.295- 0.634LL$ with $R^2 =0.663$ (6.4)

Where LL and CBR in percent (%)

6.3.2.2 Regression analysis between CBR and PL

Table 6.4 summary model of Regression analysis between CBR and PL

Model Summary

R	R Square	Adjusted R Square	Std. Error of the Estimate
.554	.306	.301	3.512

The independent variable is PL.

Coefficients

	Unstandardized Coefficients		Standardized Coefficients	t	Sig.
	B	Std. Error	Beta		
PL	-.585	.080	-.554	-7.342	.000
(Constant)	28.493	2.270		12.553	.000

The model equation is then:

CBR= 28.493- 0.585pL with R^2 =0.306 (6.5)

Where PL and CBR in percent (%)

6.3.2.3 Regression analysis between CBR and PI

Table 6.5 summary model of Regression analysis between CBR and PI

Model Summary

R	R Square	Adjusted R Square	Std. Error of the Estimate
.528	.279	.273	3.580

The independent variable is PI.

Coefficients

	Unstandardized Coefficients		Standardized Coefficients	t	Sig.
	B	Std. Error	Beta		
PI	-.560	.081	-.528	-6.874	.000
(Constant)	19.389	1.123		17.258	.000

The model equation is then:

CBR= 19.389- 0.560PI with R^2 =0.279 (6.6)

Where PI and CBR in percent (%)

6.3.2.4 Regression analysis between CBR and OMC

Table 6.6 summary model of Regression analysis between CBR and OMC

Model Summary

R	R Square	Adjusted R Square	Std. Error of the Estimate
.542	.293	.287	3.545

The independent variable is OMC.

Coefficients

	Unstandardized Coefficients		Standardized Coefficients	t	Sig.
	B	Std. Error	Beta		
OMC	-.507	.071	-.542	-7.114	.000
(Constant)	24.175	1.742		13.876	.000

The model equation is then:

CBR= 24.175- 0.507OMC with R^2 =0.293 (6.7)

Where OMC and CBR in percent (%)

6.3.2.5 Regression analysis between CBR and MDD

Table 6.7 summary model of Regression analysis between CBR and MDD

Model Summary

R	R Square	Adjusted R Square	Std. Error of the Estimate
.758	.575	.568	2.761

The independent variable is MDD.

Coefficients

	Unstandardized Coefficients		Standardized Coefficients	t	Sig.
	B	Std. Error	Beta		
MDD	-116.640	33.230	-4.611	-3.510	.001
MDD ** 2	43.524	10.716	5.335	4.062	.000
(Constant)	86.943	25.524		3.406	.001

The model equation is then:

$CBR= 24.175- 116.640MDD + 43.524MDD^2$ with $R^2 =0.575$ (6.8)

Where MDD in g/cm^3 and CBR in percent (%)

By carrying out multiple regressions the following model is found.

6.3.2.7 Multiple Regression analysis between CBR and all parameters.

Table 6.9 summary model of Regression analysis between CBR and All parameters

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.879[a]	.772	.756	2.07435

a. Predictors: (Constant), MDD^2, PI, OMC, PL, , LL,MDD

Coefficients [a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	69.555	20.316		3.424	.001
	LL	-.607	.209	-.780	-2.904	.004
	PL	.203	.210	.192	.965	.337
	PI	.188	.218	.177	.863	.390
	OMC	-.222	.049	-.237	-4.538	.000
	MDD	-53.305	26.464	-2.107	-2.014	.046
	MDD^2	19.273	8.627	2.362	2.234	.027

a. Dependent Variable: CBR

It can be notice that parameters which were significant on above tables become insignificant on table 6.9 therefore to determine the improve model we use stepwise regression analysis. This analysis works by excluding the parameters which have less influence on the prediction of CBR. And these are shown below on table 6.10.

Table 6.10 summary model of stepwise Regression analysis between CBR and included variables

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.814[a]	.663	.661	2.44713
2	.859[b]	.737	.733	2.17128
3	.872[c]	.761	.755	2.08044
4	.877[d]	.770	.762	2.04888

a. Predictors: (Constant), LL

b. Predictors: (Constant), LL, OMC

c. Predictors: (Constant), LL, OMC, MDD2

d. Predictors: (Constant), LL, OMC, MDD2, MDD

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	1439.035	1	1439.035	240.302	.000[b]
	Residual	730.589	122	5.988		
	Total	2169.624	123			
2	Regression	1599.176	2	799.588	169.604	.000[c]
	Residual	570.449	121	4.714		
	Total	2169.624	123			
3	Regression	1650.239	3	550.080	127.092	.000[d]
	Residual	519.385	120	4.328		
	Total	2169.624	123			
4	Regression	1670.073	4	417.518	99.459	.000[e]
	Residual	499.551	119	4.198		
	Total	2169.624	123			

a. Dependent Variable: CBR

b. Predictors: (Constant), LL

c. Predictors: (Constant), LL, OMC

d. Predictors: (Constant), LL, OMC, MDD2

e. Predictors: (Constant), LL, OMC, MDD2, MDD

Coefficient

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	38.295	1.711		22.379	.000
	LL	-.634	.041	-.814	-15.502	.000
2	(Constant)	41.521	1.616		25.693	.000
	LL	-.554	.039	-.712	-14.292	.000
	OMC	-.272	.047	-.290	-5.828	.000
3	(Constant)	31.484	3.307		9.521	.000
	LL	-.451	.048	-.580	-9.455	.000
	OMC	-.217	.047	-.232	-4.578	.000
	MDD2	1.822	.530	.223	3.435	.001
4	(Constant)	72.663	19.222		3.780	.000
	LL	-.421	.049	-.541	-8.603	.000
	OMC	-.220	.047	-.235	-4.701	.000
	MDD2	20.021	8.389	2.454	2.387	.019
	MDD	-55.860	25.699	-2.208	-2.174	.032

a. Dependent Variable: CBR

From the above summary analysis table, 4 models are presented.

- ✓ Equation of model 1;

CBR =38.295 – 0.634LL with R^2=0.663 (6.10)

- ✓ Equation of model 2;

CBR = 41.521 –0.554LL – 0.272OMC with R^2=0.737(6.11)

- ✓ Equation of model 3;

CBR=31.484 – 0.451LL- 0.217OMC + 1.822MDD^2 with R^2= 0.761 (6.12)

It can be observe that the value of R square is greater on model 4 compared to other models. As well as it is significant. So the improved equation will be:

- ✓ Equation of model 4;

$$CBR=72.663–0.421LL–0.220OMC-55.86MDD+20.021MDD^2 \text{ with } R^2=0.770 \quad (6.13)$$

Where LL, OMC and CBR in percent (%) and MDD in g/cm^3

The above equation is therefore the final prediction equation of CBR. This means the equation model is able to express 77% of the variation of CBR. On other hand about 23% CBR is not explained by the develop equation.

Table 6.11 Summary of regression equations and their corresponding R^2 value in predicting CBR

M/N	Coefficients of predictors						Output Equations	R^2
	LL	PL	PI	OMC	MDD	MDD^2		
1	-0.634						CBR= 38.295- 0.634LL	0.663
2		-0.59					CBR= 28.493- 0.585pL	0.306
3			-0.56				CBR= 19.389- 0.560PI	0.279
4				-0.507			CBR= 24.175- 0.507OMC	0.293
5					-116.6	43.524	$CBR= 24.175- 116.640MDD + 43.524MDD^2$	0.575
6	– 0.554			– 0.272			CBR = 41.521 – 0.554LL – 0.272OMC	0.737
7	– 0.451			-0.217		1.822	$CBR=31.484 – 0.451LL- 0.217OMC + 1.822MDD^2$	0.761
8	–0.421			–0.220	-55.86	20.021	$CBR=72.663–0.421LL–0.220OMC-55.86MDD+20.021MDD^2$	0.77

Where LL, PL, PI, OMC and CBR in percent (%) and MDD in g/cm^3

Note: M/N=model number, LL=liquid limit, PL=plastic limit, PI=plastic limit, CBR=California bearing ratio, OMC=Optimum moisture content and MDD=maximum dry density.

6.4 Comparisons between the actual CBR and developed equation

The purposes of having laboratory tested samples were to have control points for the collected data from different consulting firms, for verification and test of the developed equations. The comparison table is present below.

Table 6.12 comparison of actual value with developed equation.

sample code	LL	OMC	MDD	Actual CBR	Developed equation	Variation, %
SS1	49	25	1.35	8	7.6112725	5.10726032
SS2	48	24.5	1.41	9	8.1061501	11.0268116
SS3	42	20.5	1.42	14	11.5201444	21.5262545
SS4	47	24	1.4	11	8.63316	27.4156856
SS5	46	22.5	1.45	12	9.4441525	27.0627513
SS6	41	19	1.44	18	12.2991456	46.3516295
SS7	49	27.5	1.38	10	7.0251924	42.3448559
SS8	45	33	1.44	13	7.5351456	52.524868
SS9	44	21.5	1.45	17	10.5061525	56.809949
SS10	43	24.5	1.43	16	10.2311429	56.3852656

6.5 Discussions

As the aim of the thesis is to formulate a correlation between the variables of the stated parameters, which is expected to predict the value of CBR, an attempt is made to obtain which one of the predictors can be strongly related with dependent variables. In predicting the value of CBR two options or statistical methods are used.

The first is the correlation table. Both from the primary and secondary data correlation table it can be noted that CBR is strongly related with LL from other atterberg limits as well as with

MDD from compaction test. The correlation table also helps to show that the positive and negative (the increase or decrease) effect of each variable on the dependent variable CBR as well as level of significant and number of samples used.

The other method is using regression analysis. First each independent variable is analyzed its effect and prediction capacity one by one. The analysis is then expressed using equations. From these equations it can be observed that CBR has relatively strong correlation with LL and MDD which have R^2=0.663 and R^2=0.575 respectively.

After having these equations and the corresponding coefficient of determination values, multiple regression is used by stepwise method. This is done to include and exclude the more influential and not influential variables respectively. The result of this analysis gave 4 models with their corresponding coefficient of determination values. According to the analysis the model with large ‘r ‘value is the more that explains or predicates CBR.Therefore, model number 4 with R^2=0.770 is chosen as final equation to predict CBR. This means the CBR value is more influenced by LL,OMC and MDD of the silty clay soil.

As the liquid limit of the soil increase, it shows the weak quality of the soil type for being used as pavement material. On the other hand, higher dry density results higher CBR value at constant moisture content. This condition might be the possible reason that the CBR value has good correlation with LL,OMC and MDD.

7. CONCLUSION AND RECOMMENDATION

The main focus of this research is on the outcome of predicting of CBR from index properties of fine grained silty clay soils. Considering this all the necessary data and statistical methods are used in order to achieve the goal.Based on these a number of conclusions and recommendations are made.

7.1 Conclusion

- ✓ Liquid limit highly affects the value of CBR with R^2=0.663. This means any increment in LL will decrease the value of CBR by 66% since they have inversely proportional relation.
- ✓ The prediction of CBR value by PI and PL expresses with R^2=0.279 and R^2=0.306 in which the can predict less than 50% with negative slope.
- ✓ Maximum dry density has directly proportional relation with CBR. The increment in MDD is the increment in CBR by 57% which mean it has R^2=0.575.
- ✓ The prediction of CBR using OMC is weak with R^2=0.293 by inversely proportional relation.

Generally,the larger the correlation coefficient, the larger the coefficient of determination, and the more influence changes in the independent variable have on the dependent variable.

- ✓ Liquid limit and maximum dry density of the soils affects the value of California bearing ratio.
- ✓ From the developed equations, the combination of LL, OMC and MDD gives relatively strong correlation and can be able to express the CBR value about 77% with R^2=0.770
- ✓ The comparison between the developed equation and the actual CBR value is relatively good. The small percent difference comes because of that the developed equation can be express about 77%. In other word it is not a perfect relation about 23% a change in CBR cannot be explain by the predictors of LL, OMC and MDD.

7.2 Recommendation

Even if this thesis is done with high effort to get good results as much as can, but there are some limits that may fulfill by other studies.

i. The predicted correlation of CBR value is limited to the standard compaction effort. However, when the soil is compacted at different energy the result may vary. Therefore, it is better to do further study using different compaction energies such as modified proctor test.
ii. The sample size which were used as a control point may affect the required result. Therefore, more number of sample size with accurate laboratory test may increase the prediction result.
iii. In order to get the perfect relation among the variables and to develop the extent of applicability further works are required.
iv. For comparison and better results further studies can be done using Ridge Regression and Artificial Neural Network.

REFERENCES

[1] Arora, K.R., *Soil Mechanics and Foundation Engineering*, Re-print Standard Publishers Distributer, Nai Sarak, Delhi, 2004.

[2] Atsbeha Nerea "prediction of compaction characteristics from atterberg limits for fine-grained soils" Thesis for Master of Science degree, Faculty of Technology, Addis Ababa University, 2012.

[3] Bao Thach Nguyen, Abbas Mohajerani "Prediction of California Bearing Ratio from Physical Properties of Fine-Grained Soils" International Journal of Civil, Structural, Construction and Architectural Engineering Vol:9, No:2, 2015.

[4] Breytenbach I.J "The relationship between index testing and California Bearing Ratio values for natural road construction materials in South Africa" University of Pretoria, South Africa,2009.

[5] Dilip Kumar Talukdar "A Study of Correlation Between California Bearing Ratio (CBR) Value with Other Properties of Soil"International Journal of Emerging Technology and Advanced Engineering. ISSN 2250-2459, ISO 9001:2008 CertifiedJournal,Volume 4, Issue 1, January 2014.

[6] Douglas C. Montgomery. "Applied Statistics and Probability for Engineers" Third Edition, Arizona State University.2002.

[7] Farshad Amini, "Potential Applications of Dynamic and Static Cone Penetrometers in Moot Pavement Design and Construction" final report, Mississippi Department of Transportation and The U.S. Department of Transportation Federal Highway Administration, Jackson State University, Mississippi, September 2003.

[8] Gopal Ranjan." Basic and applied soil mechanics", second edition, printed in India at print perfect, New Delhi-110064,2003.

[9] Harini HN and Sureka Naagesh "predicting cbr of fine grained soils by artificial neural network and multiple linear regression" Volume 5, Issue 2, pp. 119-126, February 2014.

[10] Habtamu Kassahun, "Examining Atterberg Limits for Expansive Soils." MSc Thesis, Addis Ababa University, Addis Ababa. 2006.

[11] Joseph E. Bowles"Foundation analysis and design" Fifth Edition Printed in Singapore,1997.

[12] Lance Midgley, Pavement Design, A Guide to the structural Design of Road Pavement, Australia, 1997.

[13] Muni Budhu, "Soil mechanics and Foundation." University of Arizona. Hamilton Printing Company,2010.

[14] Murthy,N.V.S. "Geotechnical Engineering, principle and practice of soil mechanics and foundation engineering." New York, 2001.

[15] Paul W. M. and et.al, Subsurface Investigations (Geotechnical Site Characterization), FHWA-NHI-01-031, technical report, National Highway Institute Federal Highway Administration U.S. Department of Transportation, Ryan R. Berg & Associates Inc., Woodbury, USA, May 2002.

[16] Radhakrishnan,G Anjan,M Kumar and GVR PrasadaRaju,"Swelling Properties of Expansive Soils Treated with Chemical sand Fly ash"; American Journal of Engineering Research (AJER) e-ISSN : 2320-0847 p-ISSN : 2320-0936 Volume-03, Issue-04, pp-245-250, 2014.

[17] Shampo MA. "Statistics for clinicians Regression." Mayo Clin Proc; 56:452-454, 1981.

