JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDY

JIMMA INSTITUTE OF TECHNOLOGY

DEPARTMENT OF CIVIL ENGINEERING

CHARACTERIZING THE SUITABILITY OF SELECTED SUBGRAD MATERIALS FOR USE AS SUBGRADE MATERIALS IN JIMMA ZONE

BY: SELAMAWIT SHEFERAW (BSc)

A THESIS SUBMITTED TO THE DEPARTMENT OF CIVIL ENGINEERING, JIMMA INSTITUTE OF TECHNOLOGY, JIMMA UNIVERSITY, IN PARTIAL FULFILLMENT OF THE DEGREE OF MASTER OF SCIENCE IN GEOTECHNICAL ENGINEEERING.

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JIMMA, ETHIOPIA

ACKNOWLEDGEMENTS

My first thanks goes to almighty God who has given me good instructors, friends and family who have been part of this research work and who gave me courage and support throughout my study and during my research work.

I am very grateful for my research advisor Dr. Kifle Woldaregay (PhD), and Co advisor Mr Elmer C.Agon who has encouraged, directed, and given me academic advice during the course of the research work.

I would like to acknowledge Jimma University for giving me the educational opportunity and for ERA (Ethiopian road authority) for sponsoring my MSc study.

All my good friends and relatives are acknowledged for direct and indirect support in my study and research work.

Last but not least, my family (my father, my mother, my brother, my sister and my husband) who are always with me, supporting and encouraging my studies and deserve respect and thanks for their help, without them I could not be fruitful.

Abstract

Soil is a primary engineering material for road construction and maintenance. Subgrade soils and materials characterizations are the main parameter for flexible pavement both in the design and construction phases. The roads in Jimma area are frequently affected by different geotechnical problems and this result in failures of roads before its design period. This study focused on to characterize the suitability of selected materials for use as subgrade materials in Jimma area. This study site was already used as subgrade materials. So it has been done this research to approve this quarry sites to be used as suitable subgrade materials. So representative samples were taken from 4 different quarry sites and tests are also conducted at laboratory for the determination of CBR, penetration, moisture content, grain size, Atterberg limit, Permeability and Compaction properties. According to the laboratory test CBR value govern the suitability of subgrade materials hence the result of Seka and Seto site results (i.e. 9.025 & 8.2) greater than 5%, so according to ERA manual this quarry site soil are satisfy suitability property of subgrade soils. Therefore these two quarries site soils are suitable for subgrade materials. Finally I recommend further studies are required to determine the property of the soil that are found in the quarry site around Jimma town.

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List of abbreviations & definitions

AASHTO	American Associations of State Highway and Transportation Officials
ASTM	American Society of Testing and Materials
CBR	California Bearing Ratio
CSA	Central Statistical Agency
DCP	Dynamic Cone Penetration
ERA	Ethiopian Roads Authority
FHWA NHI	Federal Highways Administration, National Highways institute
GI	Group Index
LL	Liquid Limit
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
PI	Plasticity Index
PL	Plastic Limit

1. INTRODUCTION

1.1 General

In a country like Ethiopia which is developing at high growth rate and which needs many construction works in the future, geotechnical investigation on the engineering property of soil is very essential (Dagnachew, 2011). Because these data are very important for civil engineers in preliminary design and in designing foundation, road pavement, and retaining structures, etc for future construction projects in the country.

Some part of the land of Ethiopia is covered with expansive soils and some with non-expansive soil (Hagos, 2006). Big cities like Addis Ababa, Bahir Dar and Mekelle as well as main trunk roads are situated on these soil types and currently different construction activities are taking place in the road and building sector on these soil types (Awoke, 2006).

Jimma is located in southwestern of Ethiopia, west of main Ethiopian rift. Most of Jimma area is covered by volcanic rocks (GSE, 2012). The mean annual rainfall of the area is between 1800 mm to 2300 mm with maximum rainfall between months of June and September. The annual mean temperature of the area is between 15 ^{0}C and 25 ^{0}C. Due to the temperature change and its rainfall, most of the natural soils that are found around Jimma town are predominantly loose.

Failures of roads before their design period are being observed and are greatly affecting the economic growth of the country (ERA, 1996). Proper understanding of the causes of failure of roads may lead to proper remedial measures as well as design of safe and economical roads in the future. Such failures could be overcome by undertaking thorough investigation on the subgrade material and the materials overlaying the subgrade and incorporating it in the design (Hagos, 2006).

Roads performed a very vital role in meeting the strategic developmental requirements and accelerating all round development (Gupta, 2010). Technical progress in planning and road construction technology has kept pace with rapid changes in the field of infrastructural development (Nageshwar, 2008). Jimma economic growth is highly dependent on the agricultural sector. Especially the main crop that found at Jimma is coffee which is their main source of income. Since Jimma is a big market center. Therefore, development efforts to change the existing socio-economic condition of the nation would also be dependent on the efficiency of this sector for the foreseen future (Awoke, 2006).

However a better performance of the agricultural sector in particular and the sustainable economic growth of the country at large would be achieved through an improvement of the basic infrastructure. Consequently the road network has been identified as a serious bottleneck for the economic development of the country (ERA 1996).

Subgrades play an important role in imparting structural stability to the pavement structure as it receives loads imposed on it by road traffic (Gupta, 2010). The loads on the pavement are ultimately received by the soil subgrade for dispersion to the earth mass. It is essential that in no time the soil subgrade is over stressed. It means that the pressure transmitted on the top of subgrade is within allowable limit, not to cause excessive stress condition to deform the same beyond the elastic limit. Therefore it is desirable that at least the top 50 cm layer of subgrade soil is more suitable and well compacted under controlled conditions of optimum moisture content and maximum dry density (Deepika, 2012). It is necessary to evaluate the strength properties of the soil subgrade. Geotechnical soils characterization is important for the design and construction of road projects. During characterization visual descriptions, sampling and testing of suitable subgrade soil is required.

In the process of characterization, different techniques and procedures are applied for interpretation of subgrade soil condition. These interpretation techniques are often firms specific and are influenced by geological, topographic, and climatic conditions (Nibret, 2011). Where the naturally occurring local subgrade soils that are found around Jimma have poor engineering properties and low strength (Daniel, 2004).So improvement of subgrades are provided by way of lime/cement treatment or by mechanical stabilization and other to remove and replace the suitable borrow fill materials (Awoke, 2006).

The behavior how the soil responds for the applied mechanical load can be taken as the mechanical behavior of the soil. The mechanical behaviors of soils are different for various types of soil and affected by several conditions (Merihun, 2010). The volume change, deformation, strength and hydraulic conductivity of fine grained soils are very important for engineering problems (Mitchel, 1976). Jimma subgrade soils are affected by the mechanical behavior of the soil. So to remove this affected Soil and replace the suitable subgrade soil characterization is required to determine its suitability as subgrade materials. In this study it has not been determine the property of the soil, but it has been done its suitability for subgrade soil and I have studied already used quarry sites but it is not studied in detail its suitability property so I have studied its suitability and recommend which site is the suitable soil.

1.2 Statement of the problem

Road failure is a common problem in developing countries, including Ethiopia. Some of the problems plaguing these roads are faulty design, inadequate drainage system, poor maintenance, weak subgrade materials, poor construction techniques and corruption practices (Mohammed, 2012). Whereas the failure of roads constructed on expansive soils is mainly due to the expansive nature of the subgrade material (Hagos 2006). Addis Ababa–Jimma rehabilitation road project which is highly affected by the expansive natural subgrade is also affected by subgrade problem which causes road failure before its design period (Awoke 2006).Therefore, identification of factors that influence the soil strength, studying their relationship with different parameters and performing necessary tests on local representative soil sample can be considered as good insight of soil behavior used as subgrade materials.

Present practice in determining the adequacy of compacted soil subgrade is to determine the bearing capacity, water content, California bearing ratio test (CBR), plastic index (PI), permeability, grain size distribution, and dynamic cone penetration test (DCPT) of the soil which then helps to know its suitability for road construction. Proper understanding of the properties of the subgrade could help understand the actual causes of failure of road and will lead to proper design and construction by avoiding or minimizing the failure of structure.

This research focuses on characterizing the suitability selected subgrade materials from Seto, Seka, Dedo and Jeren sites in Jimma Zone which are already used as subgrade materials for road construction. These four sites are the main sources of materials for subgrade materials in road construction in and around Jimma area.

1.3 General Objective

The main objective of this research was to characterize the suitability of selected soils for use as subgrade materials in Jimma zone.

1.3.1 Specific Objectives

- To determine the geotechnical properties of soils in relation to subgrade parameters.
- To determine and classify the suitability of the soils those are found around Jimma area for use as subgrade materials.

1.4 Significance of the study

The research result gave:

a) Some understanding to the Jimma town administration, contractors that are found around Jimma town and the next researcher on the properties of the soil in the study sites.

b) Some knowledge to the stake holder on the suitable type of subgrade materials that are found around Jimma town.

1.5 Scope of the Study

The scope of the research study was to provide geotechnical characterizations of the subgrade materials and its suitability for road construction that were found around Jimma Town. The research was conducted based on data collected from four sites located at the distance of 5, 7, 10 and 34 km from Jimma town. The data were collected at 150m interval along the route corridor for each site. The research involved field and laboratory testing, and analysis of the results. Field test applied is dynamic cone penetration test (DCPT) and laboratory tests which include California bearing ratio (CBR), grain size, Atterberg limit, permeability and compaction tests were carried out. Despite the shortage of fund for the research all efforts were done to collect and analyze the necessary data for the research.

1.6 Limitation of the Research

In this research, there were limited materials testing equipments such as triaxial compression test, direct shear tests, limited budget, literatures and previous works related to the geotechnical condition of the Jimma area. However, every effort was made to perform the present study in a scientific and logical manner.

2. LITERATURE REVIEW

2.1 Introduction

In this study a detailed literature review was carried out to acquire the necessary knowledge regarding the research objectives. The literature review mainly focused on the problems and possible solutions to the loose subgrade materials. The review gave consideration on the delineation of the design subgrade material into homogenous sections. Moreover, the literature review enabled to give a general description related to the specific project area such as; the local geology, vegetation, climate, soil and construction techniques, etc. Further, more the literature review also helped to understand, what methodology were adopted by the previous researchers and what are their ultimate findings were. A systematic compilation of relevant literature review to this study is presented in the following paragraphs.

Although a pavement's wearing course is important component of a road, the success or failure of a pavement is dependent upon the underlying subgrade material upon which the pavement structure is built. Thus, the subgrade must be able to support the loads transmitted from the pavement structure without undergoing excessive settlement (Hagos, 2006). Its load bearing capacity, moisture content volume changes and its load bearing capacity depends on subgrade strength such as compaction, moisture content and soil type. Hence, the relationships among the strength, density and moisture content of subgrade studied thoroughly.

There are varieties of material properties that can be used to characterize the behavior of pavement materials. As defined in the discipline of mechanics material properties are those characteristics of materials that do not depend upon the testing apparatus procedure or condition (Edris, 1976). Instead they are test independent relationships between causative and response quantities that can be observed or measured the suitable subgrade materials and its geotechnical behavior of the soil.

Material properties are important to define because they permit the use of mechanics in predicting the behavior of a pavement under the service conditions of traffic and weather (Lesic, 1995). Currently, empirical correlations developed between field and laboratory material properties are used to obtain highway performance characteristics (Barksdale, 1990). According to Ethiopian Roads Authority (ERA 2002), soil samples for subgrade material shall be taken at 0.5km interval for identification test and at 1km interval for CBR tests during final design phase. As per American Association of State Highway and Transportation Officials (AASHTO) design guide (1993) sample spacing shall be in the range from 150m to 450m interval during construction phase. Sample spacing for this research was at 150m regular interval.

Soils are formed by the process of weathering of the parent rock. The weathering of the rocks might be by mechanical disintegration, and/or chemical decomposition. The properties of the soil materials depend upon the properties of the rocks from which they are derived (Murthy, 1990). At any given site, a number of different soil types may be present, and the composition may vary over intervals of a little as a few inches (James, 1984**.** It has long been appreciated that the engineering classification of soils is greatly facilitated by taking into account the soil forming processes by which nature has created the various types of soil conditions. Similar combinations of soil-forming processes in different parts of the world have been found to lead to materials of similar index properties and similar engineering characteristics (Taylor, 1990).