[18] Yang, H. Pavement Design and Analysis, Pearson Education Inc., New Saddle River, NJ, 2004.

[19] Yared Leliso "Correlation of CBR value with soil index properties for Addis Ababa subgrade soils" Addis Ababa university, Addis Ababa ,2013.

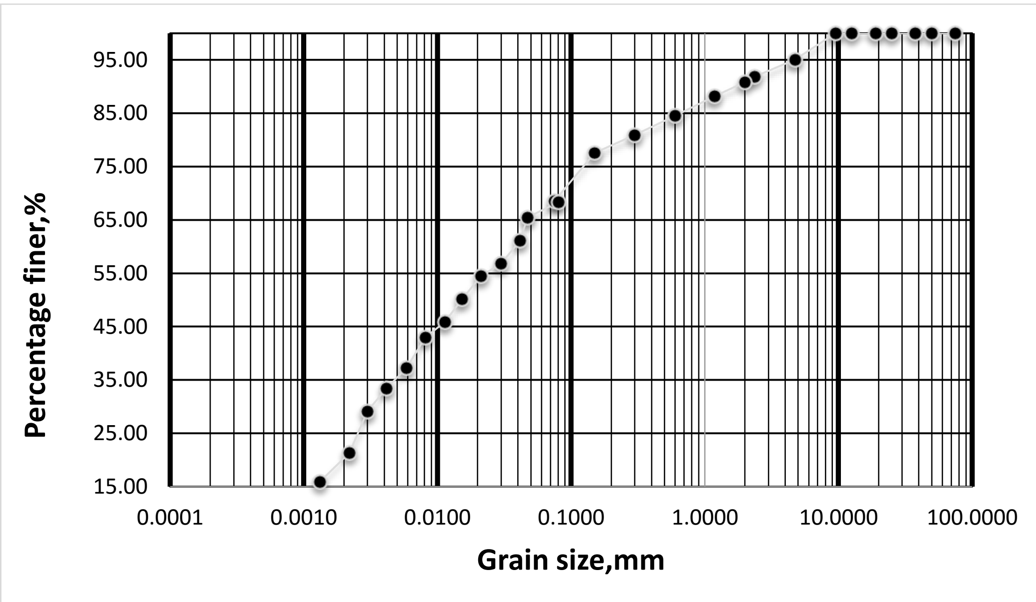
[20] Zelalem Worku, "Prediction of California Bearing Ratio (CBR) value from index properties of soil". Thesis for Master of Science degree, Faculty of Technology, Addis Ababa University, 2010.

APPENDIX

A. Laboratory experimented tests

1. Sample 1

a) Grain size analysis.



b) Specific gravity

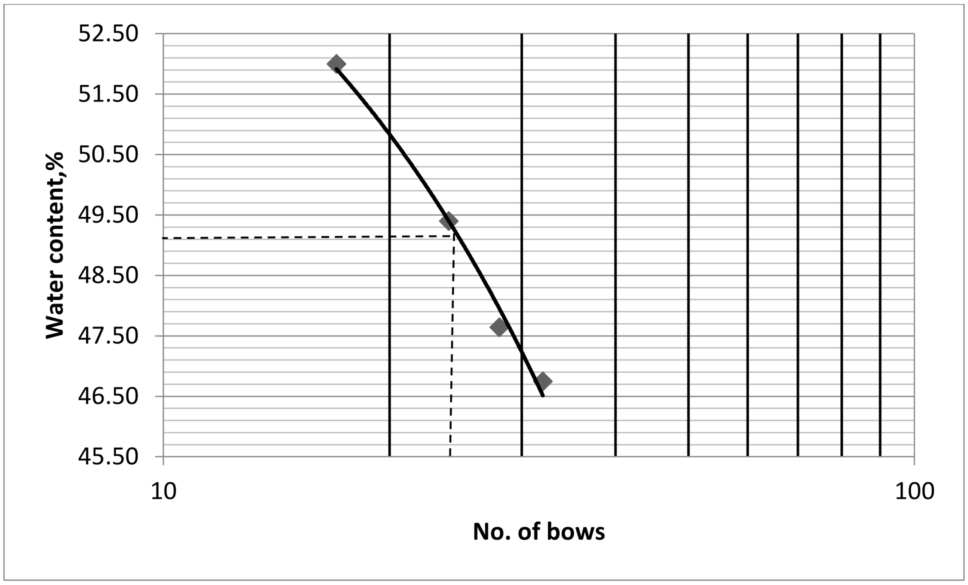
Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	178	177.9
Temperature, T_x(^{o}c)	23.2	23.3
Weight of pycnometer + water at T_x , W_{pw}(atT_x) (g)	162.60	162.40
Weight of dry soil,w_s (gm)	25	25
Conversion factor, K	0.9993	0.9993
Specific gravity of soil at 20°c.	2.60	2.63

Average specific gravity of soil.	2.62

c) Atterbrg limit

	Liquid Limit					Plastic Limit	
Trial No	1	2	3	4		1	2
Container No	L-1	T-2	BH	T10		T-1	T-2
Mass of container, g	15	15.8	15	15.2		15.7	15.4
Mass of container + Wet soil, g	39.80	47.10	39.80	38.00		23.00	22.60
Mass of container + Dry soil, g	31.90	37.00	31.60	30.20		21.30	20.80
Mass of water, g	7.90	10.10	8.20	7.80		1.70	1.80
Mass of dry soil, g	16.90	21.20	16.60	15.00		5.60	5.40
Water content, %	46.75	47.64	49.40	52.00		30.36	33.33
No of blows	32	28	24	17		------	------

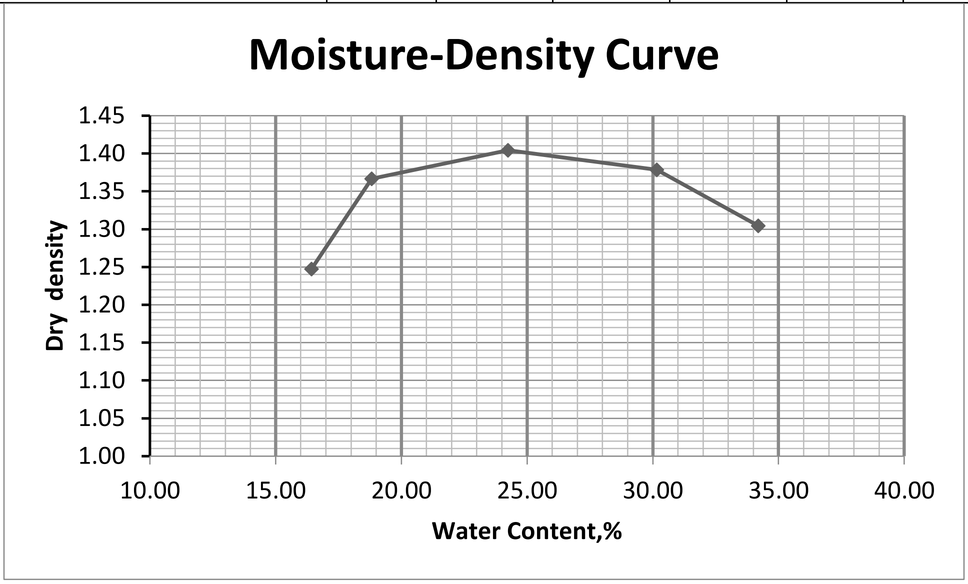
Liquid Limit, % = 49		Plastic Limit, % =32			PI, % =	17



d) Compaction test

Moisture content Vs dry density computation table						
Trial No.		1	2	3	4	5
Mass of Mold, g		3137.4	3137.4	3137.4	3137.4	3137.4
Mass of mold + Compacted Soil, g		4511	4670.4	4780.6	4810.7	4740
Mass of Compacted soil, g		1373.6	1533	1643.2	1673.3	1602.6
Volume of Mold, cm3		944	944	944	944	944
Bulk density, g/cm3		1.46	1.62	1.74	1.77	1.70
Water Contenet, %		17.37	20.83	24.23	28.71	31.76
Dry density, g/cm3		1.24	1.34	1.40	1.38	1.29
Water Content						
Conatainer No		L-1	T-3	WI	W-P	3-F
Mass of container, g		15	15.2	15.8	15.8	15.8
Mass of container + wet		40	50	39.9	41.8	54.8

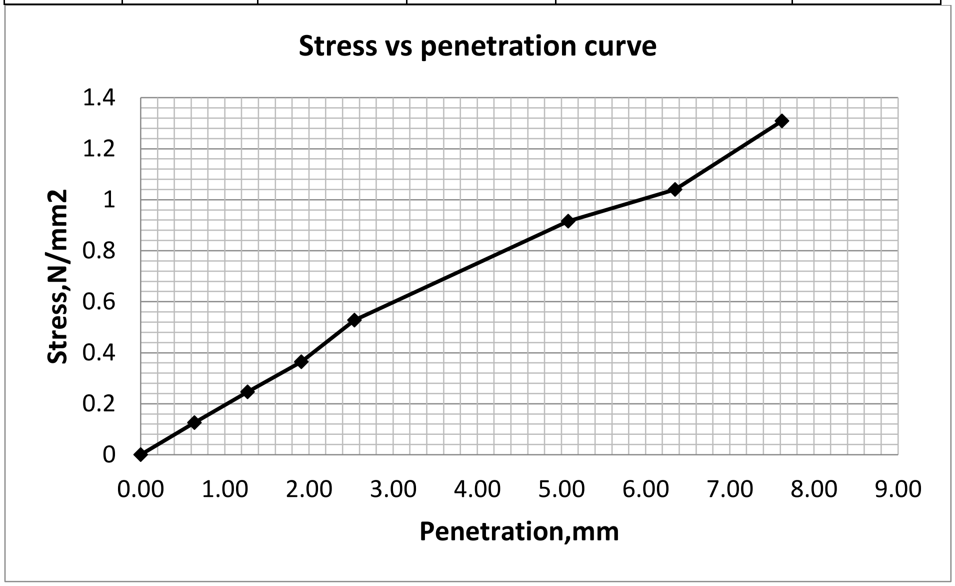
soil, g						
Mass of ontainer + Dry soil, g		36.3	44	35.2	36	45.4
Mass of Water, g		3.7	6	4.7	5.8	9.4
Mass of Dry soil, g		21.3	28.8	19.4	20.2	29.6
Water content, %		17.37	20.83	24.23	28.71	31.76
Dry Unit Weight, g/cm3		1.24	1.34	1.40	1.38	1.29



e) California bearing ratio

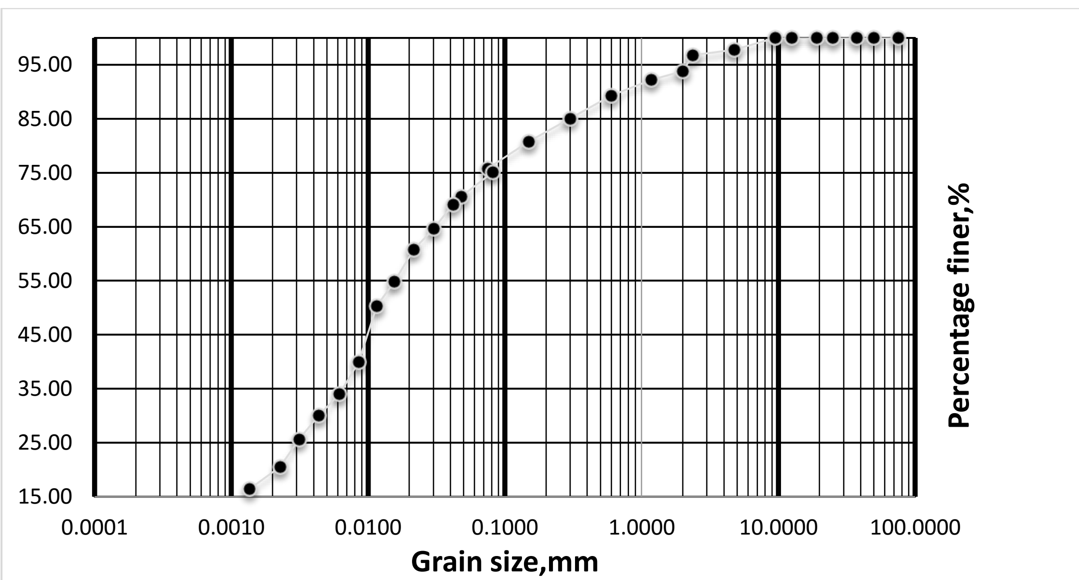
Penet.	Ring Reading	load	Stress	Standard stress		CBR
(mm)	(Div.)	N	N/mm2	(N/mm^2)		%
0.00	0.0	0	0			
0.64	9.5	244.2165	0.12621			
1.27	18.5	475.5795	0.245778			
1.91	27.4	704.3718	0.364016			
2.54	39.7	1020.568	0.527425	6.9		7.643845
5.08	68.9	1771.212	0.915355	10.3		8.886944
6.35	78.3	2012.858	1.040237			

7.62	98.5	2532.14	1.308599		



2) Sample 2

a) Grain size analysis

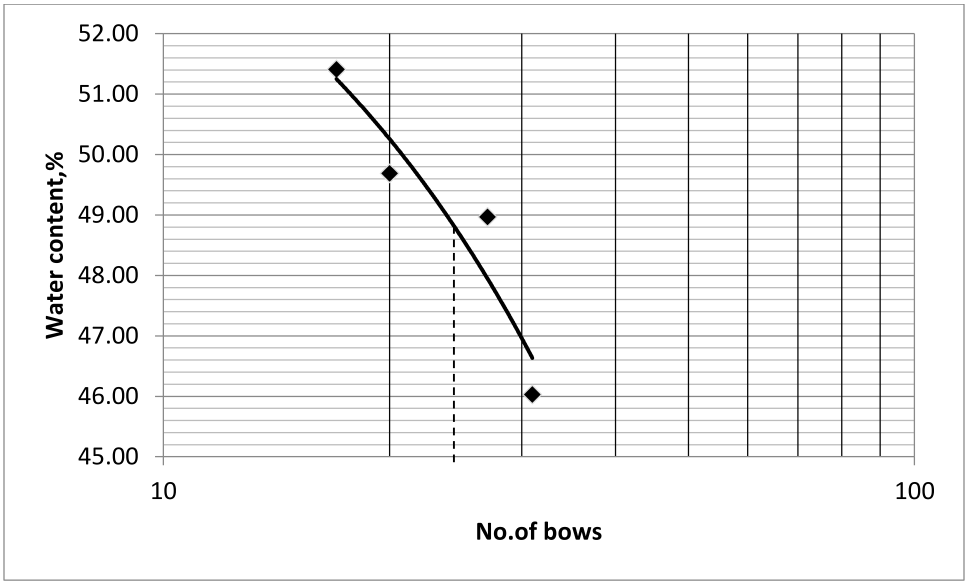


b) Specific gravity

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	178.3	163.2
Temperature, T_x(°c)	23.1	23.4
Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g)	162.80	147.20
Weight of dry soil , w_s (gm)	25	25
Conversion factor , K	0.9993	0.9993
Specific gravity of soil at 20°c.	2.63	2.78
Average specific gravity of soil .	2.70	

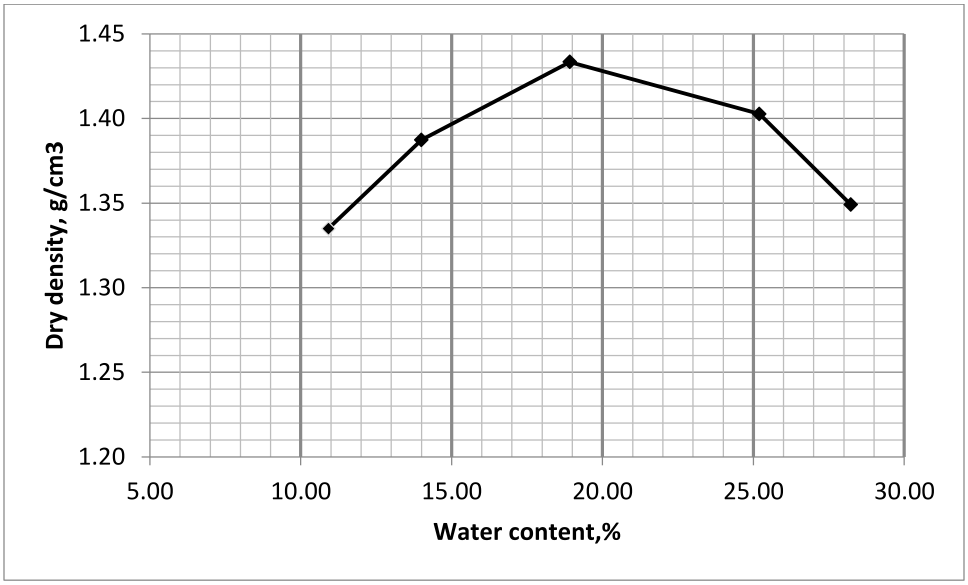
c) Atterberg limit

	Liquid Limit					Plastic Limit	
Trial No	1	2	3	4		1	2
Container No	T-3	T-4	W-2-P	WT2		A-1	T-1
Mass of container, g	15.6	15.4	15	15.7		15.7	15.3
Mass of container + Wet soil, g	34.00	37.00	39.10	37.20		20.80	21.60
Mass of container + Dry soil, g	28.20	29.90	31.10	29.90		19.60	20.20
Mass of water, g	5.80	7.10	8.00	7.30		1.20	1.40
Mass of dry soil, g	12.60	14.50	16.10	14.20		3.90	4.90
Water content, %	46.03	48.97	49.69	51.41		30.77	28.57
No of blows	31	27	20	17		------	------
Liquid Limit, % =		48	Plastic Limit, =30%				PI=18%



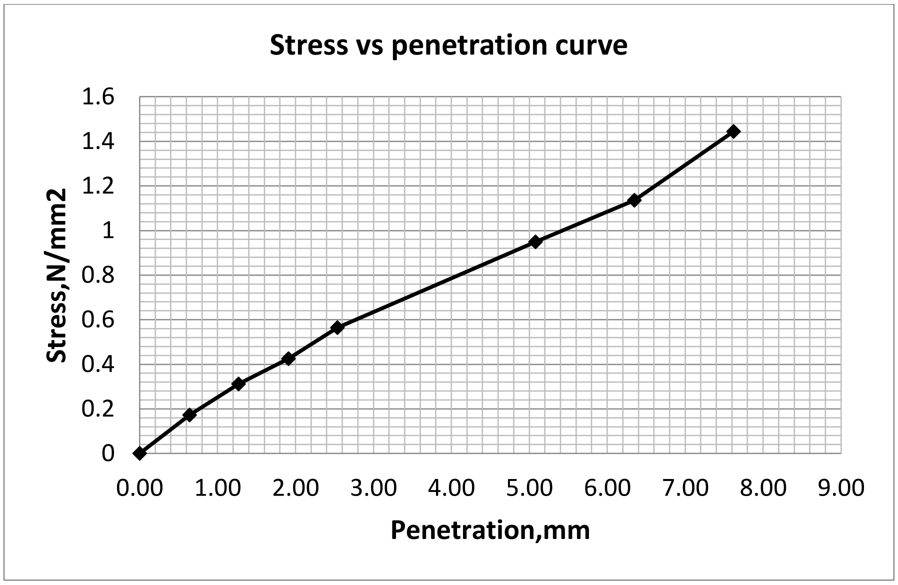
d) **Compaction test**

Moisture content Vs dry density computation table					
Trial No.	1	2	3	4	5
Mass of Mold, g	3137.4	3137.4	3137.4	3137.4	3137.4
Mass of mold + Compacted Soil, g	4575.9	4705	4815	4860.6	4801.5
Mass of Compacted soil, g	1438.5	1567.6	1677.6	1723.2	1664.1
Volume of Mold, cm3	944	944	944	944	944
Bulk density, g/cm3	1.52	1.66	1.78	1.83	1.76
Water Contenet, %	20.30	22.27	24.54	29.02	30.87
Dry density, g/cm3	1.27	1.36	1.43	1.41	1.35
Water Content					
Conatainer No	A-1	A-3	F-4	3-B	T-1
Mass of container, g	15.6	15.2	15.2	15.8	15.4
Mass of container + wet soil, g	39.9	41	42.1	44.7	45.5
Mass of ontainer + Dry soil, g	35.8	36.3	36.8	38.2	38.4
Mass of Water, g	4.1	4.7	5.3	6.5	7.1
Mass of Dry soil, g	20.2	21.1	21.6	22.4	23
Water content, %	20.30	22.27	24.54	29.02	30.87
Dry Unit Weight, g/cm3	1.27	1.36	1.43	1.41	1.35



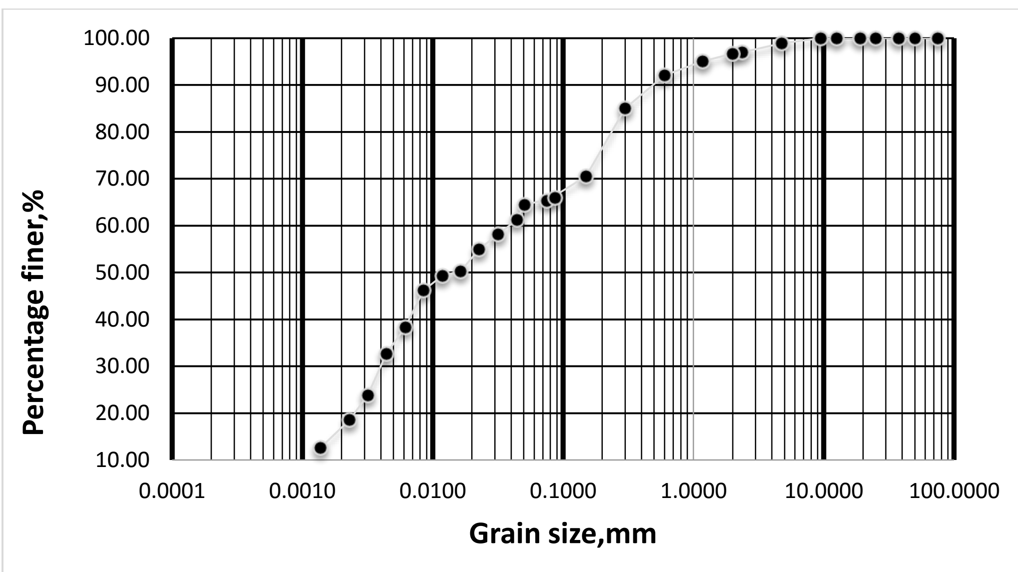
f) California bearing ratio

Penet.	Ring Reading	load	Stress	Standard stress	CBR
(mm)	(Div.)	N	N/mm2	(N/mm^2)	%
0.00	0.0	0	0		
0.64	13.0	334.191	0.172709		
1.27	23.5	604.1145	0.312204		
1.91	32.0	822.624	0.425129		
2.54	42.5	1092.548	0.564624	6.9	8.182957
5.08	71.4	1835.48	0.948568	10.3	9.209402
6.35	85.5	2197.949	1.135891		
7.62	108.7	2794.351	1.444109		



3) Sample 3

a) Grain size analysis



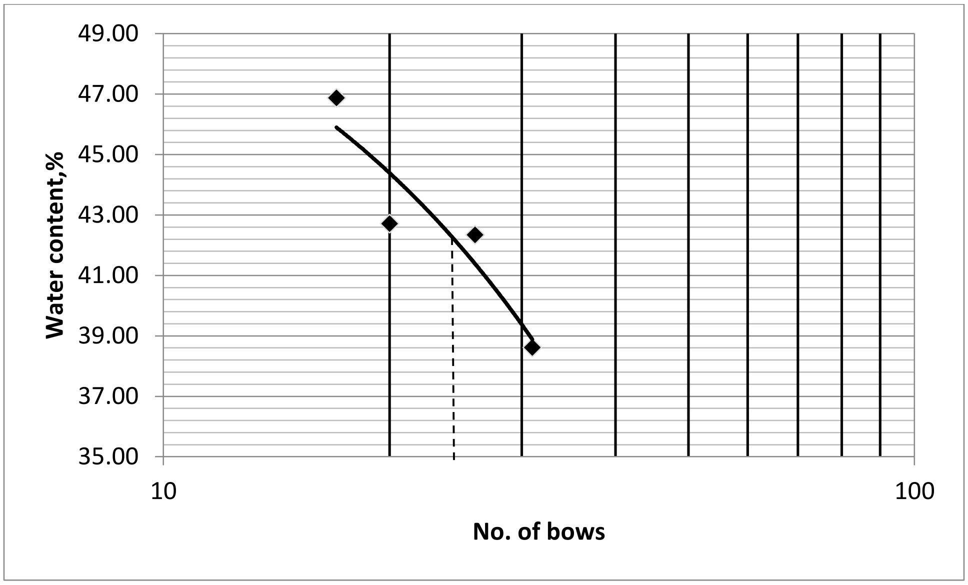
b) Specific gravity

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	178.3	162.8
Temperature, T_x(°c)	20.2	22
Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g)	162.70	147.30
Weight of dry soil , w_s (gm)	25	25
Conversion factor , K	1	0.9996
Specific gravity of soil at 20°c.	2.66	2.63
Average specific gravity of soil .	2.65	

c) Atterberg limit

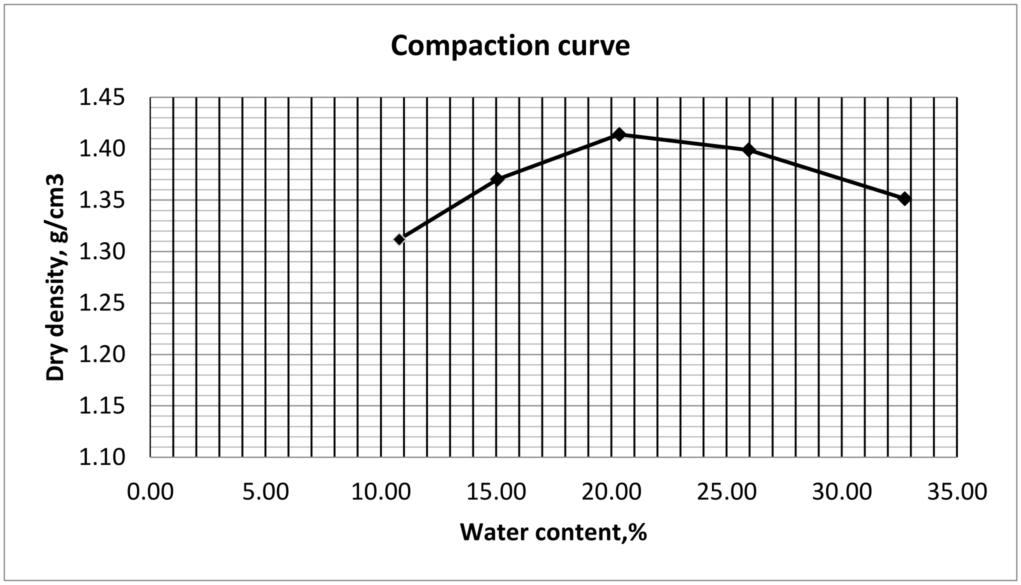
	Liquid Limit					Plastic Limit	
Trial No	1	2	3	4		1	2
Container No	WI1	WI2	B-6	T-6		WI3	WI3
Mass of container, g	15.4	15.6	15.6	15.6		15.8	15.9
Mass of container + Wet soil, g	35.50	43.50	44.00	43.80		22.40	22.10
Mass of container + Dry soil, g	29.90	35.20	35.50	34.80		20.80	20.60
Mass of water, g	5.60	8.30	8.50	9.00		1.60	1.50
Mass of dry soil, g	14.50	19.60	19.90	19.20		5.00	4.70
Water content, %	38.62	42.35	42.71	46.88		32.00	31.91
No of blows	31	26	20	17		------	------

Liquid Limit, % =	42	Plastic Limit, =32%			PI, % =10	



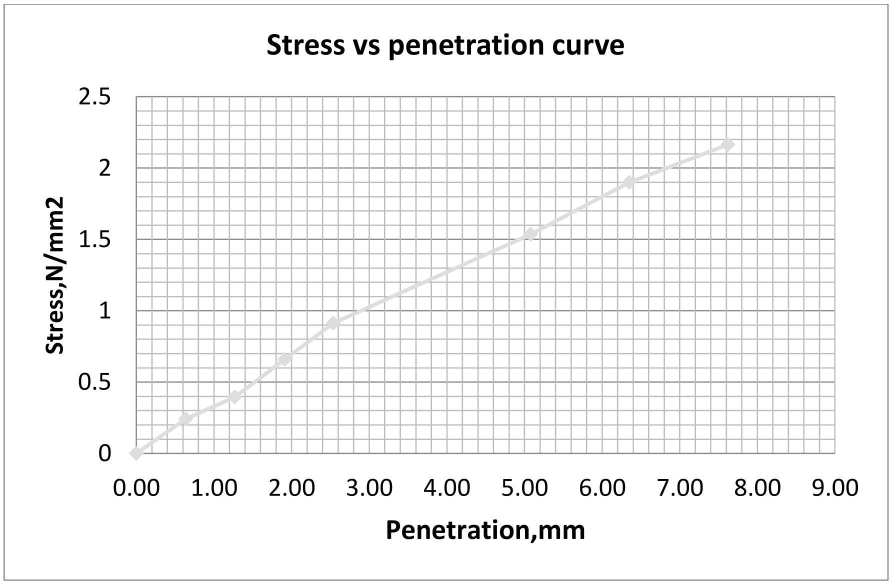
d) **Compaction test**

Moisture content Vs dry density computation table					
Trial No.	1	2	3	4	5
Mass of Mold, g	3137.4	3137.4	3137.4	3137.4	3137.4
Mass of mold + Compacted Soil, g	4509.7	4625.8	4743.7	4800.8	4830.7
Mass of Compacted soil, g	1372.3	1488.4	1606.3	1663.4	1693.3
Volume of Mold, cm3	944	944	944	944	944
Bulk density, g/cm3	1.45	1.58	1.70	1.76	1.79
Water Contenet, %	10.80	15.07	20.35	25.97	32.74
Dry density, g/cm3	1.31	1.37	1.41	1.40	1.35
Water Content					
Conatainer No	wt	T-3	W-3	T-1	T-5
Mass of container, g	15.5	15.7	15.7	15.3	15.7
Mass of container + wet soil, g	47.3	40.9	42.9	47.8	60.7
Mass of ontainer + Dry soil, g	44.2	37.6	38.3	41.1	49.6
Mass of Water, g	3.1	3.3	4.6	6.7	11.1
Mass of Dry soil, g	28.7	21.9	22.6	25.8	33.9
Water content, %	10.80	15.07	20.35	25.97	32.74
Dry Unit Weight, g/cm3	1.31	1.37	1.41	1.40	1.35