Ethiopian Roads Authority Pavement Design manual (ERA, 2002), mentioned the following points regarding delineations of homogenous sections in the subgrade areas. A road section for which a pavement design is under taken should be sub-divided into Subgrade areas where the subgrade CBR can be reasonably expected to be uniform, or without significant variations. Significant variations in this respect mean variations that would yield different subgrade classes as defined here further below. However, it is not practical to create delineation between subgrade areas that would be too precise, and indeed this could be the source of confusion during construction. The soil investigations should delineate subgrade design units on the basis of geology, pedology, drainage conditions and topography, and consider soil categories which have fairly consistent geotechnical characteristics (e.g. grading, plasticity, CBR). Usually, the number of soil categories and the number of uniform subgrade areas will not exceed 4 or 5 for a given road project.

2.2 General Types of Soils

According to grain size, analysis soil particles are classified as cobbles, gravel, sand, silt and clay (ASTM D-2487).Grains having diameters in the range of 4.75 to 76.2 mm are called gravel. If the grains are visible to the naked eye, but are less than about 4.75 mm in size the soil is described as sand. The lower limit of visibility of grains for the naked eyes is about 0.075 mm. Soil grains ranging from 0.075 to 0.002 mm are termed as silt and those that are finer than 0.002 mm as clay (ASTM D 422). This classification is purely based on size which does not indicate the properties of fine grained materials (Murthy, 1990).

2.3 Soil Types

For the design of any highway construction pavement, it is obligatory for the civil engineers to identify and classify the soil as per the nature. Broadly, the soil types can be categorized as laterite soil, moorum / red soil, desert sands, alluvial soil, clay including black cotton soil (Tom, 2009).

According to AASHTO most of the researchers classified the soil as follows:

- Gravel: Gravels are course materials with particle size less than 2.36 mm with little or no fines contributing to cohesion of materials.
- Moorum: These are the decomposition and weathering products of the pavement rock. These are the finer contents and visually similar to that of gravel.
- Silts: Silts are finer than sand and exhibit little cohesion and as compared to clay, these are brighter in color. Another property of this soil is dilatancy, i.e. a lump of silty soil when mixed with water, squeezed and tapped a shiny surface makes its appearance.
- Clays: These are finer materials. These kinds of soils possess stickiness, high strength when dry, and show no dilatancy. Soils like Black cotton and other expansive clays show swelling and shrinkage properties

Tabel2. 1Soil classification based on grain size adapted from (Khanna, 2001)

Gravel	Sand			Silt			Clay		
	coarse	medium	fine	coarse	medium	fine	coarse	medium	fine
2.0mm	0.6	0.2	0.06	0.02	0.006	0.002	0.0006	0.0002	

2.4 Design Guide for Flexible Pavement AASHTO (1993)

Design guide for flexible pavement as per AASHTO (American Associations of State Highways and Transportation Officials) (1993) suggests determination of homogenous sections using the CBR at 95% of the MDD (Maximum dry density) and analysis of Unit delineation by cumulative differences. In this method, Group index value and quality of subgrade materials are correlated. The (AASHTO, 2004) soils classification includes seven basic groups (A-1 to A-7) and twelve subgroups. Of particular interest is the Group Index, which is used as a general guide to the load bearing ability of a soil. The group index is a function of the liquid limit, the plasticity index and the amount of material passing the 0.075mm sieve. Under average conditions of good drainage and thorough compaction, the supporting value of a material may be assumed as an inverse ratio to its group index, i.e. a group index of '0' indicates a "good" subgrade material and a group index of '20' or more indicates a poor subgrade material.

Using AASHTO classification and test methods M145, Group Index (GI) is calculated by equation 2.1.

$$GI = (F - 35)(0.2 + 0.005(LL - 40)) + 0.01(F - 15)(PI - 10) \quad \text{.......eq. 2.1}$$

Where:

F = the percentage passing sieve size 0.075mm (N0. 200), expressed as a whole number

LL = liquid Limit

PI = Plasticity index of the soil.

2.5 Pavement Support

The repaired subgrade is the upper portion (typically 30.48cm) of a roadbed upon which the pavement and subbase are constructed. Pavement performance is expressed in terms of pavement materials and thickness. Although pavements fail from the top, pavement systems generally start to deteriorate from the bottom (subgrade), which often determines the service life of a road. Subgrade performance generally depends on two interrelated characteristics:

1. Load-bearing capacity. The subgrade must be able to support loads is transmitted from the pavement structure, which is often affected by degree of compaction, moisture content, and soil type (Subhash, 2011)

2. Volume changes of the subgrade. The volume of the subgrade may change when exposed to excessive moisture or freezing conditions (Schaefer, 2008).

(Atkins, 1983) disclosed that the main variables in the design of pavement structure is its thickness; the criteria being the strength of the subgrade soils and magnitudes of the imposed loads. He also discussed regarding highway materials and soils in brief. In his statement, he rated silts and clays poor materials only under the following conditions:

i)When they occur in low lying areas where the natural drainage is very poor and will not be improved.

ii) Where the condition of water table and climate are such that severe frost heave can be expected.

iii) Where high percentage of mica-like fragments or diatomaceous particles produce a highly elastic condition.

iv) Where it is desired to “bury” highly expansive soils usually A-7-6 (CH) deeper in the section to limit the effect of seasonal variations in moisture.

(Atkins, 1983) further mentioned regarding the main functions of pavement structures, the load imposed by vehicles and the distributions of loads down the subgrade layer that are low enough to be carried without failure due to rutting, excessive settlement, or other types of stresses. The various methods for measuring the imposed loads subgrade strength values, the required pavement structures, have been suggested and used.

In addition, (Atkins, 1983) gave emphasis on the main components of a pavement structures, such as:

(i) Surface

(ii) Base

(iii) Subbase, collectively known as the pavement.

(iv) Compacted subgrade.

(v) Natural subgrade, collectively called the subgrade.

Finally (Atkins, 1983) suggested that additional thickness of asphalt pavement, above the minimum required level can be placed by base and subbase materials using the following equivalencies:

50.8mm base = 25.4mm asphalt

68.58mm sub base = 25.4mm asphalt

2.6 Need for the Study

Subgrades play an important role in imparting structural stability to the pavement structure as it receives loads imposed upon it by road traffic. Traffic loads need to be transmitted in a manner that the subgrade deformation is within elastic limits, and the shear forces developed is within safe limits under adverse climatic and loading conditions (Gupta 2010). The subgrade comprises unbound earth materials such as gravel, sand, silt and clay that influence the design and construction of roads.

Traditionally, flexible pavements are designed based on the CBR approach or by considering elastic deformations. The CBR approach to pavement design gained popularity among practicing engineers in the late1980s with the use of advanced computing power and speed (Rollings, 2003). But this approach to flexible pavement design gives more importance to the estimation of the density of the subgrades and the pavement layers. Other design philosophies for flexible pavement do exist, including those with more of a basis in the theory of the mechanics of materials such as layered elastic and finite element approaches. In the classical approach to design of flexible pavements using the Burmister's (1958) layer theory, it is required to estimate the elastic modulus of the subgrade in order to determine the required layer-thickness of a pavement structure. Despite the advances in these state of the art approaches to pavement design, the CBR approach continues to be one of the most reliable methods for pavement design, especially in the design of pavements for military and civilian aviation (Semen, 2006).

This method is supported by more than 60 years of field experience under a wide range of conditions throughout the world. In addition to this, the approach to quality control of pavements gives more importance to the determination of in-situ density and moisture content. Of the various methods of evaluating the subgrade strength, the use of dynamic cone penetrometers (DCP) has become wide spread since they can be easily fabricated, and they facilitate rapid testing and evaluation of subgrades based on the resistance offered to penetration. In the new developments in the field of pavement evaluation, there exists a need to correlate the results obtained by using DCP to those obtained using traditional approaches such as the CBR for the benefit of road engineers. (Gupta, 2010).

2.7 Subgrade Performance

A subgrade's performance generally depends on three of its basic characteristics all of which are highly interrelated (Subhash, 2011).

i. Load bearing capacity - the subgrade must be able to support load transmitted from the pavement structure. This load bearing capacity is often affected by degree of compaction, moisture content, and soil type (Subhash, 2011).
ii. Moisture content - moisture tends to affect a number of subgrade properties including bearing capacity shrinkage and swelling. Moisture content can be influenced by a number of things such as drainage ground water table elevation infiltration or pavement porosity (which can be assisted by cracks in the pavement generally highly wet subgrades will deform excessively under load (Subhash, 2011).

Shrinkage and/or swelling - some soils shrink or swell depending on their moisture content additionally soils with excessive fines content may be susceptible to frost heave in some climates. Shrinkage swelling and frost heave will tend to deform and crack any pavement type constructed over them (Subhash, 2011).

2.8 Density - Moisture Relationship

The most common measure of compaction of soil is its density. Soils density and optimum moisture content should be determined according to AASHTO T180. Optimal engineering properties such as shear strength for a given soil type occur near its Maximum dry density (MDD) and Optimum moisture content (OMC). At this state, soils void ratio, potential to shrink and swell is minimized. Field density is then correlated to moisture density relationship in the laboratory for quality controlling purposes in the field (Arora 1997).

2.8.1 Effect of Moisture Content

Moisture migration depends on the geological formation, climatic condition, topographic features, soil type and ground water level. The most common method of moisture transfer is by gravity. The seepage of surface water, precipitation and snow melting in to the soil are common examples (Murthy, 2007).

2.8.2 Factor Affecting Volume Change

ASTM defines swelling pressure is the pressure, which prevents the specimens from swelling. i.e., it is the pressure required to return the specimens back to its original state after swelling (Void ratio, height). Swell is influenced by initial moisture content, initial dry density, surcharge pressure, time allowed for swell, size and stratum thickness and degree of saturation.

i) Initial moisture content

As the initial moisture content increases, the change in volume decreases but the absolute value of the swelling pressure does not depend on the initial moisture content. It is a constant at any initial moisture content

ii) Initial dry density

As the initial dry density increases, the swelling potential increase and the swelling pressure also increases. Therefore, compaction of expansive soil to a higher density is not good during construction.

2.9 Relationship between swelling potential and plasticity index, PI

(Chen, 1988) defines, swelling soils, which are clayey soils, are also called expansive soils. When these soils are partially saturated, they increase in volume with the addition of water. They shrink greatly on drying and develop cracks on the surface. These soils possess a high plasticity index. The clay mineral that is mostly responsible for the expansiveness belongs to the montmorillonite groups. The soils containing a considerable amount of monmorrilonite minerals exhibit high swelling and shrinkage characteristics (Harishkumar, 2011). (Chen,1988) added that expansive soils are residual soils which are the result of weathering of parent rocks. The depth of these soils in some region may be up to 6m or more. Swelling and expansion pressures of clayey soils also increase with an increase in dry density, up to the volumetric shrinkage limits. However, beyond this shrinkage limit, the swelling pressure becomes constant while the dry densities further increase (Harishkumar, 2011).

2.10 Relationship between CBR and Shear Strength

Treating CBR test as a bearing capacity problem in which the standard plunger acts as a small foundation, the following relation can be obtained (Carter, 1991).

$CBR = qu * 100 / 6900 = .097 qu$ ………………………………..Eq.2.2

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/m2 at penetration of 2.5mm, giving

Terzaghi‘s bearing capacity equation for foundation is:

$qu = CNc + poNq + 0.5\gamma BN\gamma$ …………………….Eq.2.3

Where

C=is the cohesion of the soil ϒ=the bulk density of the soil

Po =the overburden pressure at the base of the plunger. B=the diameter of the plunger and N_C, N_q, N_Υ are Terzaghi's bearing capacity factors.

2.11 Subgrade Soil

2.11.1 Significance of subgrade soil

Subgrade soil is the integral part of the road pavement structure which provides support to the pavement. The subgrade and its different properties are very much important in the pavement design structure. The major function of the subgrade is to provide the support to the pavement against traffic loading and for this the subgrade that should possess sufficient stability under adverse climate and heavy loading conditions (Subhash, 2011). When soil is used in the embankment construction, along with stability incompressibility is also an important factor as differential settlement may cause failures. Compacted and stabilized soil is often used as subbase or base course (Berhanu, 2005). The soil or subgrade is therefore considered as one of principal highway material.

2.11.2 Subgrade strength

The strength of a soil or subgrade can be determined by using a test known as California Bearing Ratio Test which was developed by corps of engineer in California in the year 1930's and it is way to determine the standard soil properties such as density (Bowles, 1984). It is graph showing the values for aspect of design of road pavement. Mostly all the design charts are based on the value of CBR for the subgrade (Subhash, 2011).

3. Methodology

3.1 Study Area

Jimma town is located at about 350km from Addis Ababa, and it is bordered on the south by the Southern Nations, Nationalities and Peoples Region (SNNPR), the Northwest by Illubabor Zone, on the North by East Wellega, and on the Northeast by West Shewa Zone of the Oromia Regional State. The town covers a total area of 18,412.54 square kilometers. The altitude of the area ranges from 1300m to 2100m. The mean annual rainfall of the area is between 1600 mm to 2000 mm with maximum rainfall between months of June and September. The annual mean temperature of the area is between 15 ^{0}C and 25 ^{0} C. Based on data from the CSA (2005), the town has an estimated total population of 2,773,730, of whom 1,382,460 are men and 1,391,270 are women; 340,666 or 12.3% of its population are urban dwellers (CSA, 2005).Sites from the nearest two weredas, from Jimma town were included in this study. These are Dedo and Seka. The main crop is coffee which is their main source of income and Jimma is a big market center.

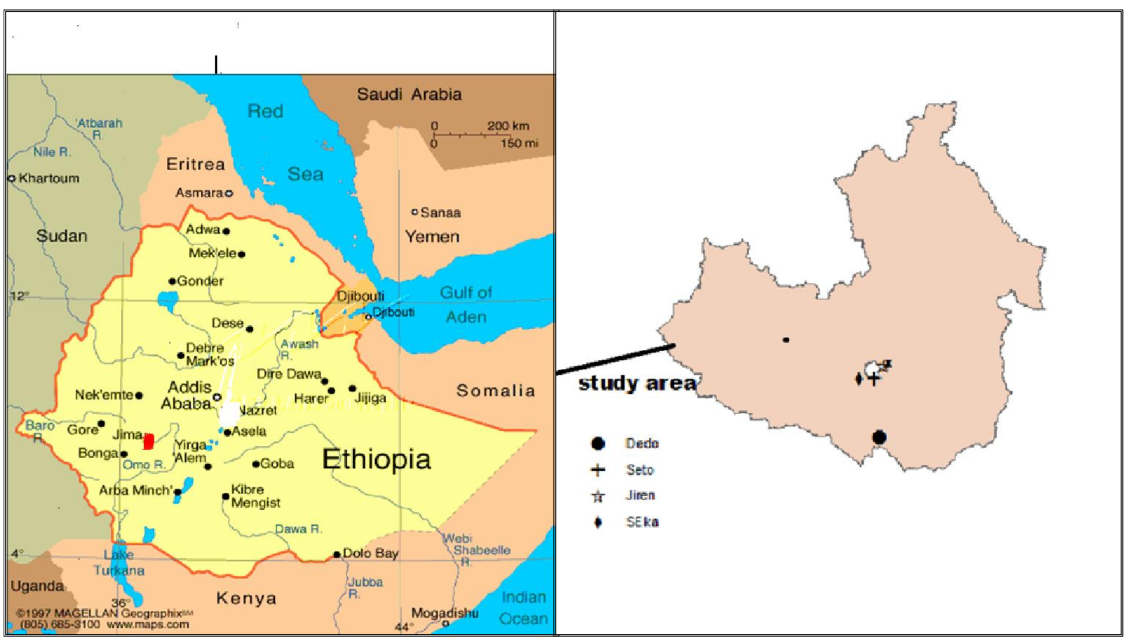

Figure3. 1 Location map of the study area

3.2 Soil of the study area

From the visual inspection of the subgrade soil during the site visit, in most parts of the study area the soil is characterized by reddish, reddish brown, yellowish and dark gray soils. The soil samples from Dedo and Seka sites are course grained but visually dark gray and yellowish colure respectively. The soil samples from Seto and Jeren sites are fine grand but visually reddish and reddish brown colure.

(a)

(b)

(c)

Figure3. 2 Sample photos of the study area (a)Seka (b)Jeren(c)Seto and Dedo respectively

3.3 Laboratory investigation for subgrade soils

The subgrade soil investigation conducted under present study incorporated field laboratory investigations and test results evaluation. In the field, subgrade soil extensive survey investigation and sampling has been carried out from July 2013 to middle of December, 2013 in connection with the detailed investigation during the construction phase of the road. Representative soil sampling at 150m interval has also been taken. The data consist of suitability of subgrade CBR with corresponding consequent Atterberg limit test, compaction test, and necessary classification based on AASHTO standard, for various geographical locations in Ethiopia.

Tabel3. 1Standard test procedures

Parameters	Standard
Sieve analysis	ASTMD422-63
Atterberg limit test	ASTMD4318
Compaction test	ASTMD1557T99
CBR test	AASHTO T180)
DCPT	AASHTO
Permeability	ASMMD2434

3.4 Representative photographs of laboratory test

Figure3. 3 Hydrometer analysis **of the studied soils.**

Figure3. 4 Liquid limit and Plastic Limit Test

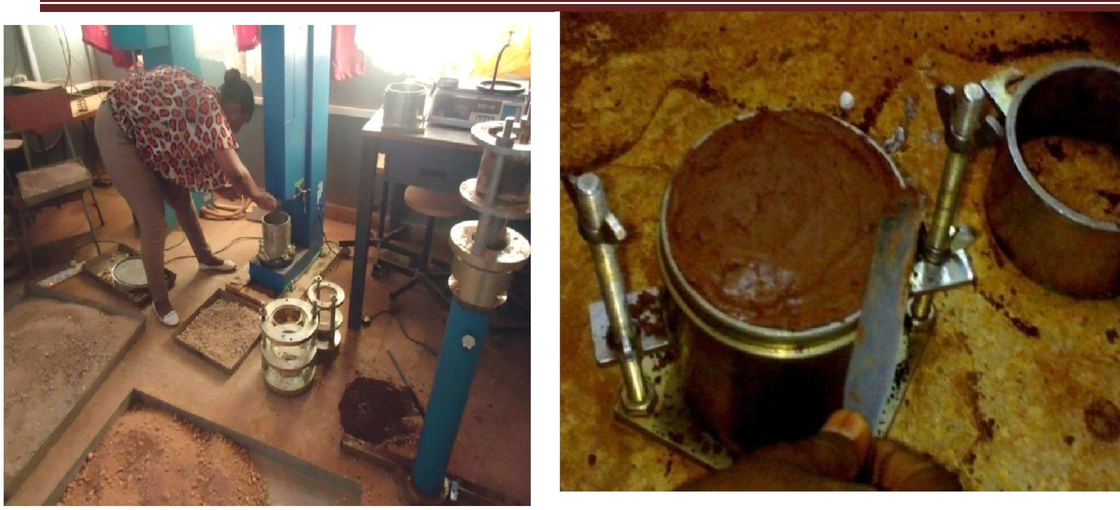

Figure3. 5Compaction test

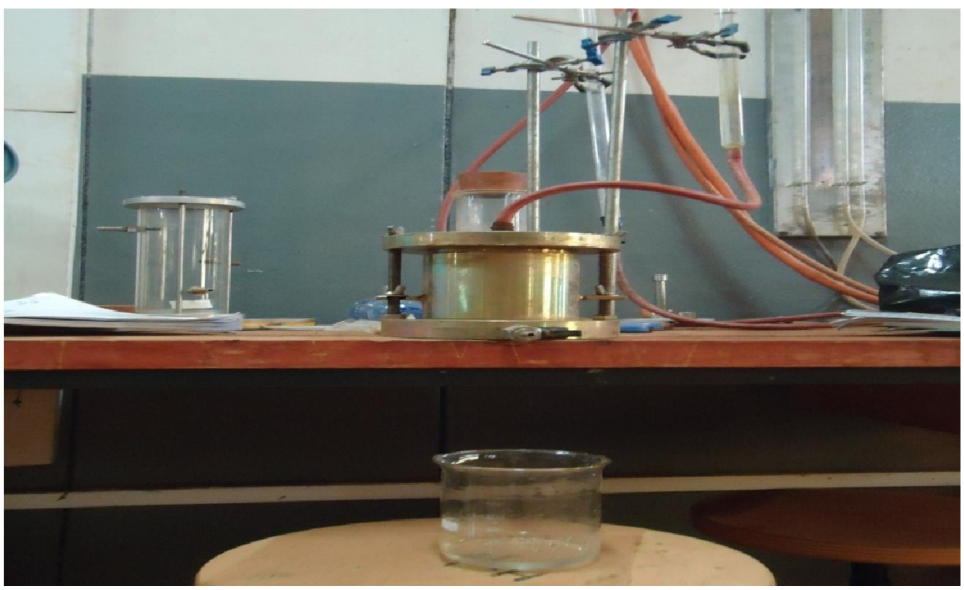

Figure3. 6 Falling head Permeability test set up

Figure3. 7 CBR Test Arrangement

4 GENERAL CHARACTERIZATION OF SUBGRADE SOILS RESULTS AND DISCUSSION

4.1 General

For the design and construction of highway and airfield, it is imperative to carry out tests on construction materials. The inherent economy in construction depends upon the maximum use of local material. The prime objective of the different tests in use is to know and to classify the pavement materials into different groups depending upon their physical and strength or stability characteristics.

Soil is very essential highway material because of:

I) Soil subgrade is part of the pavement structure; further the design and behavior of pavement especially the flexible pavements depend to a great extent on the subgrade soil.

II) Soil is one of the principal materials of construction in soil embankment and in stabilizing soil base and subbase courses.

Based on data collected in the field, index tests conducted for subgrade soil samples and samples in the laboratory, analyses has been made by integrating the primary results with secondary data obtained. The interpretation and discussion to characterize the subgrade have been made in the following paragraphs.

4.2 Grain Size Distributions (Gradation)

Gradation, or the distribution of particle size within a soil, is an essential descriptive feature of soils. Soil texture such as gravel, sand, silty and clay and engineering classifications are based large on gradation. Many engineering properties like relative compaction, permeability, strength, swelling potential, and susceptibility to frost action are closely correlated with gradation parameters. Gradation is measured in the laboratory using two tests: a mechanical sieve analysis for the sand and coarser fraction, and a hydrometer test for the silt and finer clay material (FHWA -05-037, 2006).

Tabel4. 1 Results of the grain size distribution of soil samples

SIEVE ANALYSIS, PERCENT PASSING (%)	Seka	Seto	Jeren	Dedo
4.75	90.3	74.88	92.1	97.5
2	76.9	60.38	87.6	70.02
1.18	73.1	57.48	75.6	60.92
1	62.5	56.88	72.4	41.32
.0.5	50	50.48.	66.00	28.318
.015	41	34.68	61.00	24.62
.075	34.2	27.78	59.4	20.72
.046	21.6	25.38	57.6	17.48
.033	15.3	21.54	52.2	14.89
.021	11.7	19.23	46.8	12.95
.012	6.3	16.92	43.2	9.71
.008	3.6	13.85	41.4	7.77
.006	3.6	10.77	37.8	6.47
.004	2.7	9.23	32.4	5.83
.003	1.8	8.46	23.4	5.18
.002	.9	7.69	16.2	4.53
.001	0	6.92	9	3.88

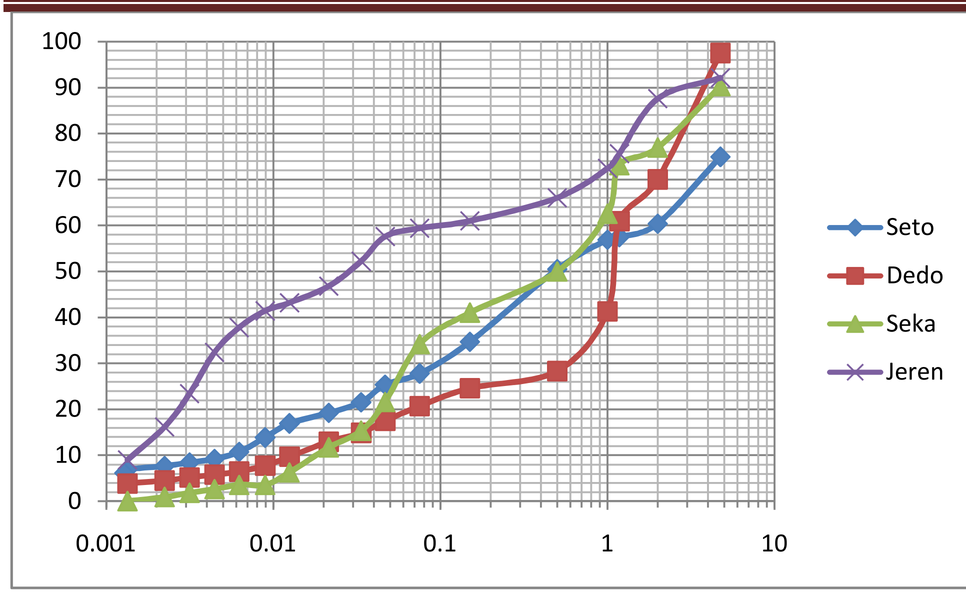

Figure4. 1 Chart showing the results of the grain size distribution analysis of the samples

As observed from the Figure 4.1and Table 4.1 grain size analysis tests revealed that, the four sites sample result implied according to AASHTO Soil Classification the most suitable soils are Seto, and Dedo site these sites soils indicates that course grained but Seka and Jeren site sample result show that fine grained soil .