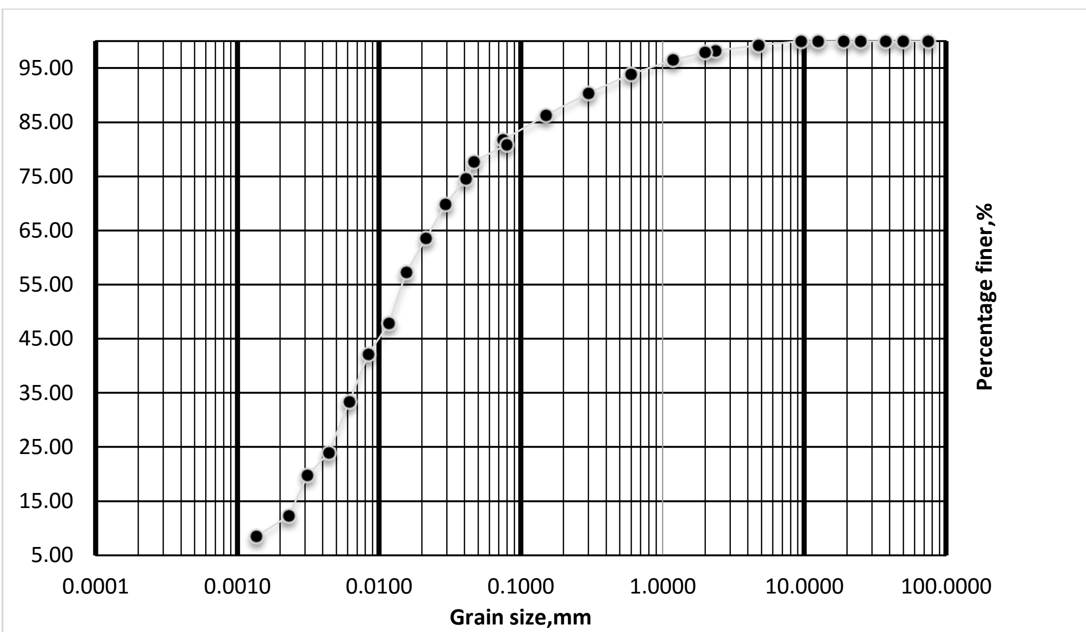
g) California bearing ratio

Penet.	Ring Reading	load	Stress	Standard stress	CBR
(mm)	(Div.)	N	N/mm2	(N/mm^2)	%
0.00	0.0	0	0		
0.64	18.3	470.4381	0.24312		
1.27	29.8	766.0686	0.395901		
1.91	49.6	1275.067	0.658949		
2.54	68.7	1766.071	0.912698	6.9	13.22751
5.08	115.6	2971.729	1.535777	10.3	14.91046
6.35	143.0	3676.101	1.899794		
7.62	163.0	4190.241	2.165499		



4) Sample 4

a) Grain size analysis

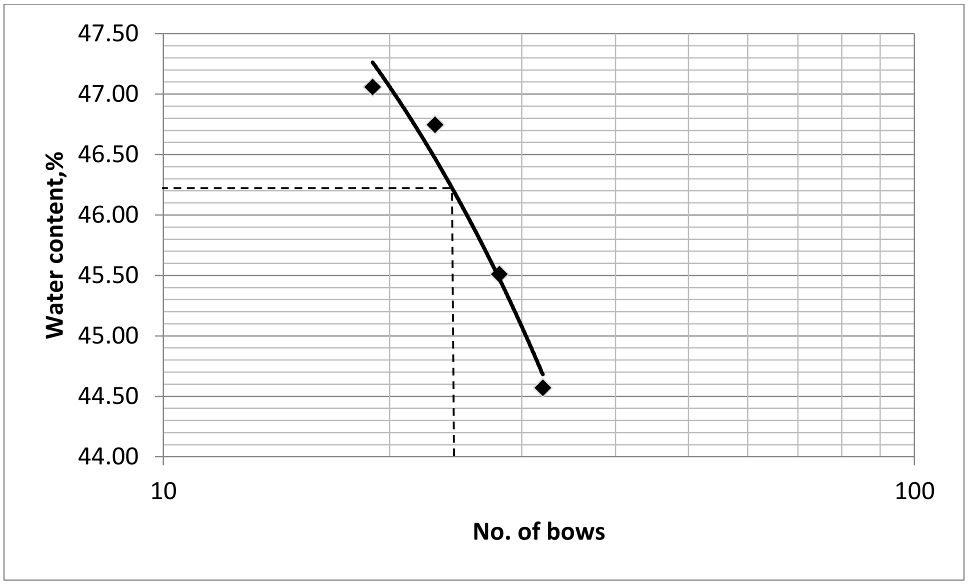


b) Specific gravity

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	178.2	178.5
Temperature, T_x(°c)	23	23.1
Weight of pycnometer + water at T_x , W_{pw}(atT_x) (g)	162.50	163.00
Weight of dry soil , w_s (gm)	25	25
Conversion factor , K	0.9993	0.9993
Specific gravity of soil at 20°c.	2.69	2.63
Average specific gravity of soil .	2.66	

c) Atterberg limit

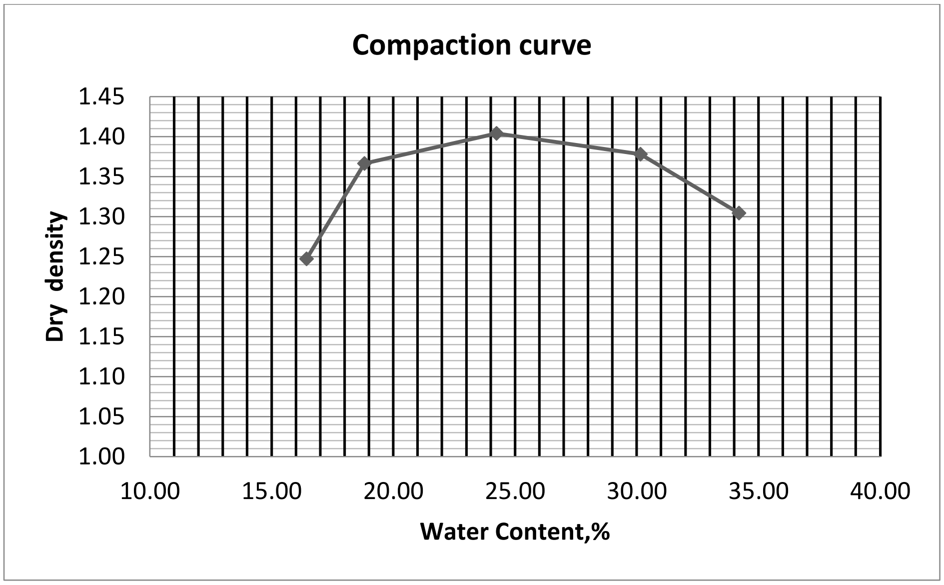
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	L-2	T-2	T-4	T-3	TH	T-6
Mass of container, g	15.8	15.9	15.7	15.6	16	15
Mass of container + Wet soil, g	41.10	40.20	40.50	40.60	22.60	20.80
Mass of container + Dry soil, g	33.30	32.60	32.60	32.60	20.80	19.50
Mass of water, g	7.80	7.60	7.90	8.00	1.80	1.30
Mass of dry soil, g	17.50	16.70	16.90	17.00	4.80	4.50
Water content, %	44.57	45.51	46.75	47.06	37.50	28.89
No of blows	32	28	23	19	------	------
Liquid Limit, % = 47	Plastic Limit, % =33				PI, % =14	


47.50
47.00
46.50
46.00
45.50
45.00
44.50
44.00
Water content,%
10
100
No. of bows

d) **Compaction test**

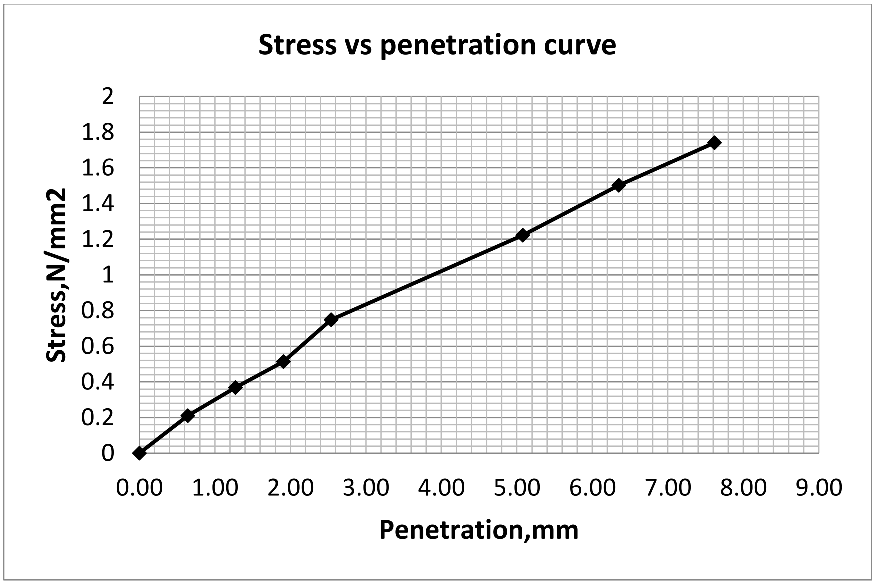
Moisture content Vs dry density computation table					
Trial No.	1	2	3	4	5
Mass of Mold, g	3137.4	3137.4	3137.4	3137.4	3137.4
Mass of mold + Compacted Soil, g	4508.2	4670.3	4784.2	4831	4789.8
Mass of Compacted soil, g	1370.8	1532.9	1646.8	1693.6	1652.4
Volume of Mold, cm3	944	944	944	944	944
Bulk density, g/cm3	1.45	1.62	1.74	1.79	1.75
Water Contenet, %	16.43	18.82	24.24	30.16	34.19
Dry density, g/cm3	1.25	1.37	1.40	1.38	1.30

Water Content					
Conatainer No	T-2	6D	PL	T-6	MA
Mass of container, g	15.6	15.6	15.4	15.6	15.7
Mass of container + wet soil, g	40.4	49.7	40	40.2	57.3
Mass of ontainer + Dry soil, g	36.9	44.3	35.2	34.5	46.7
Mass of Water, g	3.5	5.4	4.8	5.7	10.6
Mass of Dry soil, g	21.3	28.7	19.8	18.9	31
Water content, %	16.43	18.82	24.24	30.16	34.19
Dry Unit Weight, g/cm3	1.25	1.37	1.40	1.38	1.30



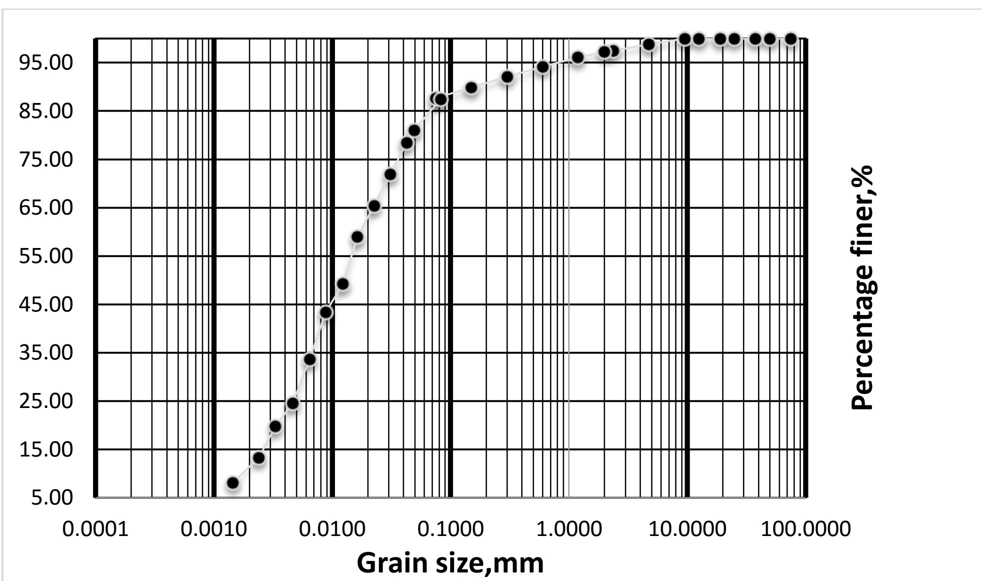
h) California bearing ratio

Penet.	Ring Reading	load	Stress	Standard stress	CBR
(mm)	(Div.)	N	N/mm2	(N/mm^2)	%
0.00	0.0	0	0		
0.64	15.8	406.1706	0.209907		
1.27	27.7	712.0839	0.368002		
1.91	38.6	992.2902	0.512811		
2.54	56.4	1449.875	0.749289	6.9	10.85927
5.08	92.0	2365.044	1.222245	10.3	11.86646
6.35	113.0	2904.891	1.501236		
7.62	131.0	3367.617	1.740371		



5) Sample 5

a) Grain size analysis

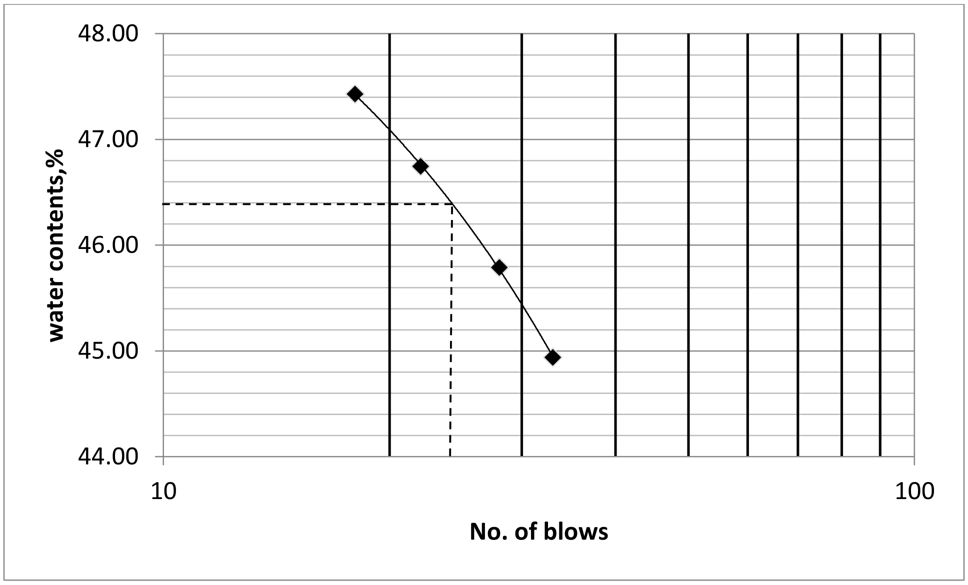


b) Specific gravity

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	178.4	163.2
Temperature, T_x(°c)	21.4	23.6
Weight of pycnometer + water at T_x , W_{pw}(atT_x) (g)	162.80	147.50
Weight of dry soil , w_s (gm)	25	25
Conversion factor , K	0.9998	0.9991
Specific gravity of soil at 20°c.	2.66	2.69
Average specific gravity of soil .	2.67	

c) Atterberg limit

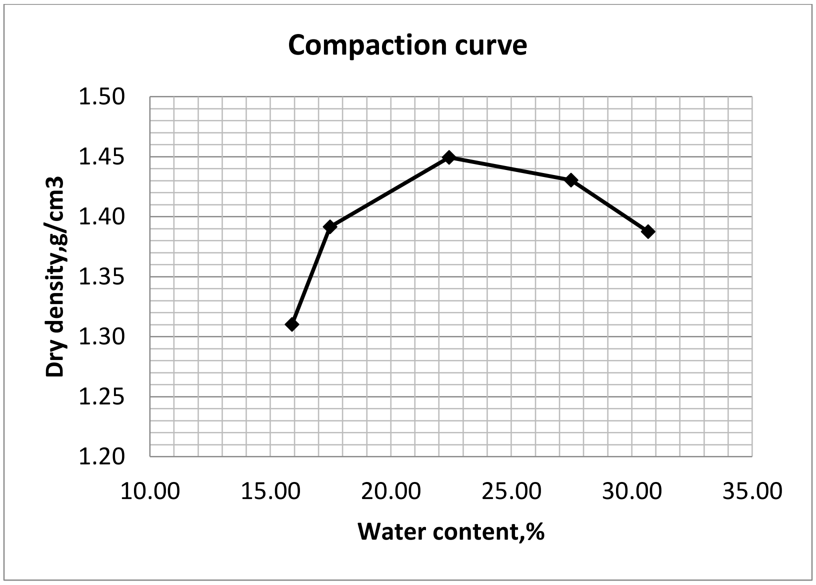
	Liquid Limit					Plastic Limit	
Trial No	1	2	3	4		1	2
Container No	WI	A-1	WT2	W-3		T-2	L1
Mass of container, g	15.7	15.4	15.7	15.8		15.7	15.5
Mass of container + Wet soil, g	40.05	43.10	40.50	41.60		23.00	21.90
Mass of container + Dry soil, g	32.50	34.40	32.60	33.30		21.30	20.60
Mass of water, g	7.55	8.70	7.90	8.30		1.70	1.30
Mass of dry soil, g	16.80	19.00	16.90	17.50		5.60	5.10
Water content, %	44.94	45.79	46.75	47.43		30.36	25.49
No of blows	33	28	22	18		------	------
Liquid Limit, % = 46			Plastic Limit, % = 28			PI, % =18	


48.00
47.00
46.00
45.00
44.00
water contents,%
10
100
No. of blows

d) Compaction test

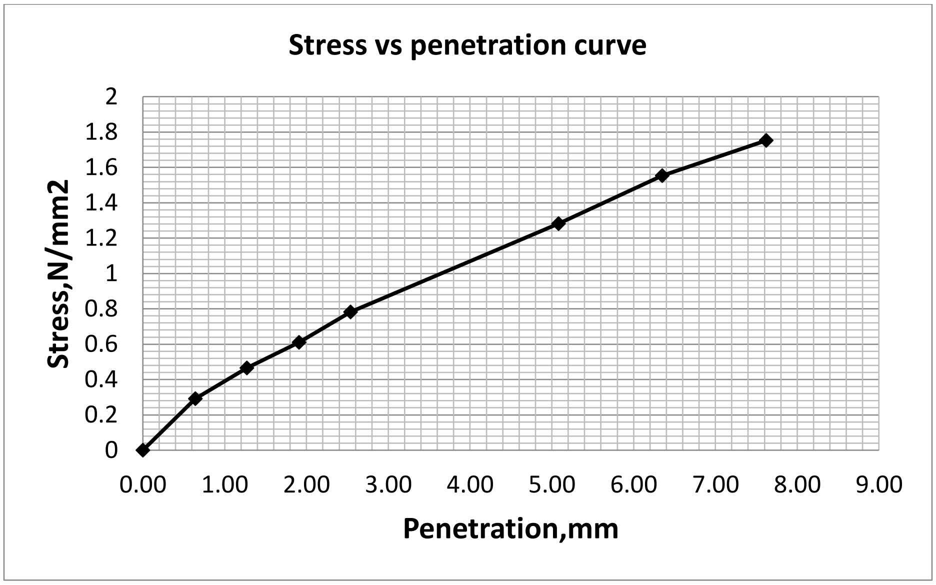
Moisture content Vs dry density computation table

Trial No.		1	2	3	4	5
Mass of Mold, g		3137.4	3137.4	3137.4	3137.4	3137.4
Mass of mold + Compacted Soil, g		4570.9	4680.6	4812.3	4858.8	4849.2
Mass of Compacted soil, g		1433.5	1543.2	1674.9	1721.4	1711.8
Volume of Mold, cm3		944	944	944	944	944
Bulk density, g/cm3		1.52	1.63	1.77	1.82	1.81
Water Contenet, %		15.90	17.48	22.41	27.48	30.67
Dry density, g/cm3		1.31	1.39	1.45	1.43	1.39
Water Content						
Conatainer No		G1	T-2	T-5	TH	W-3
Mass of container, g		15.5	15	15.8	15.8	15.6
Mass of container + wet soil, g		48.3	39.2	44.2	49.2	58.2
Mass of ontainer + Dry soil, g		43.8	35.6	39	42	48.2
Mass of Water, g		4.5	3.6	5.2	7.2	10
Mass of Dry soil, g		28.3	20.6	23.2	26.2	32.6
Water content, %		15.90	17.48	22.41	27.48	30.67
Dry Unit Weight, g/cm3		1.31	1.39	1.45	1.43	1.39



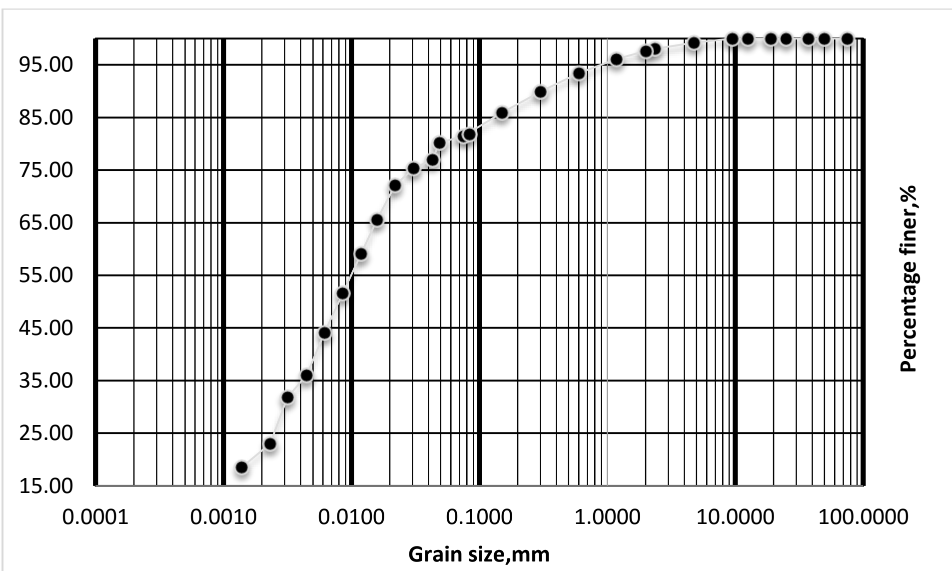
i) California bearing ratio

Penet.	Ring Reading	load	Stress	Standard stress	CBR
(mm)	(Div.)	N	N/mm2	(N/mm^2)	%
0.00	0.0	0	0		
0.64	22.0	565.554	0.292276		
1.27	35.0	899.745	0.464984		
1.91	45.9	1179.951	0.609794		
2.54	58.9	1514.142	0.782502	6.9	11.34062
5.08	96.5	2480.726	1.282029	10.3	12.44688
6.35	117.0	3007.719	1.554377		
7.62	132.0	3393.324	1.753656		



6**) Sample 6**

a) Grain size analysis

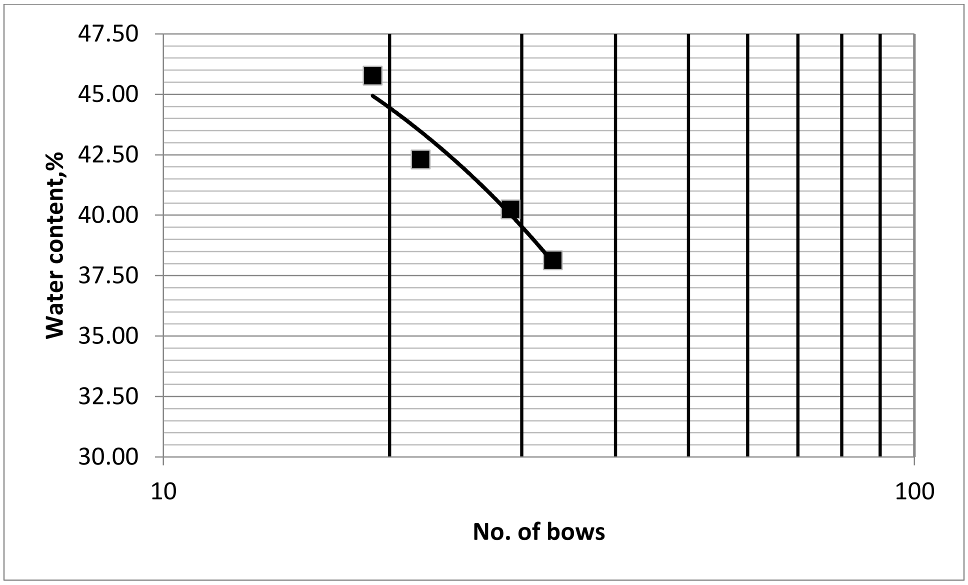


b) Specific gravity

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	178.3	178.5
Temperature, T_x(°c)	23	24
Weight of pycnometer + water at T_x , W_{pw}(atT_x) (g)	162.70	163.00
Weight of dry soil , w_s (gm)	25	25
Conversion factor , K	0.9993	0.9991
Specific gravity of soil at 20°c.	2.66	2.63
Average specific gravity of soil .	2.64	

c) Atterberg limit

	Liquid Limit					Plastic Limit	
Trial No	1	2	3	4		1	2
Container No	3D	W-3-P	MT9	3T		A-1	6
Mass of container, g	15.6	15.6	15.8	15.6		15.5	15.4
Mass of container + Wet soil, g	42.40	38.60	45.40	39.80		22.70	22.50
Mass of container + Dry soil, g	35.00	32.00	36.60	32.20		21.10	20.90
Mass of water, g	7.40	6.60	8.80	7.60		1.60	1.60
Mass of dry soil, g	19.40	16.40	20.80	16.60		5.60	5.50
Water content, %	38.14	40.24	42.31	45.78		28.57	29.09
No of blows	33	29	22	19		------	------
Liquid Limit, % =41			Plastic Limit, % = 29				PI, % =12

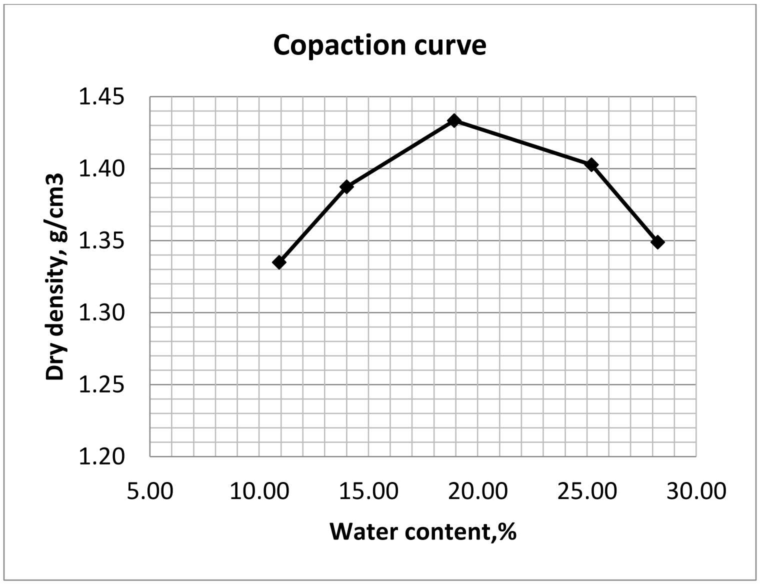


47.50
45.00
42.50
40.00
37.50
35.00
32.50
30.00
Water content,%
10
100
No. of bows

d) **Compaction test**

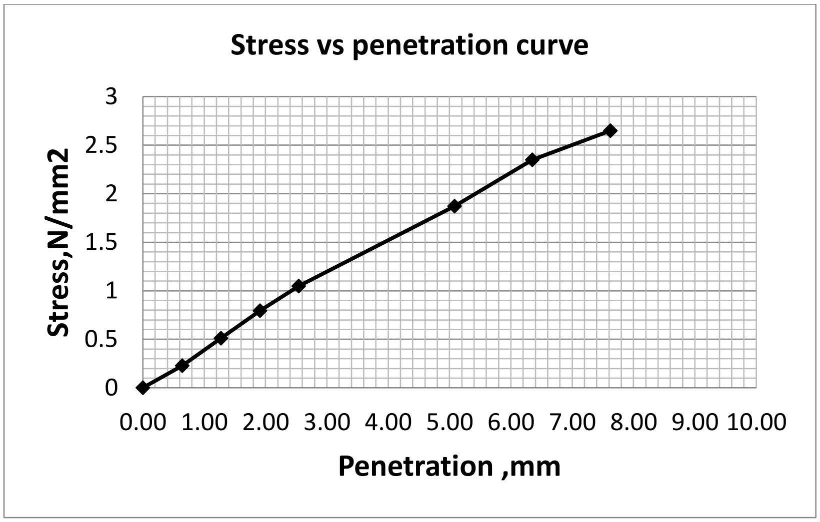
Moisture content Vs dry density computation table					
Trial No.	1	2	3	4	5
Mass of Mold, g	3137.4	3137.4	3137.4	3137.4	3137.4
Mass of mold + Compacted Soil, g	4535.2	4630.4	4746.5	4795.2	4770.5
Mass of Compacted soil, g	1397.8	1493	1609.1	1657.8	1633.1
Volume of Mold, cm3	944	944	944	944	944
Bulk density, g/cm3	1.48	1.58	1.70	1.76	1.73
Water Contenet, %	10.92	14.00	18.92	25.20	28.24
Dry density, g/cm3	1.33	1.39	1.43	1.40	1.35

Water Content					
Conatainer No	G2	F-3	TH	T-1	WP
Mass of container, g	15.6	15.8	15.2	15.8	15.6
Mass of container + wet soil, g	34.9	38.6	41.6	47.1	48.3
Mass of ontainer + Dry soil, g	33	35.8	37.4	40.8	41.1
Mass of Water, g	1.9	2.8	4.2	6.3	7.2
Mass of Dry soil, g	17.4	20	22.2	25	25.5
Water content, %	10.92	14.00	18.92	25.20	28.24
Dry Unit Weight, g/cm3	1.33	1.39	1.43	1.40	1.35



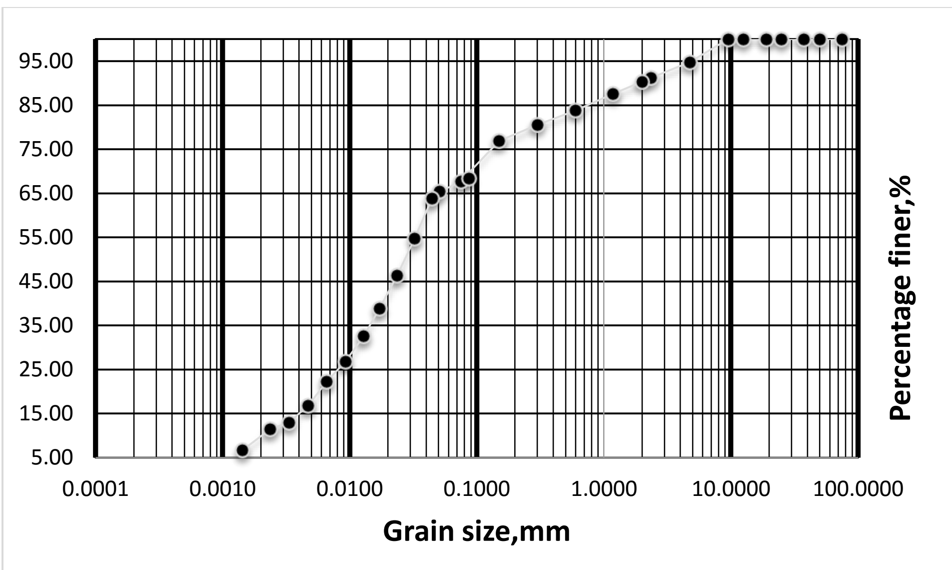
j) California bearing ratio

Penet.	Ring Reading	load	Stress	Standard stress	CBR
(mm)	(Div.)	N	N/mm2	(N/mm^2)	%
0.00	0.0	0	0		
0.64	17.1	439.5897	0.227178		
1.27	38.5	989.7195	0.511483		
1.91	59.7	1534.708	0.793131		
2.54	78.9	2028.282	1.048208	6.9	15.19142
5.08	140.9	3622.116	1.871895	10.3	18.17374
6.35	176.9	4547.568	2.350164		
7.62	199.5	5128.547	2.650412		