According to AASHTO classification the most suitable soils that pass less than 35% passes the No .075 sieve are course grained soils and greater than 35% pass No .075 sieve then this soil type shows fine grained soil.

Grain size analysis tests revealed that, starting from few centimeters below the ground level to the depth of investigation which indicate that samples from Seto and Dedo sites are course grained soils while samples from Jeren and Seka site show fine grained soil.

4.3 Atterberg Limits

Atterberg limit correspond to values of moisture content where the consistency of the soils change as it is progressively dried from slurry. Plasticity is the response of a soil to changes in moisture content. When adding water to a soil it changes its consistency from hard and rigid to soft and pliable, the soil is said to exhibit plasticity. Clays can be very plastic, silts are only slightly plastic, and sands and gravels are non-plastic. For fine-grained soils, engineering behavior is often more closely correlated with plasticity than gradation (Robert, 1981). Plasticity is a key component of AASHTO soils classification. The shear strength of clay soils at its liquid limit is constant but varying with plastic limits (Arora, 1997) Chen (1988) suggested some relationship between plasticity index and swelling potential shown in (Table 4.2).

Tabel4. 2Relationship between swelling potential and plasticity index (PI) (Chen, 1988)

Plasticity index, PI (%)	Swelling potential
0-15	low
10-35	medium
20-55	high
≥35	Very high

Atterberg tests were carried out on samples from the four sites and results of the tests and associated index properties are indicated in Table 4.3 and Figure 4.2.

Tabel4. 3Results of the Atterberg limit tests and associated index properties of soil sample using ASHTO Classification System

sample		LL	PL	PI	Soil group	GI	ASHTO classification
Seka	Trial-1	31.31	22.21	9.1			
	Trial-2	24.25	22.31	1.94			
	Trial-3	25.22	22.24	2.98			
	average	26.93	22.25	4.67	A-2-4	0	A-2-4(0)
Seto	Trial-1	51.5	35.84	15.66			
	Trial-2	51.24	34.93	16.31			
	Trial-3	51.35	35.6	15.75			
	average	51.36	35.46	15.91	A-2-7	3	A-2-7(3)
Jeren	Trial-1	65	43.72	21.28			
	Trial-2	55.5	43.06	12.44			
	Trial-3	58.2	44.09	14.11			
	average	59.6	43.62	15.95	A-2-7	11	A-2-7(11)
Dedo	Trial-1	65.4	46.62	18.78			
	Trial-2	65.9	45.9	20			
	Trial-3	61.75	48.15	13.6			
	average	64.35	46.89	17.46	A-2-7	0	A-2-7(0)

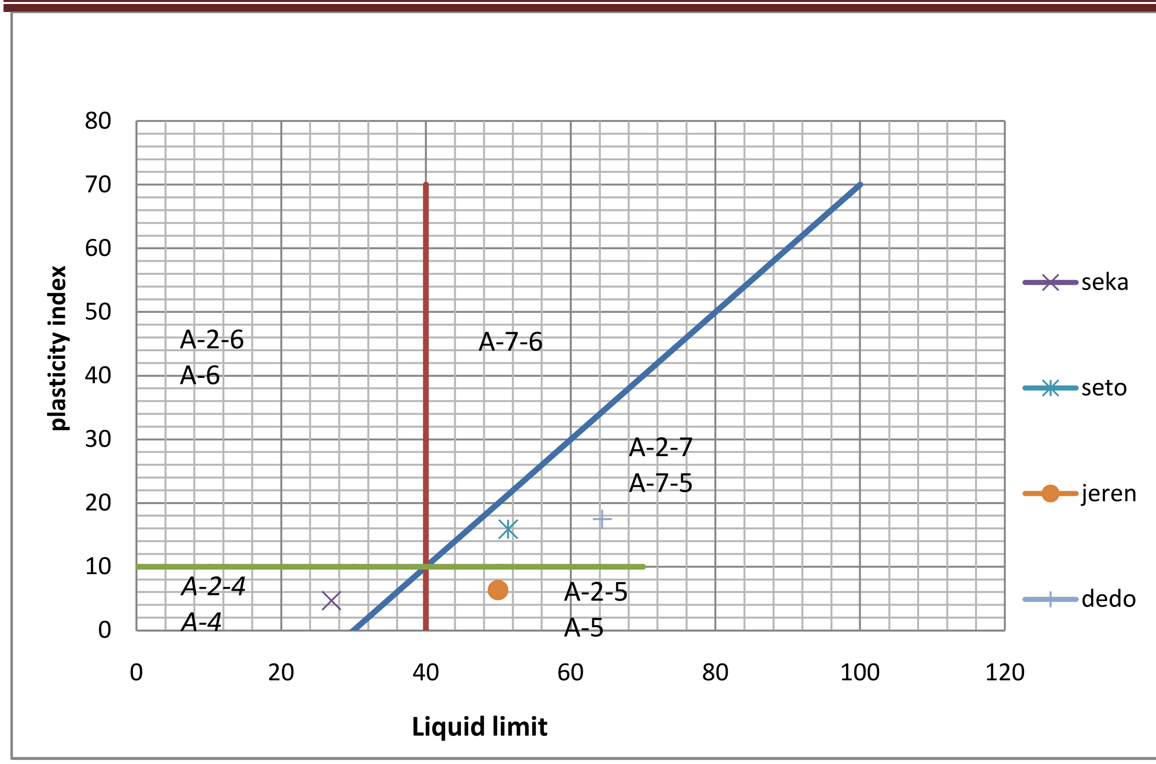

Figure4. 2 Plasticity chart of the soil in study area (according to AASHTO soil classification system).

According to ERA manual (2002), soils with PI values greater than 20% are suitable subgrade materials so all sites results show that suitable subgrade materials.

Relationship between swelling potential and plasticity index,(PI) Table4.2 showed that soils from Seka, Seto and Jeren sites are characterized by low swelling potential while the soils from Dedo sites have medium swelling potential.

According to AASHTO classification all site samples were A-2 silty or clayey gravel and sand so general subgrade rating showed that excellent to good subgrade materials.

The higher the Group Index the lesser suitable the soil as subgrade material. If this number is near 20 or more, then the subgrade support is usually considered poor because of the presence of a high percentage of fines with moisture sensitivity.

Generally according to GI and PI, laboratory result all my study quarry sites are suitable subgrade materials.

4.4 Compaction Test

Compaction tests are performed using disturbed, prepared soils with or without additives. Normally, soil passing the No. 4 (4.75mm) or 19mm sieve is mixed with water to form samples at various moisture contents ranging from the dry state to wet state. These samples are compacted in three layers in a mold by a hammer in accordance with specified nominal compaction energy. Dry density is determined based on the moisture content and the unit weight of compacted soil. The water content at which this dry density occurs is termed as the optimum moisture content (OMC). The test is done in the laboratory according to AASHTO T 180 (Modified Proctor).

Samples collected from the four sites were compacted according to manual and were the results analyzed of Compaction test was shown in Table 4.4 and Figure 4.3.

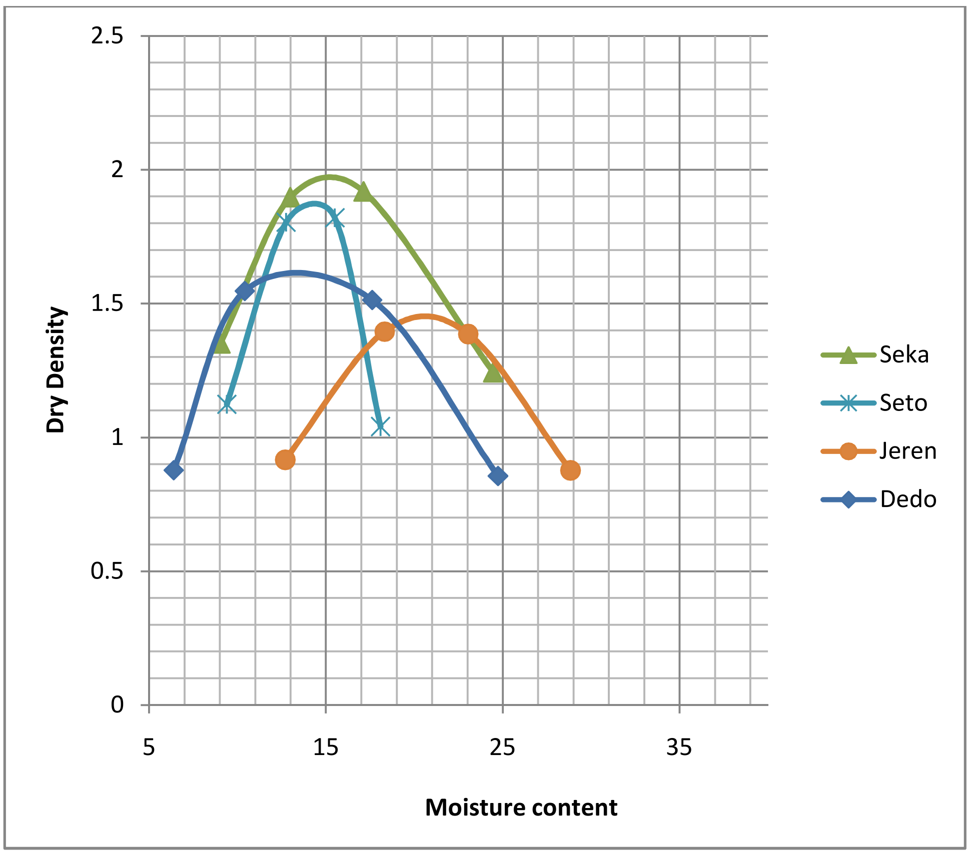

Figure4. 3 MDD vs MC for the soils from different sites

Tabel4. 4Compaction test result and classification according to AASHTO and pavement manual (2007)

site		OMC %	MDD(g/cm3)	Typical soils based on Dry Densities
Seka	Trial-1	17.4	1.91	
	Trial-2	16.9	1.94	
	Trial-3	17.3	1.91	
	average	17.2	1.92	Gravel and Sand
Seto	Trial-1	14.9	1.79	
	Trial-2	16.23	1.83	
	Trial-3	15.72	1.82	
	average	15.6	1.81	Silts and Clay
Jeren	Trial-1	18.6	1.34	
	Trial-2	18.23	1.48	
	Trial-3	18.12	1.53	
	average	18.3	1.45	Silts and Clay
Dedo	Trial-1	10.34	1.52	
	Trial-2	10.27	1.58	
	Trial-3	10.56	1.56	
	average	10.4	1.55	Silts and Clay

Tabel4. 5Typical Dry Densities of soils (Pavement Manual, 2007)

a Soil	ϒd kg/m3(lb/ft3)
Gravel and Sand	1,900 – 2,250 (120 – 140)
Silts and Clay	1,450 – 1,750 (90 – 110)
Peat	~ 300 (20)

According to Table 4.5 compaction test result and classification shows that soils from Seka are gravel and sand, while soils from Seto are silts and clay. Moreover, soils from Dedo and Jeren site are Silts and Clay respectively. The last criterion used for the classification of the subgrade soils is the Maximum Dry Density. As mentioned above in section, the maximum dry density is obtained by AASHTO T 180 (D) with the modified proctor methods of 56 blows. The minimum values of MDD for subgrade material are supposed to be 1.5gm/cc (Mytsebri Shire, 2009). Hence, taking this in to account, the soils from Seka ,Seto Jeren and Dedo are suitable subgrade materials

- Compaction test result according to AASHTO showed that:

  (a) Samples from Seka site are characterized as A-2 (gravel and sand)

  (b) Samples from Seto site are typically of A-2 (silts and clay), and

  (C) Samples from Dedo and Jeren are classified as A-2 (silts and clay).
- This result show that samples from Seka, Seto and Dedo sites are suitable borrow fill materials.

4.5 California Bearing Ratio

The California bearing ratio test was first developed in California by the Division of Highways in 1929 as a means of classifying the suitability of a soil for use as subgrade or base course material in highway construction. During World War II, the US corps of engineers adopted the test for use in airfield construction (Bowles, 1984). The CBR test (ASTM terms the test simply as a bearing ratio test) measures the shearing resistance of a soil under controlled moisture and density conditions. The test yields the bearing ratio number, but from previous statement, it is evident this number is not a constant for a given soil but applies only for the tested state of the soil. The CBR number is obtained as the ratio of the unit load (in KN/m2) required to effect a certain depth of penetration of the penetration piston in to a compacted specimen of soil at some water content and density to the standard unit load required to obtain the same depth of penetration on a standard sample of crushed stone.

According to ERA (2002), the quality of materials to be used for replacement must have the following quality parameters:

i) Its CBR value shall be greater than 5% at 95% of AASHTO (T180)

(ii) The plasticity index shall be not exceeding 20%

(iii) The maximum swell value of 1.5%

Samples were collected from the four study sites were carried at for CBR tests were carried out on those samples. The results were presented in Table4. 6.

Tabel4. 6CBR value of each site

site	No of blow				
	10	30	65	CBR at 95%	CBR %
Seka	9.9	10.9	11.3	10.2	10.7
Seto	10	10	11	9.82	10.33
Jeren	2	2	3	2.22	2.33
Dedo	3	3	5	3.48	3.67

Tabel4. 7Subgrade strength categories according ERA manual

Sub grade strength Classes	Range (CBR %)	CBR value			
		Seka	Seto	Jeren	Dedo
S1	2			2.33	
S2	3-4				3.67
S3	5-7				
S4	8-14	10.7	10.33		
S5	15-29				
S6	30+				

The CBR value of the samples from Seto and Seka sites are 10.7% and 10.33% respectively. For the samples from Dedo and Jeren the CBR values are 3.67% and 2.33% respectively. According to ERA (2002) CBR value >5% are good subgrade materials. Based on this, the materials from Seto and Seka sites are suitable borrow fill materials. But the CBR values of samples from Jeren and Dedo sites are categorized as unsuitable for subgrade as their CBR values fall below 5%.

4.6 Dynamic Cone Penetration Test (DCPT)

The dynamic cone penetration test (DCPT) was originally developed as an alternative to evaluating the properties of flexible pavement or subgrade soils. The conventional approach to evaluate strength and stiffness properties of asphalt and subgrade soils involves a core sampling procedure and a complicated laboratory testing program such as resilient modulus, Marshall tests and others(Livneh et al. 1994). Due to its economy and simplicity, better understanding of the DCPT results can reduce significantly the effort and cost involved in the evaluation of pavement and subgrade soils. The relationship developed between field DCP value and the laboratory CBR obtained from the study is given in Eq. (4.1) and Eq. (4.2). The relationship is plotted in Figure 4.4.The DPI for each drop was used to calculate an average DPI for both the upper 75-mm and 150-mm. The first seating drop was not used. These average DPIs were then used to calculate the California bearing ratio (CBR) using equations 4.1 and 4.2 developed by the Corps of Engineers (Webster. 1992, 1994).

$CBR(percent) = 292 / DPI^{1.12}$ …………………………….. Eq (4.1)

Equation 4.1 is used for (CBR greater than 10 percent and DPI units are in mm/blow).

$CBR(percent) = 1/(0.017019 * DPI)^2$ ……………………… Eq (4.2)

Equation 4.2 is used for (CBR less than 10 percent and DPI units are in mm/blow).

Tabel4. 8DCPT result of each sample sites

Sample site	Reading	No. of blows												
		0	2	4	6	8	10	12	14	16	18	20	22	24
Jeren	Trial-1	0	21	41	65	84	95	111	123	135	148	163	179	196
	Trial-2	0	26	50	77	100	123	142	155	169	183	198	215	230
	Trial-3	0	16	30	45	57	70	87	102	111	120	130	137	147
	Avg	0	21	40	62	80	96	113	127	138	150	164	177	191
Seto	Trial-1	0	16	30	41	55	76	89	105	118	134	152	173	208
	Trial-2	0	25	41	57	77	101	121	143	163	184	205	231	254
	Trial-3	0	16	30	38	47	54	58	64	68	71	74	79	83
	Avg	0	19	34	45	60	77	89	104	116	130	144	161	182
Seka	Trial-1	0	14	24	34	44	55	67	75	87	102	116	137	147
	Trial-2	0	22	38	54	68	83	98	112	127	141	155	171	188
	Trial-3	0	21	35	42	54	69	83	100	115	131	145	156	186
	Avg	0	19	32	43	55	69	83	96	110	125	139	155	174
Dedo	Trial-1	0	39	41	42	55	70	100	130	145	155	170	175	180
	Trial-2	0	39	46	55	68	95	106	135	145	165	178	185	197
	Trial-3	0	40	41	45	58	86	97	128	135	156	165	177	185
	Avg	0	39	43	47	60	84	101	131	142	159	171	179	187

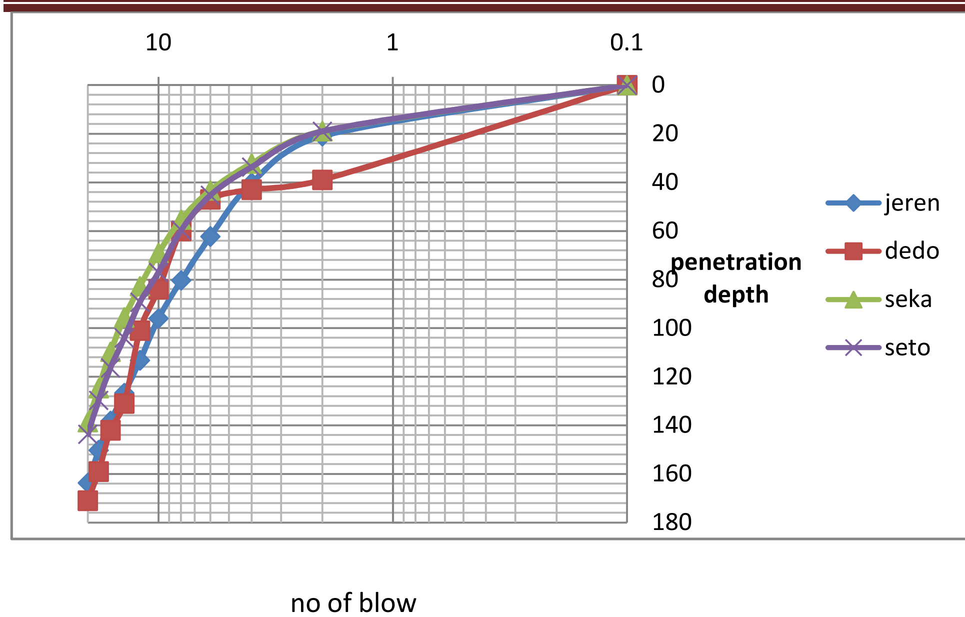

Figure4. 4DCPT chart

Tabel4. 9Relationships between DCP with CBR values of samples from the four sites

Site	CBR	PI(blow/mm)	DCPT
Jeren	28.6	7.95	191
Dedo	29.3	7.79	187
Seka	31.8	7.25	174
Seto	30	7.58	182

Most of the earlier relationships estimate higher values compared to the CBR value obtained from the present study. The reason why, most of the DCP tests were conducted in the laboratory on the specimens in a mould, which was affected by the lateral confinement as observed by Kleyn (1975) and Harrison (1987). Only the model by Sahoo et al. (2009) was performed in the field conditions. Hence the present model will be more useful to estimate the laboratory CBR value from the field DCP tests for subgrades with high bearing strength. Table 4.9 shows relationships between DCP with CBR values by correlation. According to ERA (2002), all the samples from the four sites are suitable subgrade materials.

4.7 Permeability

Permeability is the rate of flow of a fluid through a porous medium. When that fluid is ground water, the terms hydraulic conductivity (K) and coefficient of permeability (k) are essentially equivalent. However, for fluids other than water the permeability coefficient includes additional factors relating to the viscosity of the fluid. The terms hydraulic conductivity and coefficient of permeability are used interchangeably in much of the present literature. Although hydraulic conductivity is the preferred term in technical documents, the term permeability is often more easily understood (pavement manual, 2007).

The coefficient of permeability can be computed using the ASTM D-2434 equation.

$$K = (2.3aL/(At) * \log(h_o / h_f) \qquad \text{Eq4.3}$$

The permeability classification according to Head (1985), permeability less than 10^{-7}cm/sec are practically impermeable. The permeability of the samples taken was determined from the falling head permeability tests following ASTM D-2434 standard performed on each compacted samples.

Tabel4. 10Permeability test results and Permeability classification according to Head (1985)

Sample site		Coefficient of Permeability cm/s(k)	Degree of Permeability
Seka	Trial-1	$1.73*10^{-6}$	Very Low
	Trial-2	$1.64*10^{-6}$	Very Low
	average	$1.68*10^{-6}$	Very Low
Seto	Trial-1	$3.24*10^{-7}$	Very Low
	Trial-2	$3.39*10^{-7}$	Very Low
	average	$3.31*10^{-7}$	Very Low
Jeren	Trial-1	$2.72*10^{-6}$	Very Low
	Trial-2	$2.61*10^{-6}$	Very Low
	average	$2.67*10^{-6}$	Very Low
Dedo	Trial-1	$4.28*10^{-7}$	Very Low
	Trial-2	$4.31*10^{-7}$	Very Low
	average	$4.3*10^{-7}$	Very Low

According to table 4.10 result of the permeability analysis using pavement manual 2007 triangular textural classification, the samples from Seka, Seto , Jeren and Dedo sites are Sandy Clay Loam. This showed that the studied soils are characterized by impervious to semi pervious soil.

- Pavement manual, 2007showed that triangular textural classification of soil which indicates that permeability of soils was determined, so the results that shows the soils from Seka, Jeren , Seto and Dedo site is Clay with Very impervious property.
- Based on description of soil and by the unified soil classification system, K (coefficient of permeability) value were 10^{-5}–10^{-9} m/s shows that sand-silt mixtures SM, SL, SC (Bowles, 1997).
- By AASHTO soil classification system sandy-silt soil were excellent to good subgrade materials. Finer soils can be expected to have lower permeability, and well graded soils can be expected to be less permeable than more uniform soils. Furthermore, a decrease in permeability should be expected with increased dry density.

5 CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

In order to achieve the objectives of the present study a systematic methodology was followed. A total of 12 quintal samples for Seka, Seto, Jeren and Dedo site. For each site three quintal samples were taken at 150 m interval and the samples were taken randomly. Tests were conducted at field (i.e. dynamic cone penetration test (DCPT) and laboratory such as Atterberg limits, grading, and MDD, OMC and California bearing ratio test (CBR) and permeability tests) for the determination of geotechnical properties of selected quarries subgrade material suitability. Based on the test results, interpretations were made to meet the general objectives of the present study.

From the findings of the field and laboratory test results the following conclusions are drawn:

- The laboratory test results found during study proved that the subgrade materials of these quarries are suitable for the road construction at Jimma, since the investigations were done as per the ERA specification, and fulfill all the requirements of AASHTO.
- From consistency limit test results the liquid limit of the area ranges from26.93-64.35% plastic limit rages from 22.25-46.89% and plastic index from 4.67-17.46%.
- Grain size analysis tests revealed that, starting from few centimeters below the ground level to the depth of investigation which indicate that samples from Seka, Seto and Dedo sites are course grained soils while samples from Jeren site show that fine grained soil.
- The index properties result shows that samples from Seka, Seto and Dedo sites are silty or clayey gravel and sand while samples from Jeren site clayey soils. Therefore these two geotechnical test result showed that Seka, Seto and Dedo site samples are suitable borrow fill materials for road contraction.
- Results of the CBR test show that samples from Seto and Seka sites have CBR values greater than 5% which is 10.33% for Seto and 10.7% for Seka These samples are therefore good subgrade materials and suitable borrow fill materials.
- The void ratio of a soil has an important effect on permeability. Cohesive soils which are compacted on a high density ratio will have lower permeability than those compacted to a low density ratio. The void ratio also has an effect on the permeability of granular soils,

with soils compacted to a small void ratio (dense) having lower permeability than those with a high void ratio (loose).