7) **Sample 7**

a) Grain size analysis



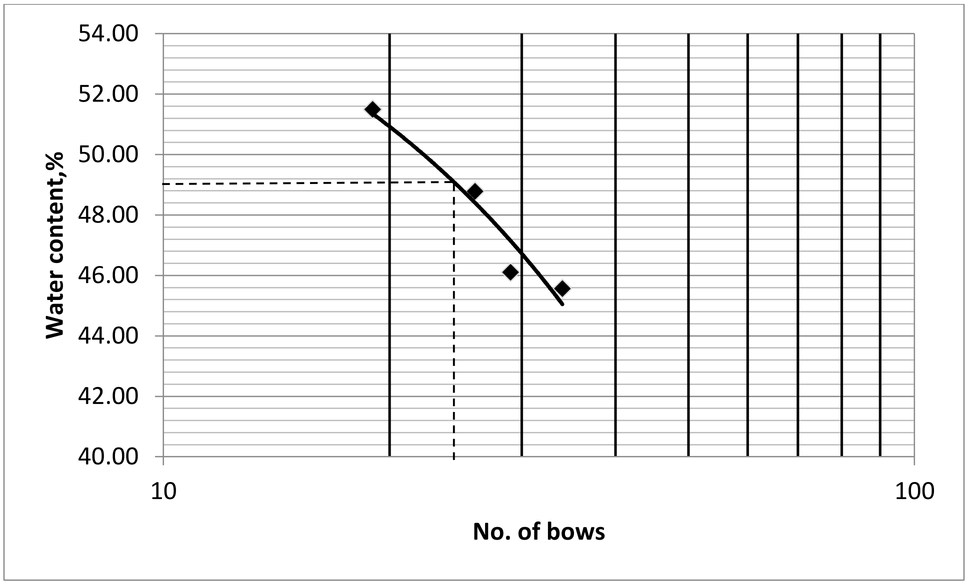
b) Specific gravity

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	178.6	162.7
Temperature, T_x(°c)	20	20
Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g)	163.00	147.20
Weight of dry soil , w_s (gm)	25	25
Conversion factor , K	1	1.0000
Specific gravity of soil at 20°c.	2.66	2.63
Average specific gravity of soil .	2.65	

c) Atterberg limit

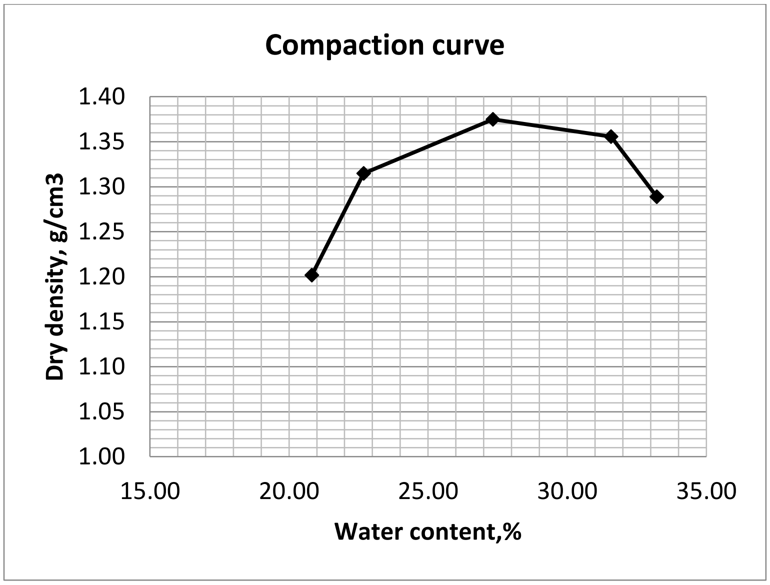
	Liquid Limit					Plastic Limit	
Trial No	1	2	3	4		1	2
Container No	MT	XZ	T-9	T-8		MT1	MT2
Mass of container, g	15.8	15.4	15.7	15.7		15.6	15.2
Mass of container + Wet soil, g	40.30	36.00	46.20	41.00		23.10	22.90
Mass of container + Dry soil, g	32.70	29.50	36.20	32.40		21.20	20.90
Mass of water, g	7.60	6.50	10.00	8.60		1.90	2.00
Mass of dry soil, g	16.90	14.10	20.50	16.70		5.60	5.70
Water content, %	44.97	46.10	48.78	51.50		33.93	35.09
No of blows	34	29	26	19		------	------

Liquid Limit, % =	49	Plastic Limit, % =	35		PI, % =	14



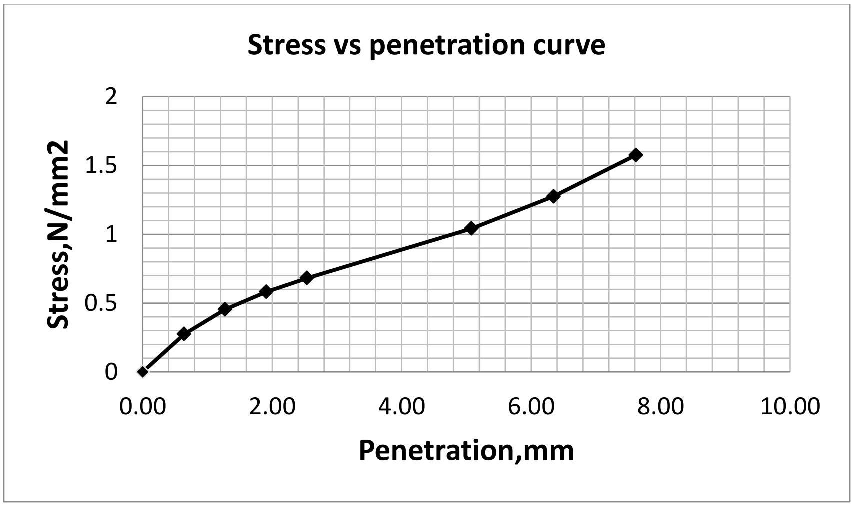
d) **Compaction test**

Moisture content Vs dry density computation table					
Trial No.	1	2	3	4	5
Mass of Mold, g	3137.4	3137.4	3137.4	3137.4	3137.4
Mass of mold + Compacted Soil, g	4508.1	4660	4790	4821.5	4758.3
Mass of Compacted soil, g	1370.7	1522.6	1652.6	1684.1	1620.9
Volume of Mold, cm3	944	944	944	944	944
Bulk density, g/cm3	1.45	1.61	1.75	1.78	1.72
Water Contenet, %	20.82	22.69	27.34	31.58	33.22
Dry density, g/cm3	1.20	1.31	1.37	1.36	1.29
Water Content					
Conatainer No	T-5	TH	MT	3-G	T-3
Mass of container, g	15.4	15.6	15.2	15	15.6
Mass of container + wet soil, g	56.6	56.7	52	55	54.9
Mass of ontainer + Dry soil, g	49.5	49.1	44.1	45.4	45.1
Mass of Water, g	7.1	7.6	7.9	9.6	9.8
Mass of Dry soil, g	34.1	33.5	28.9	30.4	29.5
Water content, %	20.82	22.69	27.34	31.58	33.22
Dry Unit Weight, g/cm3	1.20	1.31	1.37	1.36	1.29



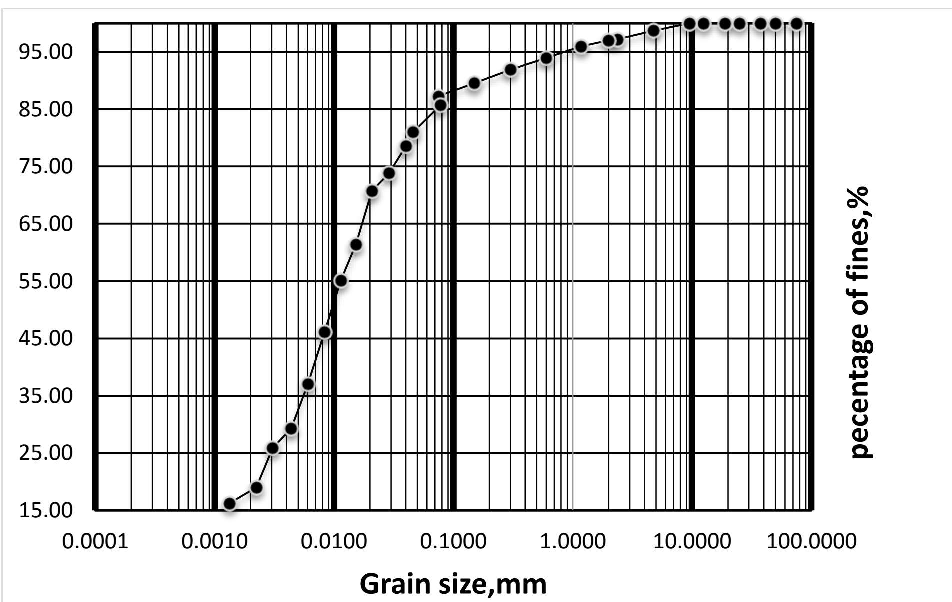
e) California bearing ratio

Penet.	Ring Reading	load	Stress	Standard stress	CBR
(mm)	(Div.)	N	N/mm2	(N/mm^2)	%
0.00	0.0	0	0		
0.64	20.7	532.1349	0.275005		
1.27	34.2	879.1794	0.454356		
1.91	43.8	1125.967	0.581895		
2.54	51.3	1318.769	0.681534	6.9	9.87731
5.08	78.4	2015.429	1.041565	10.3	10.11228
6.35	96.0	2467.872	1.275386		
7.62	118.6	3048.85	1.575633		



8**) Sample 8**

a) Grain size analysis

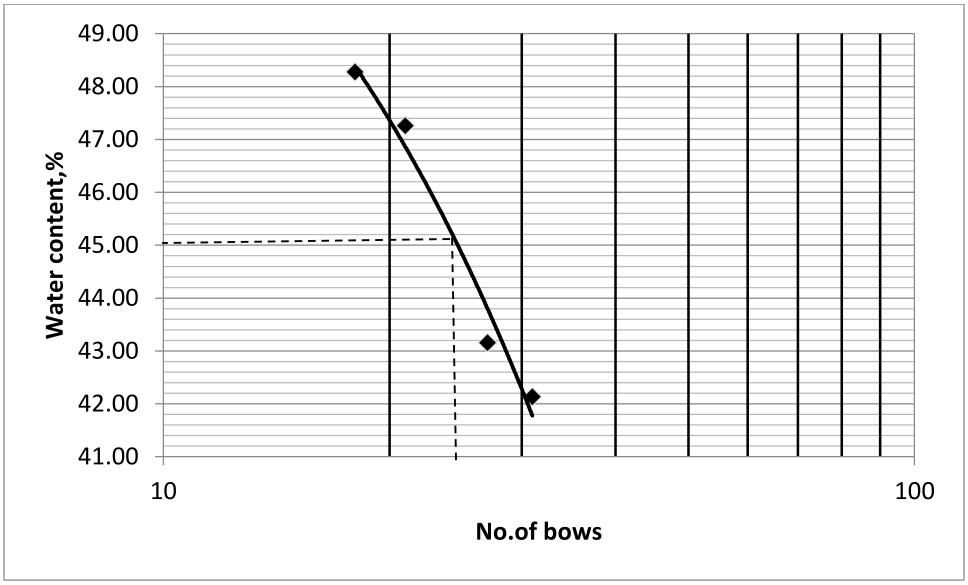


b) Specific gravity

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	178.4	178.5
Temperature, T_x(°c)	22.1	22
Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g)	162.70	163.20
Weight of dry soil , w_s (gm)	25	25
Conversion factor , K	0.9996	0.9998
Specific gravity of soil at 20°c.	2.69	2.58
Average specific gravity of soil .	2.63	

c) Atterberg limit

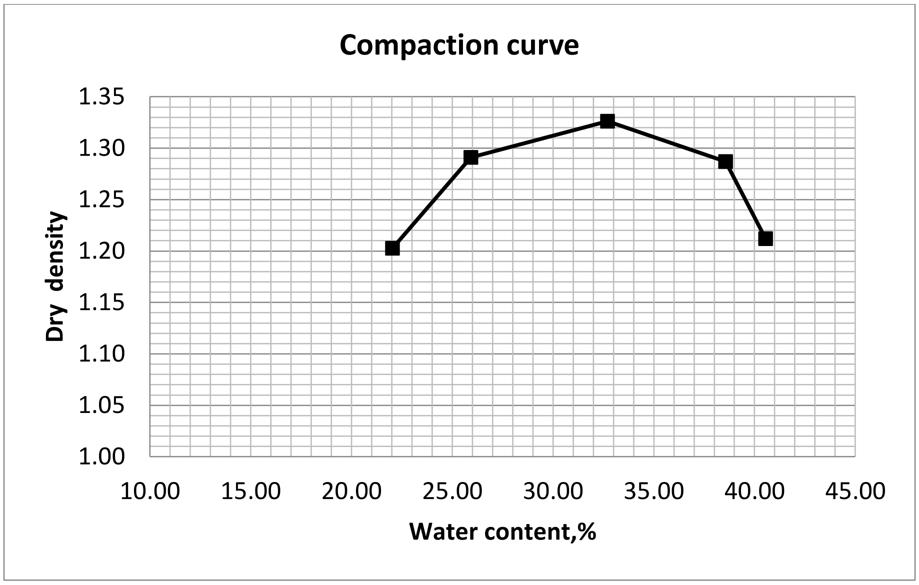
	Liquid Limit					Plastic Limit	
Trial No	1	2	3	4		1	2
Container No	B1	G2	T-5	WT		G3	G1
Mass of container, g	15.5	15.6	15	15		15.8	15.7
Mass of container + Wet soil, g	43.50	42.80	36.50	40.80		20.80	21.00
Mass of container + Dry soil, g	35.20	34.60	29.60	32.40		19.60	19.80
Mass of water, g	8.30	8.20	6.90	8.40		1.20	1.20
Mass of dry soil, g	19.70	19.00	14.60	17.40		3.80	4.10
Water content, %	42.13	43.16	47.26	48.28		31.58	29.27
No of blows	31	27	21	18		------	------
Liquid Limit, % = 45		Plastic Limit, % =30				PI, % =14	



d) **Compaction test**

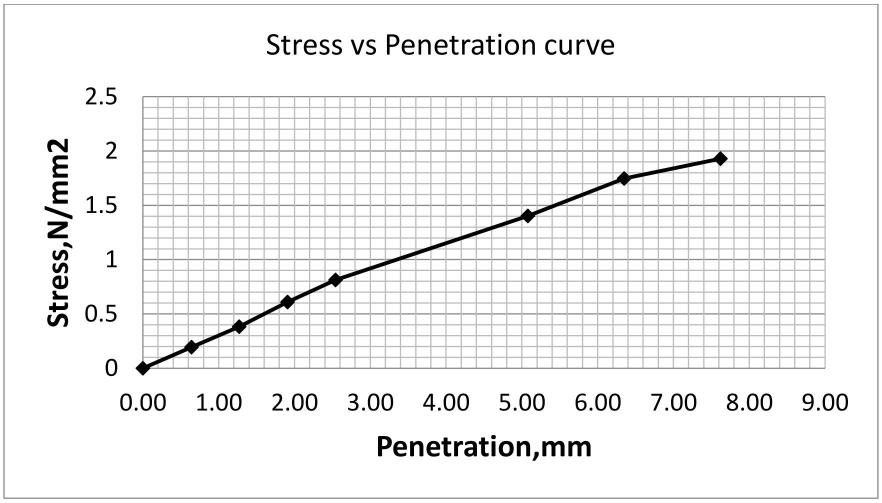
Moisture content Vs dry density computation table					
Trial No.	1	2	3	4	5
Mass of Mold, g	3137.4	3137.4	3137.4	3137	3137.4
Mass of mold + Compacted Soil, g	4523.1	4672.2	4798.6	4821	4745.2
Mass of Compacted soil, g	1385.7	1534.8	1661.2	1683	1607.8
Volume of Mold, cm3	944	944	944	944	944
Bulk density, g/cm3	1.47	1.63	1.76	1.78	1.70
Water Contenet, %	22.04	25.93	32.70	38.56	40.54
Dry density, g/cm3	1.20	1.29	1.33	1.29	1.21