- The value of MDD result shows that Seka, Seto and Dedo site results are suitable subgrade materials which is greater than 1.5gm/cc.
- For construction of new embankments or strengthening of existing pavements, DCPT will be a very useful tool for evaluating the strength of subgrade in terms of CBR value.
- So by correlation formula the relation between DCP with CBR value showed that all sites Seka, Seto, Dedo and Jeren site samples are suitable subgrade materials.
- From this study the laboratory and field test result shows that all my study site result are suitable subgrade materials and it is also used as a gravel road construction around Jimma zone.
- But according to its CBR result the more suitable subgrade materials are Seka and Seto sites because the soaked CBR result are the governing and greater than 5% according to ERA manual.

5.2 Recommendations

Recommendations given regarding construction on loose soil are;

- Seka and Seto quarries site samples are good subgrade materials for road construction around Jimma towns.
- It is recommended to apply all existing available theories and practices for the design and constructions of road located in loose soil areas. Great emphasis should be made especially on moisture control measures.
- Further works are required to determine the additional suitable subgrade quarry site around Jimma town because the natural subgrades that are found around Jimma town are loose soil.
- It is better to further investigate the extent of expansiveness of soil at Jimma area in its in-situ state and go for appropriate or suitable subgrade quarries.
- Shear strength property of soil is not studied in this research. Therefore, by studying the Shear strength characteristics, the Correlation of the index property with shear strength parameters may also be done.
- From the test results the maximum dry density (MDD) of Jimma ranges from1.42 to 1.92 g/cm3and the optimum moisture content ranges 10.4. to 18.3 percent. The summary of the test result is shown in Table 4.8. Generally course grained soils can be compacted to a higher dry density than fine gained soils for the some compaction effort. When some fines are added to the coarse 33 grained soils to file the voids, the maximum dry density further increases, but it the amount of fines is too much, more than required to fill the voids, it results in reduction of dry density; well graded soils can attain higher dry density than poorly graded soils. High plasticity clays attain much less dry density than low plasticity clays for the some completive effort. This can also be observed from Table 4.4 and Table 4.8 i.e. as the amount of courser particles increases the dry density will increase too.
- In this study it has not been determine the property of the soil, but it has been done its suitability for subgrade soil so further study are required the property of the soil that are found around Jimma town.

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7 APPENDIX

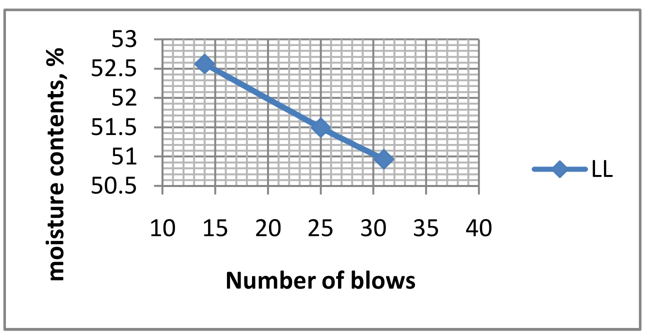
APPENDIX 1

ATTERBERG LIMIT SETO SITE

Trial 1

A) Liquid Limit

Depth Sample	1.5-4.0m		
No. of Blows	14	25	31
Can No.	A	B	C
Mass of wet soil + Can, gm	175.47	117.79	112.17
Mass of Dry soil + Can, gm	163.13	105.91	102.84
Mass of Can, gm	139.66	82.84	84.53
Mass of water, gm	12.34	11.88	9.33
Mass of Dry Soil, gm	23.47	23.07	18.31
Moisture Content, W %	52.57776	51.49545	50.95576
From the flow curve			
LL=	51.5		



B) Plastic Limit

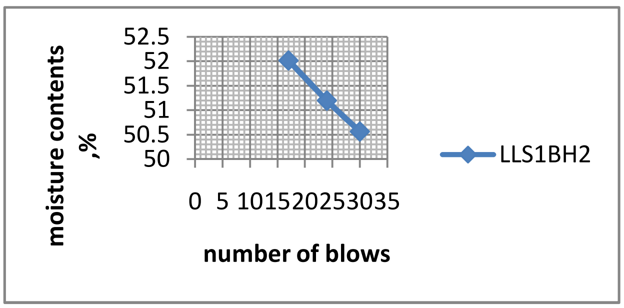
Depth Sample	1.5-4.0m	
Can No.	A	B
Mass of wet soil + Can, gm	147.92	98.23
Mass of Dry soil + Can, gm	144.84	93.23
Mass of Can, gm	136.2	79.352
Mass of water, gm	3.08	5
Mass of Dry Soil, gm	8.64	13.878
Moisture Content, W %	35.65	36.03
Plastic limit (%)	35.84	

Liquid Limit= 51.5 %
Plastic Limit= 35.84 %
Plasticity Index= 15.66

Trial 2

A) Liquid Limit

Depth Sample	1.5-4.0m		
No. of Blows	17	24	30
Can No.	A	B	C
Mass of wet soil + Can, gm	162.21	161.22	160.25
Mass of Dry soil + Can, gm	152.78	152.32	152.21
Mass of Can, gm	134.65	134.936	136.31
Mass of water, gm	9.43	8.9	8.04
Mass of Dry Soil, gm	18.13	17.384	15.9
Moisture Content, W %	52.0132	51.1965	50.56603774
From the flow curve			
LL=	51.24		



B) Plastic Limit

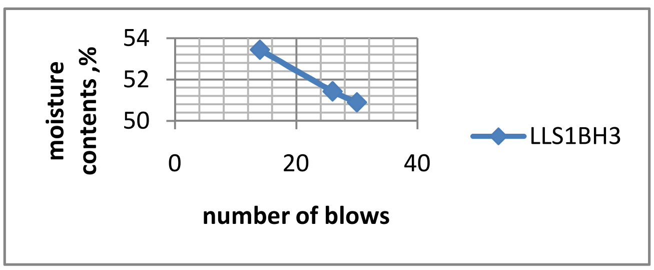
Depth Sample	1.5-4.0m	
Can No.	A	B
Mass of wet soil + Can, gm	97.15	98.23
Mass of Dry soil + Can, gm	92.43	93.21
Mass of Can, gm	78.918	79.352
Mass of water, gm	4.72	5.02
Mass of Dry Soil, gm	13.512	13.858
Moisture Content, W %	34.93	36.22
Plastic limit (%)	34.93	

Liquid Limit= 51.24 %
Plastic Limit= 34.93 %
Plasticity Index= 16.31

Trial 3

A) Liquid Limit

Depth Sample	1.5-4.0m		
No. of Blows	14	26	30
Can No.	A	B	C
Mass of wet soil + Can, gm	164.54	168.56	166.113
Mass of Dry soil + Can, gm	154.23	154.87	154.6
Mass of Can, gm	134.936	128.25	131.98
Mass of water, gm	10.31	13.69	11.513
Mass of Dry Soil, gm	19.294	26.62	22.62
Moisture Content, W %	53.4363	51.4275	50.89744
From the flow curve			
LL=	51.35		



B) Plastic Limit

Depth Sample	1.5-4.0m	
Can No.	A	B
Mass of wet soil + Can, gm	99.15	97.83
Mass of Dry soil + Can, gm	93.955	92.873
Mass of Can, gm	78.918	79.352
Mass of water, gm	5.195	4.957
Mass of Dry Soil, gm	15.037	13.521
Moisture Content, W %	34.55	36.66
Plastic limit (%)	35.60	

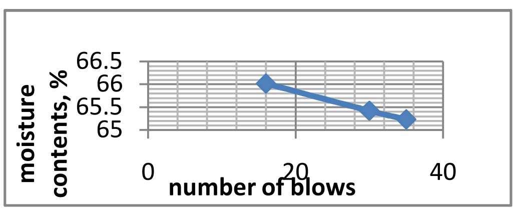
Liquid Limit= **51.35** **%**
Plastic Limit= **35.60** **%**
Plasticity Index= **15.75**

DEDO SITE

Trial 1

A) Liquid Limit

Depth Sample	1.5-4.0m		
No. of Blows	16	30	35
Can No.	A	B	C
Mass of wet soil + Can, gm	167.71	160.71	147.54
Mass of Dry soil + Can, gm	154.15	149.15	143.75
Mass of Can, gm	133.61	131.48	137.94
Mass of water, gm	13.56	11.56	3.79
Mass of Dry Soil, gm	20.54	17.67	5.81
Moisture Content, W %	66.01753	65.42162	65.23236
From the flow curve			
LL=	65.4		



B) Plastic Limit

Depth Sample	1.5-4.0m	
Can No.	A	B
Mass of wet soil + Can, gm	100.13	117.78
Mass of Dry soil + Can, gm	94.15	106.67
Mass of Can, gm	81.32	82.84
Mass of water, gm	5.98	11.11
Mass of Dry Soil, gm	12.83	23.83
Moisture Content, W %	46.61	46.62
Plastic limit (%)	46.62	

Liquid Limit= **<u>65.4</u>** **%**
Plastic Limit= **<u>46.2</u>** **%**
Plasticity Index= **<u>18.8</u>**

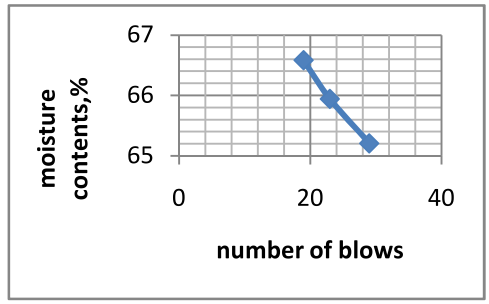
Trial 2

A) Liquid Limit

Depth Sample	1.5-4.0m		
No. of Blows	19	23	29
Can No.	A	B	C
Mass of wet soil + Can, gm	167.24	167.64	163.93
Mass of Dry soil + Can, gm	153.65	154.22	151.54
Mass of Can, gm	133.24	133.87	132.54
Mass of water, gm	13.59	13.42	12.39
Mass of Dry Soil, gm	20.41	20.35	19
Moisture Content, W %	66.585	65.9459	65.21052632
From the flow curve			
LL=	65.9		

B) Plastic Limit

Depth Sample	1.5-4.0m	
Can No.	A	B
Mass of wet soil + Can, gm	99.15	100
Mass of Dry soil + Can, gm	92.75	93.54
Mass of Can, gm	78.918	79.352
Mass of water, gm	6.4	6.46
Mass of Dry Soil, gm	13.832	14.188
Moisture Content, W %	46.27	45.53
Plastic limit (%)	45.90	

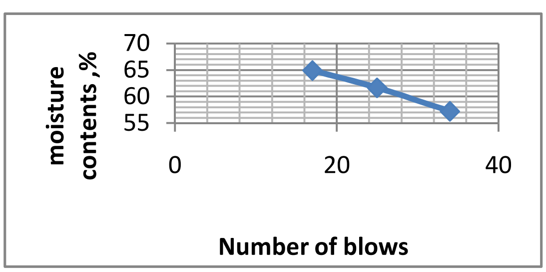


Liquid Limit= 65.9 %
Plastic Limit= 45.90 %
Plasticity Index= 20.00

Trial 3

A) Liquid Limit

Depth Sample	1.5-4.0m		
No. of Blows	17	25	34
Can No.	A	B	C
Mass of wet soil + Can, gm	134.43	127.34	149.19
Mass of Dry soil + Can, gm	114.79	110.93	124.95
Mass of Can, gm	84.52	84.31	82.59
Mass of water, gm	19.64	16.41	24.24
Mass of Dry Soil, gm	30.27	26.62	42.36
Moisture Content, W %	64.88272	61.64538	57.2238
From the flow curve			
LL=	61.75		



B) Plastic Limit

Depth Sample	1.5-4.0m	
Can No.	A	B
Mass of wet soil + Can, gm	105.33	99.84
Mass of Dry soil + Can, gm	97.33	93.32
Mass of Can, gm	81.21	79.352
Mass of water, gm	8	6.52
Mass of Dry Soil, gm	16.12	13.968
Moisture Content, W %	49.63	46.68
Plastic limit (%)	48.15	

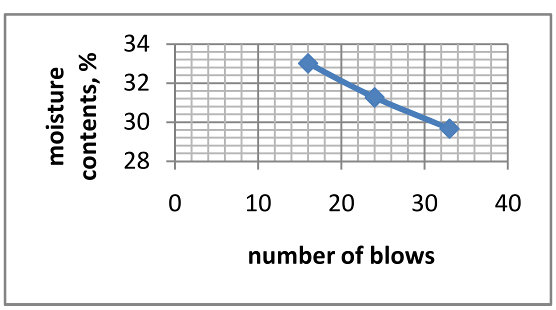
Liquid Limit= **61.75** **%**
Plastic Limit= **48.15** **%**
Plasticity Index= **13.60**

SEKA SITE

Trial 1

A) Liquid Limit

Depth Sample	1.5-4.0m		
No. of Blows	16	24	33
Can No.	A	B	C
Mass of wet soil + Can, gm	172.41	117.01	124.68
Mass of Dry soil + Can, gm	162.95	108.74	117.36
Mass of Can, gm	134.29	82.29	83.89.
Mass of water, gm	9.46	8.27	7.32
Mass of Dry Soil, gm	28.66	26.45	31.41
Moisture Content, W %	33.00768	31.26654	29.67
From the flow curve			
LL=	31.31		



B) Plastic Limit

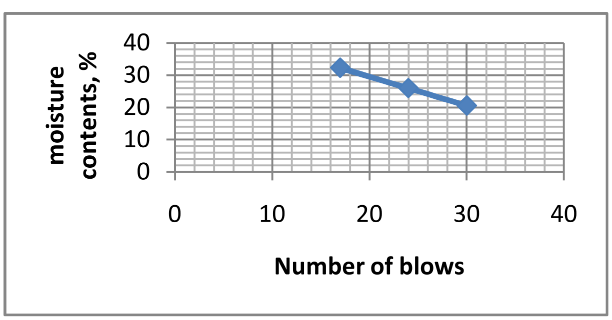
Depth Sample	1.5-4.0m	
Can No.	A	B
Mass of wet soil + Can, gm	160.95	98.22
Mass of Dry soil + Can, gm	155.84	94.78
Mass of Can, gm	132.75	79.352
Mass of water, gm	5.11	3.44
Mass of Dry Soil, gm	23.09	15.428
Moisture Content, W %	22.13	22.30
Plastic limit (%)	22.21	

Liquid Limit= **31.31** **%**
Plastic Limit= **22.21** **%**
Plasticity Index= **9.10**

Trial 2

A) Liquid Limit

Depth Sample	1.5-4.0m		
No. of Blows	17	24	30
Can No.	A	B	C
Mass of wet soil + Can, gm	203.21	198.81	198.95
Mass of Dry soil + Can, gm	187.85	186.53	187.63
Mass of Can, gm	140.25	139.25	132.54
Mass of water, gm	15.36	12.28	11.32
Mass of Dry Soil, gm	47.6	47.28	55.09
Moisture Content, W %	32.2689	25.9729	20.54819386
From the flow curve			
LL=	24.25		



B) Plastic Limit

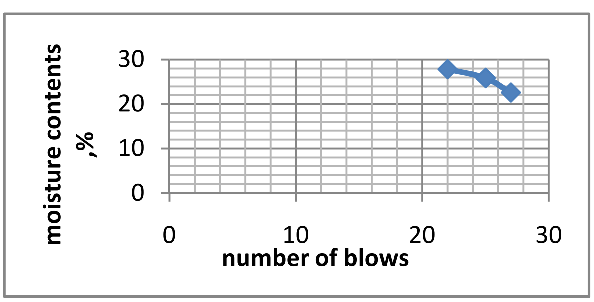
Depth Sample	1.5-4.0m	
Can No.	A	B
Mass of wet soil + Can, gm	101.2	102.62
Mass of Dry soil + Can, gm	97.17	98.34
Mass of Can, gm	78.918	79.352
Mass of water, gm	4.03	4.28
Mass of Dry Soil, gm	18.252	18.988
Moisture Content, W %	22.08	22.54
Plastic limit (%)	22.31	

Liquid Limit= **<u>24.25</u>** **%**
Plastic Limit= **<u>22.31</u>** **%**
Plasticity Index= **<u>1.94</u>**

Trial 3

A) Liquid Limit

Depth Sample	1.5-4.0m		
No. of Blows	22	25	27
Can No.	A	B	C
Mass of wet soil + Can, gm	198.45	199.79	202.21
Mass of Dry soil + Can, gm	184.25	186.25	189.38
Mass of Can, gm	133.24	133.87	132.54
Mass of water, gm	14.2	13.54	12.83
Mass of Dry Soil, gm	51.01	52.38	56.84
Moisture Content, W %	27.83768	25.84956	22.57213
From the flow curve			
LL=	25.22		



B) Plastic Limit

Depth Sample	1.5-4.0m	
Can No.	A	B
Mass of wet soil + Can, gm	103	102.82
Mass of Dry soil + Can, gm	98.65	98.52
Mass of Can, gm	78.918	79.352
Mass of water, gm	4.35	4.3
Mass of Dry Soil, gm	19.732	19.168
Moisture Content, W %	22.05	22.43
Plastic limit (%)	22.24	

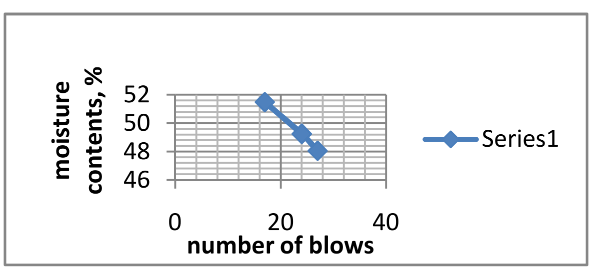
Liquid Limit= 25.22 %
Plastic Limit= 22.24 %
Plasticity Index= 2.98

JEREN SITE

Trial 1

A) Liquid Limit

Depth Sample	1.5-4.0m		
No. of Blows	17	24	27
Can No.	A	B	C
Mass of wet soil + Can, gm	122.05	126.29	129.14
Mass of Dry soil + Can, gm	108.27	111.88	114.66
Mass of Can, gm	81.5	82.62	84.53
Mass of water, gm	13.78	14.41	14.48
Mass of Dry Soil, gm	26.77	29.26	30.13
Moisture Content, W %	51.47553	49.24812	48.05841
From the flow curve			
LL=	48.85		



B) Plastic Limit

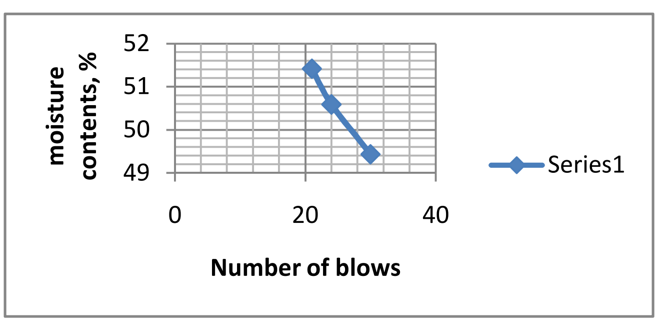
Depth Sample	1.5-4.0m	
Can No.	A	B
Mass of wet soil + Can, gm	160.23	98.22
Mass of Dry soil + Can, gm	151.53	92.46
Mass of Can, gm	131.53	79.352
Mass of water, gm	8.7	5.76
Mass of Dry Soil, gm	20	13.108
Moisture Content, W %	43.50	43.94
Plastic limit (%)	43.72	

Liquid Limit= **48.85** **%**
Plastic Limit= **43.72** **%**
Plasticity Index= **5.13**

Trial 2

A) Liquid Limit

Depth Sample	1.5-4.0m		
No. of Blows	21	24	30
Can No.	A	B	C
Mass of wet soil + Can, gm	203.21	201.44	198.95
Mass of Dry soil + Can, gm	179.45	178.74	176.98
Mass of Can, gm	133.24	133.87	132.54
Mass of water, gm	23.76	22.7	21.97
Mass of Dry Soil, gm	46.21	44.87	44.44
Moisture Content, W %	51.4174	50.5906	49.43744374
From the flow curve			
LL=	50.4		



B) Plastic Limit

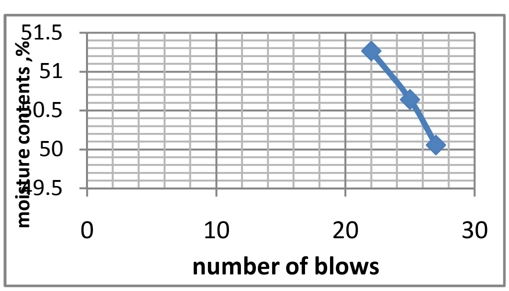
Depth Sample	1.5-4.0m	
Can No.	A	B
Mass of wet soil + Can, gm	101.2	102.62
Mass of Dry soil + Can, gm	94.47	95.64
Mass of Can, gm	78.918	79.352
Mass of water, gm	6.73	6.98
Mass of Dry Soil, gm	15.552	16.288
Moisture Content, W %	43.27	42.85
Plastic limit (%)	43.06	

Liquid Limit= 50.4 %
Plastic Limit= 43.06 %
Plasticity Index= 7.34

Trial 3

A) Liquid Limit

Depth Sample	1.5-4.0m		
No. of Blows	22	25	27
Can No.	A	B	C
Mass of wet soil + Can, gm	198.45	199.79	202.21
Mass of Dry soil + Can, gm	176.35	177.63	178.97
Mass of Can, gm	133.24	133.87	132.54
Mass of water, gm	22.1	22.16	23.24
Mass of Dry Soil, gm	43.11	43.76	46.43
Moisture Content, W %	51.26421	50.63985	50.05384
From the flow curve			
LL=	50.6		



B) Plastic Limit

Depth Sample	1.5-4.0m	
Can No.	A	B
Mass of wet soil + Can, gm	103	102.82
Mass of Dry soil + Can, gm	95.65	95.62
Mass of Can, gm	78.918	79.352
Mass of water, gm	7.35	7.2
Mass of Dry Soil, gm	16.732	16.268
Moisture Content, W %	43.93	44.26
Plastic limit (%)	44.09	

Liquid Limit= **50.6** **%**
Plastic Limit= **44.09** **%**
Plasticity Index= **6.51**

APPENDIX 2

CBR SETO SITE

	SETO SITE							
MDD (g/cc)	1.92		OMC (%)	16			REF No:	

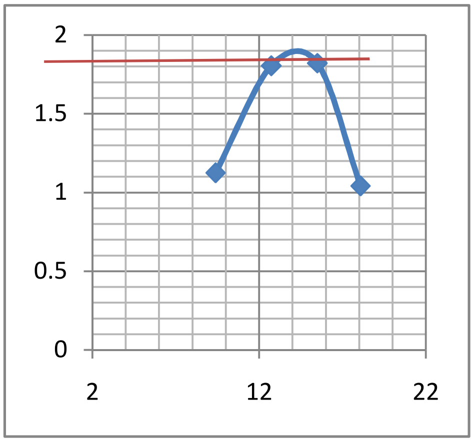
DENSITY DETERMINATION						
SOAKING CONDITION	**10 Blows**		**30 Blows**		**65 Blows**	
	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
MOLD NUMBER	9	9	M1	M1	M96	M96
WEIGHT OF SOIL + MOLD (gm)	9705	10376	10139	10763	10741	11273
WEIGHT OF MOLD (gm)	6390	6390	6543	6543	6831	6831
WEIGHT OF SOIL (gm)	3315	3986	3596	4220	3910	4442
VOLUME OF MOLD (cc)	2116	2116	2116	2116	2116	2116
WET DENSITY OF SOIL(g/cc)	1.57	1.88	1.70	1.99	1.85	2.10
DRY DENSITY OF SOIL (g/cc)	1.38	1.41	1.49	1.46	1.62	1.57

moisture determination									
soaking condition	**10 blows**			**30 blows**			**65 blows**		
	before	after		before	after		before	after	
			avg.		top 1 in.	avg.		top 1 in.	avg
CONTAINER NUMBER	10	7A		A6	O		4	1	
WET SOIL + CONTAINER (gm)	240.7	226.7		230.4	229.9		234.5	226.7	
DRY SOIL + CONTAINER (gm)	218	181.9		208.7	188.6		211.6	181.9	
WEIGHT OF WATER (gm)	22.7	44.8		21.7	50.2		22.9	44.8	
WEIGHT OF CONTAINER (gm)	50.3	50.3		54.30	50.20		49.70	50.30	
WEIGHT OF DRY SOIL (gm)	167.7	131.6		154.4	138.4		161.9	131.6	
MOISTURE CONTENT (%)	13.54	34.04		14.05	36.27		14.14	34.04	
AVG. MOISTURE CONTENT (%)									