Water Content					
Conatainer No	T-1	T-4	3-F	W-1	T-3
Mass of container, g	15.8	15.6	15.6	15.6	15
Mass of container + wet soil, g	60.1	49.6	57.8	58	67
Mass of ontainer + Dry soil, g	52.1	42.6	47.4	46.2	52
Mass of Water, g	8	7	10.4	11.8	15
Mass of Dry soil, g	36.3	27	31.8	30.6	37
Water content, %	22.04	25.93	32.70	38.56	40.54
Dry Unit Weight, g/cm3	1.20	1.29	1.33	1.29	1.21



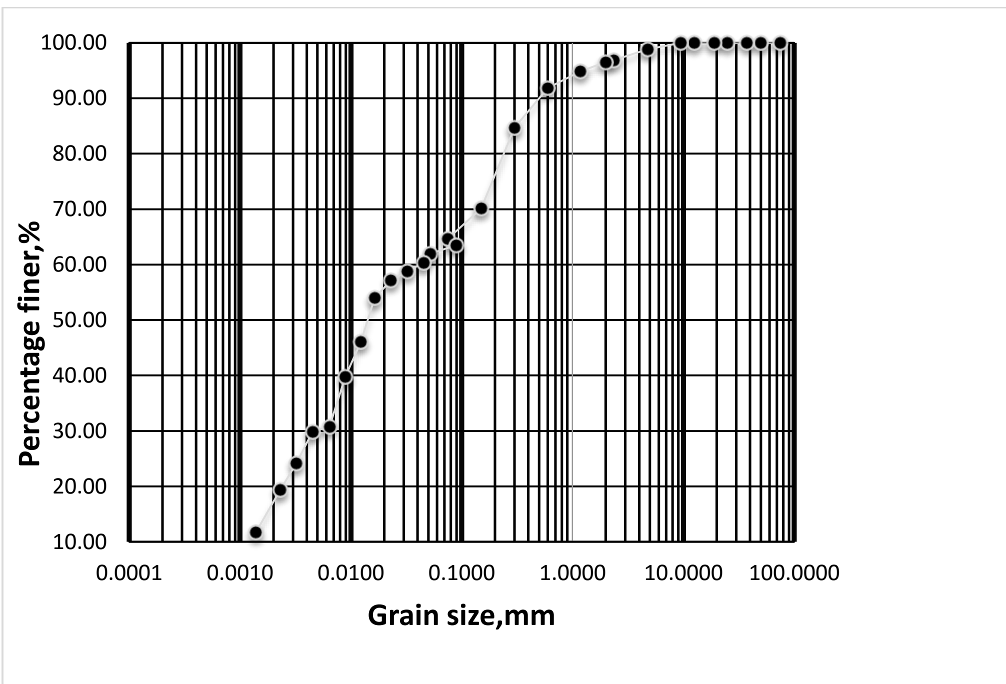
f) California bearing ratio

Penet.	Ring Reading	load	Stress	Standard stress	CBR
(mm)	(Div.)	N	N/mm2	(N/mm^2)	%
0.00	0.0	0	0		
0.64	14.7	377.8929	0.195293		
1.27	28.7	737.7909	0.381287		
1.91	45.9	1179.951	0.609794		
2.54	61.2	1573.268	0.813059	6.9	11.78346
5.08	105.5	2712.089	1.401596	10.3	13.60773
6.35	131.5	3380.471	1.747013		
7.62	145.3	3735.227	1.93035		



9) **Sample 9**

a) Grain size analysis

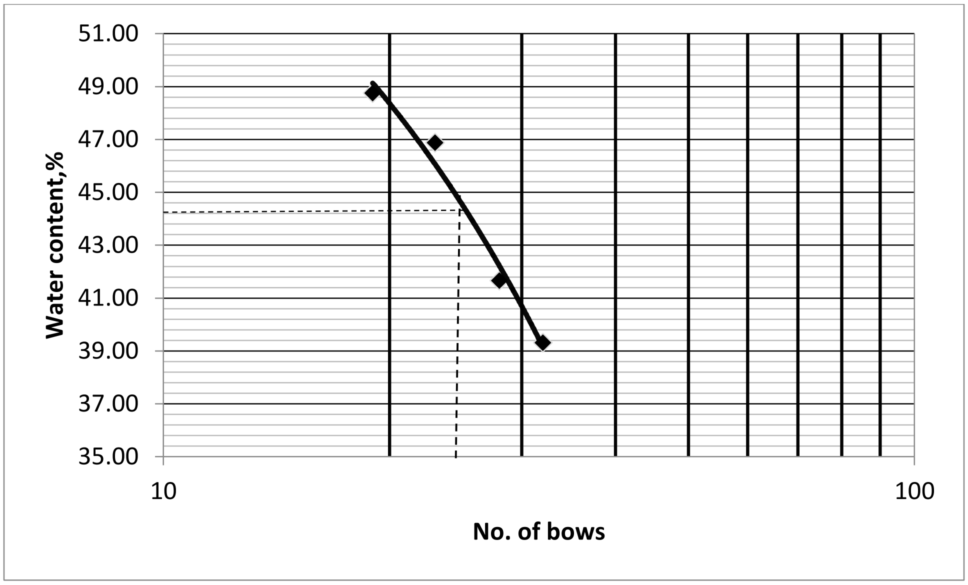


b) Specific gravity

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	178.1	161.2
Temperature, T_x(°c)	22.9	23.9
Weight of pycnometer + water at T_x , W_{pw}(atT_x) (g)	162.60	146.90
Weight of dry soil , w_s (gm)	25	25
Conversion factor , K	0.9993	0.9991
Specific gravity of soil at 20°c.	2.63	2.74
Average specific gravity of soil .	2.69	

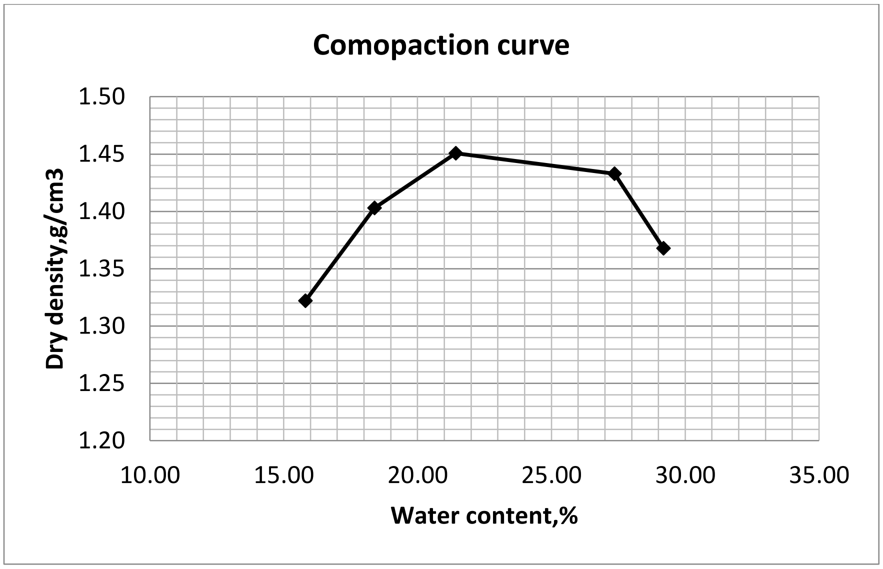
c) Atterberg limit

	Liquid Limit					Plastic Limit	
Trial No	1	2	3	4		1	2
Container No	L1	L3	L7	T-2		L4	L5
Mass of container, g	15	15.8	15.8	15.2		15.8	15.8
Mass of container + Wet soil, g	35.20	43.00	44.00	39.00		22.50	23.80
Mass of container + Dry soil, g	29.50	35.00	35.00	31.20		21.00	22.00
Mass of water, g	5.70	8.00	9.00	7.80		1.50	1.80
Mass of dry soil, g	14.50	19.20	19.20	16.00		5.20	6.20
Water content, %	39.31	41.67	46.88	48.75		28.85	29.03
No of blows	32	28	23	19		------	------
Liquid Limit, % =44		Plastic Limit, % =29				PI, % =15	



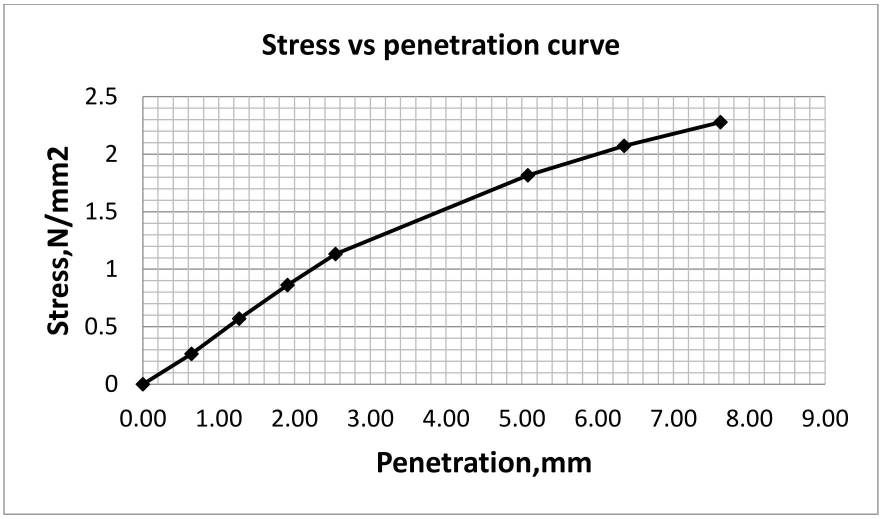
d) **Compaction test**

Moisture content Vs dry density computation table						
Trial No.		1	2	3	4	5
Mass of Mold, g		3137.4	3137.4	3137.4	3137.4	3137.4
Mass of mold + Compacted Soil, g		4582.9	4705.5	4800.5	4860	4805.5
Mass of Compacted soil, g		1445.5	1568.1	1663.1	1722.6	1668.1
Volume of Mold, cm3		944	944	944	944	944
Bulk density, g/cm3		1.53	1.66	1.76	1.82	1.77
Water Contenet, %		15.82	18.40	21.43	27.35	29.18
Dry density, g/cm3		1.32	1.40	1.45	1.43	1.37
Water Content						
Conatainer No		A-2	3-C	F-2	W3	F-3
Mass of container, g		15.6	15.8	15	15.8	15.4
Mass of container + wet soil, g		38.3	40.9	42.2	44.2	45.5
Mass of ontainer + Dry soil, g		35.2	37	37.4	38.1	38.7
Mass of Water, g		3.1	3.9	4.8	6.1	6.8
Mass of Dry soil, g		19.6	21.2	22.4	22.3	23.3
Water content, %		15.82	18.40	21.43	27.35	29.18
Dry Unit Weight, g/cm3		1.32	1.40	1.45	1.43	1.37



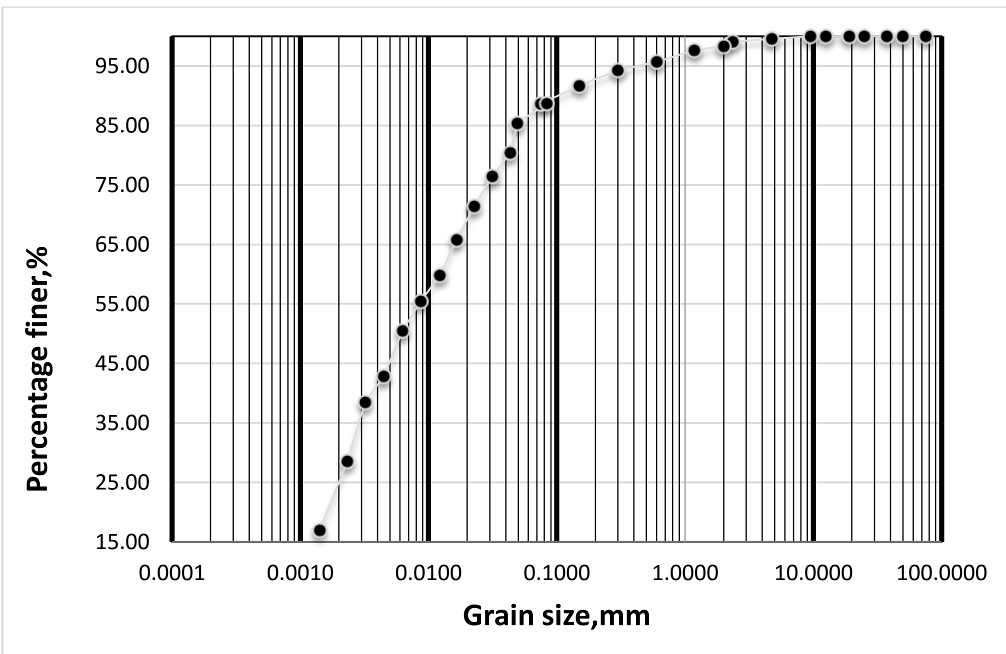
g) California bearing ratio

Penet.	Ring Reading	load	Stress	Standard stress	CBR
(mm)	(Div.)	N	N/mm2	(N/mm^2)	%
0.00	0.0	0	0		
0.64	19.8	508.9986	0.263048		
1.27	42.9	1102.83	0.569938		
1.91	64.8	1665.814	0.860886		
2.54	85.2	2190.236	1.131905	6.9	16.40442
5.08	136.7	3514.147	1.816097	10.3	17.63201
6.35	156.0	4010.292	2.072502		
7.62	171.5	4408.751	2.278424		



10**) Sample 10**

a) Grain size analysis

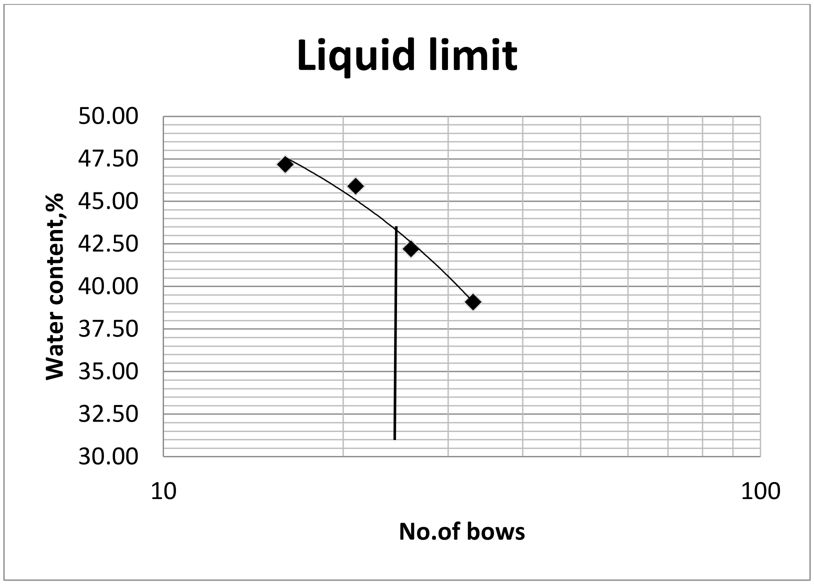


b) Specific gravity

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	178.2	177.9
Temperature, T_x(^{o}c)	23.6	24.6
Weight of pycnometer + water at T_x , W_{pw}(atT_x) (g)	162.80	162.47
Weight of dry soil,w_s (gm)	25	25
Conversion factor , K	0.9991	0.9989
Specific gravity of soil at 20°c.	2.60	2.61
Average specific gravity of soil .	2.60	

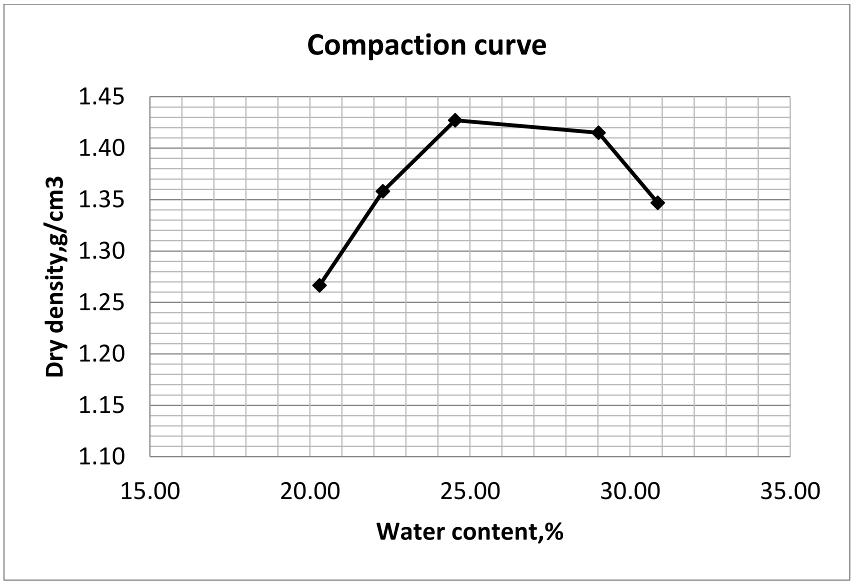
c) Atterberg limit

	Liquid Limit					Plastic Limit	
Trial No	1	2	3	4		1	2
Container No	WT4	WI	T-7	T-6		AO	G7
Mass of container, g	15.8	15.9	15.4	15.7		15.9	15
Mass of container + Wet soil, g	43.20	37.80	45.60	46.90		20.50	23.60
Mass of container + Dry soil, g	35.50	31.30	36.10	36.90		19.70	21.80
Mass of water, g	7.70	6.50	9.50	10.00		0.80	1.80
Mass of dry soil, g	19.70	15.40	20.70	21.20		3.80	6.80
Water content, %	39.09	42.21	45.89	47.17		21.05	26.47
No of blows	33	26	21	16		------	------
Liquid Limit, % = 43			Plastic Limit, % = 24				PI, % =19



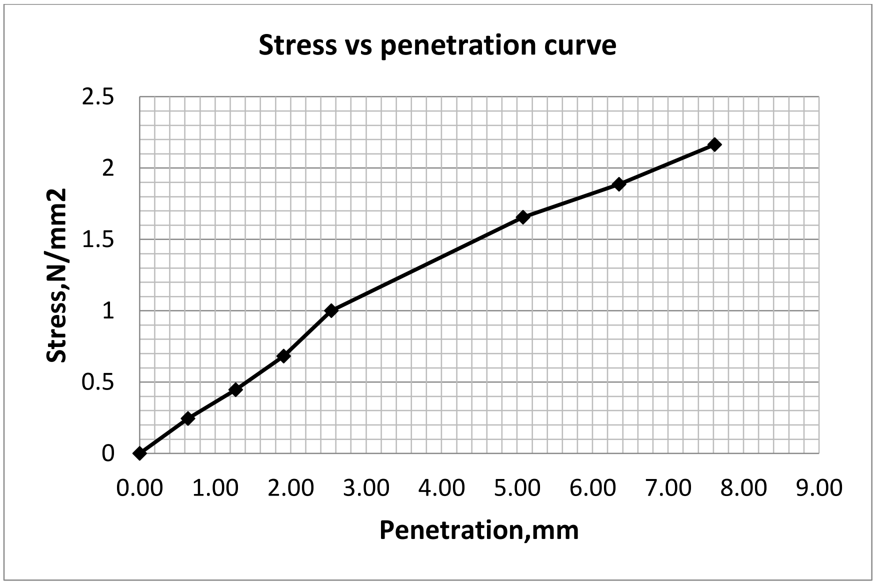
d) **Compaction test**

Moisture content Vs dry density computation table					
Trial No.	1	2	3	4	5
Mass of Mold, g	3137.4	3137.4	3137.4	3137.4	3137.4
Mass of mold + Compacted Soil, g	4575.9	4705	4815	4860.6	4801.5
Mass of Compacted soil, g	1438.5	1567.6	1677.6	1723.2	1664.1
Volume of Mold, cm3	944	944	944	944	944
Bulk density, g/cm3	1.52	1.66	1.78	1.83	1.76
Water Contenet, %	20.30	22.27	24.54	29.02	30.87
Dry density, g/cm3	1.27	1.36	1.43	1.41	1.35
Water Content					
Conatainer No	A-1	A-3	F-4	3-B	T-1
Mass of container, g	15.6	15.2	15.2	15.8	15.4
Mass of container + wet soil, g	39.9	41	42.1	44.7	45.5
Mass of ontainer + Dry soil, g	35.8	36.3	36.8	38.2	38.4
Mass of Water, g	4.1	4.7	5.3	6.5	7.1
Mass of Dry soil, g	20.2	21.1	21.6	22.4	23
Water content, %	20.30	22.27	24.54	29.02	30.87
Dry Unit Weight, g/cm3	1.27	1.36	1.43	1.41	1.35



e) California bearing ratio

Penet.	Ring Reading	load	Stress	Standard stress	CBR
(mm)	(Div.)	N	N/mm2	(N/mm^2)	%
0.00	0.0	0	0		
0.64	18.4	473.0088	0.244449		
1.27	33.7	866.3259	0.447714		
1.91	51.3	1318.769	0.681534		
2.54	75.3	1935.737	1.000381	6.9	14.49827
5.08	124.7	3205.663	1.656673	10.3	16.08421
6.35	142.0	3650.394	1.886509		
7.62	163.0	4190.241	2.165499		


Stress vs penetration curve
Stress,N/mm2
Penetration,mm
2.5
2
1.5
1
0.5
0
0.00
1.00
2.00
3.00
4.00
5.00
6.00
7.00
8.00
9.00

APPENDEX B

B. Secondary data

1.Transportation Construction Design

Sample code	P200	Atterberg limit			AASHTO Classification	Compaction test		CBR(%)
		LL	PL	PI		OMC	MDD	
WB1	51	33	24	9	A-4	21	1.21	15
WB2	70	31	21	10	A-4	26	1.28	10
WB3	73	37	28	9	A-4	31	1.6	8
WB4	72	45	25	20	A-7-6	27	1.33	4
WB5	55	32	29	3	A-4	24	1.69	14
WB6	45	34	25	9	A-4	22	1.58	12
WB7	90	50	31	19	A-7-5	33	1.36	7
WB8	77	40	28	12	A-7-6	26	1.31	6
WB9	77	39	26	13	A-6	28	1.43	12
WB10	83	41	25	16	A-7-6	29	1.23	10
WB11	77	49	30	19	A-7-5	32	1.31	6.4
WB12	95	44	21	23	A-7-6	31	1.56	9.5
WB13	41	35	26	9	A-4	18	1.78	17
WB14	49	39	30	9	A-4	18	1.72	13
WB15	58	41	30	11	A-7-5	26	1.64	11.5
WB16	47	31	24	7	A-4	20	1.82	19
WB17	68	35	27	8	A-7	19	1.78	17
WB18	47	36	26	10	A-4	23	1.75	16
WB19	77	46	34	12	A-7-5	31	1.44	9
WB20	67	42	32	10	A-5	21	1.62	11
WB21	72	42	31	11	A-7-5	32	1.6	10

WB22	68	39	30	9	A-4	23	1.73	13.5
WB23	65	35	24	11	A-6	25	1.77	16.8
WB24	57	38	29	9	A-4	25	1.7	14
WB25	47	33	26	7	A-4	25	1.8	18
WB26	84	33	22	11	A-6	22	1.81	18.4
WB27	45	42	30	12	A-7-5	26	1.61	11
WB28	88	46	34	12	A-7-5	29	1.42	9
WB29	56	37	23	14	A-6	22	1.79	15
WB30	42	39	23	16	A-6	25	1.68	13
WB31	63	35	26	9	A-4	18	1.76	17
WB32	49	39	30	9	A-4	24	1.7	13.5
WB33	58	41	30	11	A-7-5	26	1.63	11
WB34	47	31	24	7	A-4	23	1.83	19
WB35	68	35	27	8	A-7	20	1.76	16.5
WB36	47	36	26	10	A-4	19	1.73	16
WB37	77	46	34	12	A-7-5	31	1.41	9
WB38	67	42	32	10	A-5	31	1.6	9
WB39	72	42	31	11	A-7-5	30	1.62	10
WB40	68	39	30	9	A-4	23	1.71	12.5
WB41	65	35	24	11	A-6	25	1.76	16
WB42	57	38	29	9	A-4	25	1.68	13.8
WB43	84	33	22	11	A-6	21	1.79	18
WB44	45	42	30	12	A-7-5	26	1.62	12
WB45	88	46	34	12	A-7-5	29	1.42	9.2
WB46	56	37	23	14	A-6	22	1.78	14.8
WB47	42	39	23	16	A-6	25	1.69	13.5
TW48	41	41	28	13	A-7-6	20	1.63	12.5
TW49	42	31	26	5	A-4	19	1.83	20
TW50	59	40	29	11	A-6	28	1.65	13

TW51	71	43	32	11	A-7-5	29	1.63	11
TW52	63	35	24	11	A-6	26	1.76	16
TW53	61	38	28	10	A-4	23	1.69	14.5
TW54	77	45	30	15	A-7-5	24	1.48	10
TW55	81	43	27	16	A-7-6	29	1.58	11
TW56	93	49	35	14	A-7-5	30	1.3	6.7
TW57	46	39	26	13	A-6	17	1.7	13
TW58	55	39	29	10	A-4	22	1.68	12.8
TW59	62	34	20	14	A-7-6	26	1.78	17
TW60	43	39	23	16	A-6	27	1.65	12.6
TW61	64	35	19	16	A-6	23	1.78	16.2
TW62	59	36	26	10	A-4	24	1.69	16.8
TW63	48	38	30	8	A-4	24	1.6	14.2
TW64	63	38	31	7	A-4	20	1.42	14
TW65	73	42	30	12	A-7-5	28	1.32	12.5
TW66	64	39	30	9	A-4	20	1.49	13
TW67	72	36	23	13	A-6	21	1.39	17
TW68	71	32	21	11	A-6	22	1.72	21
TW69	45	35	27	8	A-4	22	1.7	19
AA70	41	39	24	15	A-6	22	1.67	13
AA71	41	45	28	17	A-7-6	15	1.4	9
AA72	63	42	29	13	A-7-6	20	1.66	12
AA73	65	43	24	19	A-7-6	29	1.44	11
AA74	94	44	28	16	A-7-6	23	1.47	10
AA75	63	37	25	12	A-6	18	1.53	15
AA76	89	44	31	13	A-7-5	30	1.36	10
AA77	58	35	22	13	A-6	19	1.45	12

Where WB= Wolenchiti-bofa,

2. **Best Consulting Engineers**

Sample code	P200	Atterberg limit			AASHTO Classification	Compaction test		CBR(%)
		LL	PL	PI		OMC	MDD	
SG1	97.1	38	26	12	A-6	26.8	1.42	7
SG2	65.5	45	31	14	A-7-5	23.52	1.52	5.8
SG3	55	44	28	16	A-4	18.9	1.55	10
SG4	65.2	48	33	15	A-7-5	23.37	1.35	8
SG5	56.6	46	33	13	A-7-5	21.45	1.45	9
SG6	50.7	47	32	15	A-7-5	24.5	1.41	8.5
SG7	97.7	49	35	14	A-7-5	32.7	1.3	6
SG8	45.5	49	31	18	A-7-5	26.6	1.32	7
SG9	54.3	48	34	14	A-7-5	27.9	1.33	8.1
SG10	55.2	42	32	10	A-5	22.8	1.63	12
SG11	89.8	47	32	15	A-7-5	23.1	1.42	8.3
SG12	58.1	38	28	10	A-5	16.6	1.72	14
SG13	80.1	48	28	20	A-7-6	23.61	1.34	8
SG14	69.2	39	21	18	A-7-6	17.54	1.71	13
SG15	41.3	38	25	13	A-6	17.94	1.73	14.5
SG16	48.1	36	23	13	A-6	17.7	1.75	16
SG17	61.1	49	37	12	A-7-5	23.16	1.3	6.1
SG18	88.6	48	35	13	A-7-5	28.63	1.33	8.2
SG19	88.5	49	34	15	A-7-5	30.5	1.31	6.5
SG20	78	47	35	12	A-7-5	21.61	1.43	8.4

Where SG= subgrade

3.Addis Abeba City Road Authority

Sample code	P200	Atterberg limit			AASHTO Classification	Compaction test		CBR(%)
		LL	PL	PI		OMC	MDD	
5KM	55.93	47.8	28.4	19.5	A-7-6	17.6	1.69	13.8
MP	44	42.4	27.2	15.3	A-7-6	15	1.87	25
KK	58.5	43	29	14	A-7-6	17	1.71	14
GK	57.41	47.8	28.4	19.5	A-7-6	26	1.34	6.2
HM	72	49.5	24.1	25.4	A-7-6	23.5	1.45	4.8
GK	71.11	49	22	17	A-7-5	22.5	1.57	5.8
JR1	70	48	32	16	A-7-5	22.5	1.6	5.8
JR2	56	46.57	26	20.6	A-7-6	18	1.76	13.5
JR3	72	48	37	11	A-7-5	34	1.33	5.6
KRS	96.1	49	28	21	A-7-6	35	1.29	2.6
AK	46	47	32	13	A-7-5	16	1.67	17
AG1	83	49	33	16	A-7-5	28.3	1.55	4.8
AG2	78	49	31	18	A-7-5	29	1.38	4.6
AM	73	48	28	20	A-7-6	22.3	1.54	5.7
A-18M	82	49	29	20	A-7-6	22.3	1.4	4
KM	58	45	25	20	A-7-6	19	1.72	12
YM	62	46	29	17	A-7-6	21	1.6	9

Where 5KM= 5 Kuter mazria, MP= millinium park, kk=kotebe-kidanmehret, GK=guara-kotebe, HM= hana Mariam, JR=jima terminal quarry