RingFact= 17.18N/div

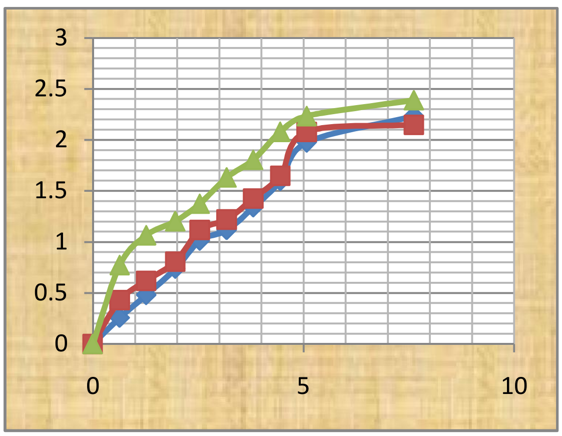
PENETRATION TEST DATA												
PENETRATION (mm)	**10 Blows**				**30 Blows**				**65 Blows**			
	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	**CBR %**	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	**CBR %**	DIAL RDG	LOAD (kn)	COR LOAD (kn)	**CBR %**
0	0	0	0		0	0			0	0		
0.64	15	0.2577			25	0			45	0.7731		
1.27	28	0.48104			36	1			62	1.06516		
1.96	43	0.73874			47	1			70	1.2026		
2.54	59	1.01362	1.01	7.6	65	1	1.12	8.4	80	1.3744	1.37	10.3
3.18	65	1.1167			71	1			95	1.6321		
3.81	78	1.34004			83	1			105	1.8039		
4.45	93	1.59774			96	2			121	2.07878		
5.08	115	1.9757	1.98	9.9	121	2	2.08	10.4	130	2.2334	2.23	11.2
7.62	130	2.2334			125	2			139	2.38802		
10.16		0				0				0		

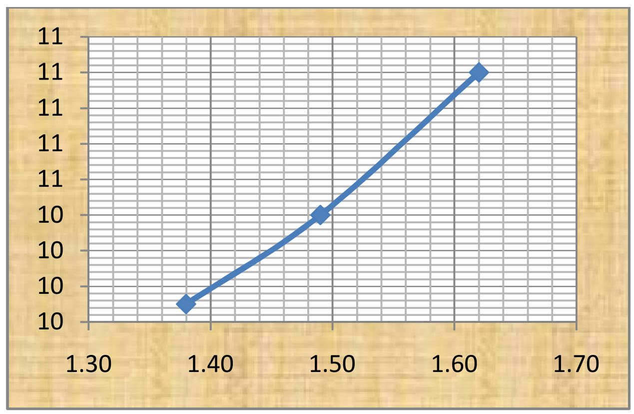
SWELL			
No. OF BLOWS	**10**	**30**	**65**
RDG (BEFORE SOAKING)	56.00	68.00	57.00
RDG (AFTER) SOAKING)	63.00	71.00	75.00
PERCENT SWELL	6.01	2.58	15.46
AVERAGE PERCENT SWELL :	8.02		



MODIFIED PROCTOR : T 180			
	MDD (g/cc) :		1.62
	OMC (%) :		15.1
	95% of MDD(g/cc)		1.54

	Before Soaking		After Soaking	
Blows	DD (g/cc)	Moisture (%)	DD (g/cc)	Moisture (%)
10	1.38	13.54	1.41	34.04
30	1.49	14.05	1.46	36.27
65	1.62	14.14	1.57	34.04





Blow	**LOAD (KN)**		**CBR(%)**		**Swell %**
	2.54mm	5.08mm	2.54mm	5.08mm	
10	1.01	1.98	7.6	10	2.47
30	1.12	2.08	8.4	10	2.13
65	1.37	2.23	10.3	11	1.68

Blow	**Dry density**	**CBR %**
10	1.38	10
30	1.49	10
65	1.62	11
CBR at 95 %		**9.82**

			DEDO SITE					
MDD (g/cc)	1.53		OMC (%)	10.4			REF No:	

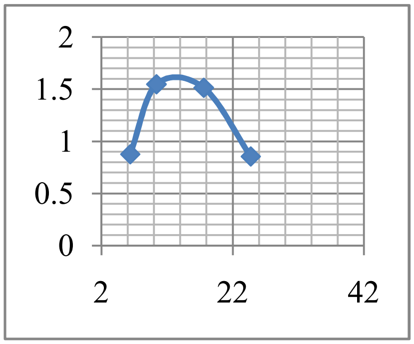
DENSITY DETERMINATION						
SOAKING CONDITION	**10 Blows**		**30 Blows**		**65 Blows**	
	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
MOLD NUMBER	M10	M10	95	95	53	53
WEIGHT OF SOIL + MOLD (gm)	9526	10975	9857	10588	10045	10313
WEIGHT OF MOLD (gm)	6302	6302	6357	6357	6583	6583
WEIGHT OF SOIL (gm)	3224	4673	3500	4231	3462	3730
VOLUME OF MOLD (cc)	2116	2116	2116	2116	2116	2116
WET DENSITY OF SOIL (g/cc)	1.52	2.21	1.65	2.00	1.64	1.76
DRY DENSITY OF SOIL (g/cc)	1.32	1.62	1.43	1.37	1.42	1.20

MOISTURE DETERMINATION									
SOAKING CONDITION	**10 Blows**			**30 Blows**			**65 Blows**		
	BEFORE	AFTER		BEFORE	AFTER		BEFORE	AFTER	
			AVG.		TOP 1 in.	AVG.		TOP 1 in.	AVG.
CONTAINER NUMBER	10	7A		A6	O		4	1	
WET SOIL + CONTAINER (gm)	229.7	215.5		245.5	234.3		217.5	233.5	
DRY SOIL + CONTAINER (gm)	205.8	171.9		219.75	176.6		195.5	175.2	
WEIGHT OF WATER (gm)	23.9	43.6		25.8	57.7		22.0	58.3	
WEIGHT OF CONTAINER (gm)	51.9	51.1		51.80	51.00		51.10	51.80	
WEIGHT OF DRY SOIL (gm)	153.9	120.8		167.95	125.6		144.4	123.4	
MOISTURE CONTENT (%)	15.54	36.09		15.33	45.94		15.24	47.24	
AVG. MOISTURE CONTENT (%)									

RingFactor=
17.18N/div

PENETRATION TEST DATA												
PENETRATION (mm)	**10 Blows**				**30 Blows**				**65 Blows**			
	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	**CBR %**	DIAL RDG	LOAD (kn)	COR.LOAD (kn))	**CBR %**	DIAL RDG	LOAD (kn)	COR LOAD (kn))	**CBR %**
0	0	0	0		0	0			0	0		
0.64	9	0.15462			11	0.19			15	0.2577		
1.27	12	0.20616			15	0.26			19	0.32642		
1.96	16	0.27488			19	0.33			25	0.4295		
2.54	21	0.36078	0.36	2.7	26	0.45	0.45	3.3	37	0.63566	0.64	4.8
3.18	23	0.39514			28	0.48			41	0.70438		
3.81	26	0.44668			32	0.55			46	0.79028		
4.45	28	0.48104			35	0.6			49	0.84182		
5.08	32	0.54976	0.55	2.8	38	0.65	0.65	3.3	53	0.91054	0.91	4.6
7.62	35	0.6013			43	0.74			57	0.97926		
10.16		0				0				0		

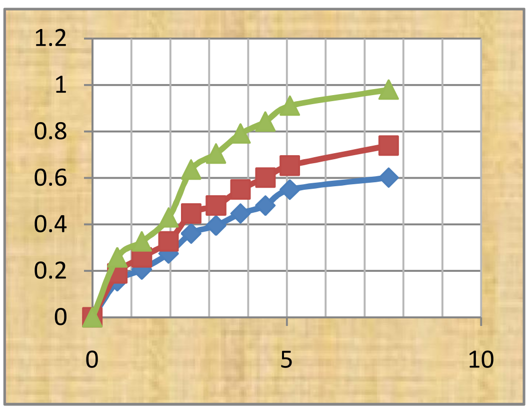
SWELL			
No. OF BLOWS	**10**	**30**	**65**
RDG (BEFORE SOAKING)	35.00	40.00	50.00
RDG (AFTER) SOAKING)	85.00	84.00	98.00
PERCENT SWELL	42.96	37.80	41.24
AVERAGE PERCENT SWELL :		40.66	

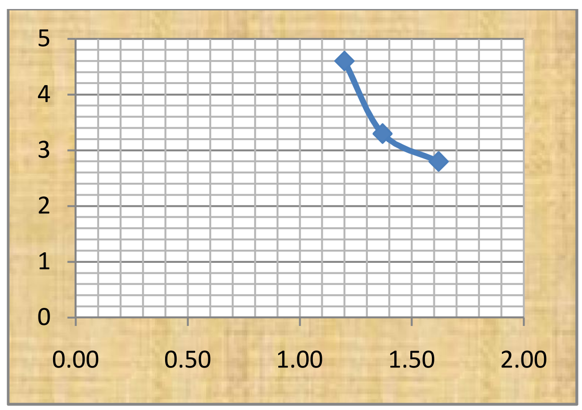


MOISTURE CONTENT

MODIFIED PROCTOR : T 180			
	MDD (g/cc) :		1.51
	OMC (%) :		5.49
	95% of MDD(g/cc)		1.43

	Before Soaking		After Soaking	
Blows	DD (g/cc)	Moisture (%)	DD (g/cc)	Moisture (%)
10	1.32	15.54	1.62	36.09
30	1.47	15.33	1.37	45.94
65	1.42	15.24	1.20	47.24





Blow	LOAD (KN)		CBR(%)		Swell %
	2.54mm	5.08mm	2.54mm	5.08mm	
10	0.36	0.55	2.7	3	6.01
30	0.45	0.65	3.3	3	2.58
65	0.64	0.91	4.8	5	15.46

Blow	**Dry density**	**CBR %**
10	1.62	3
30	1.37	3
65	1.20	5
CBR at 95 %		**3.48**

	SEKA SITE							
MD (g/c)	1.92		OMC (%)	17.2			REF No:	

DENSITY DETERMINATION						
SOAKING CONDITION	10 Blows		30 Blows		65 Blows	
	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
MOLD NUMBER	3	3	8	8	C8	C8
WEIGHT OF SOIL + MOLD (gm)	11063	11121	10874	10923	10796	10848
WEIGHT OF MOLD (gm)	6562	6562	6332	6332	6209	6209
WEIGHT OF SOIL (gm)	4501	4559	4542	4591	4587	4639
VOLUME OF MOLD (cc)	2116	2116	2116	2116	2116	2116
WET DENSITY OF SOIL (g/cc)	2.13	2.15	2.15	2.17	2.17	2.19
DRY DENSITY OF SOIL (g/cc)	1.81	1.75	2.15	1.78	1.78	1.85

MOISTURE DETERMINATION			
SOAKING CONDITION	**10 Blows**	**30 Blows**	**65 Blows**

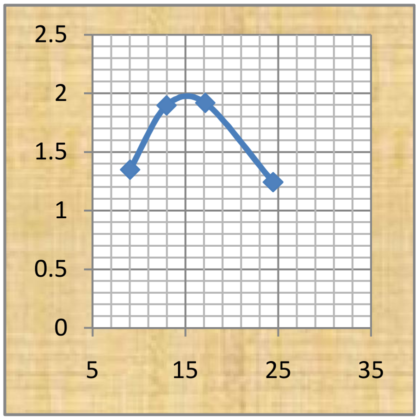
	BEFORE	AFTER		BEFORE	AFTER		BEFORE	AFTER	
			AVG.		TOP 1 in.	AVG.		TOP 1 in.	AVG.
CONTAINER NUMBER	9	6		3A	5		A5	2	
WET SOIL + CONTAINER (gm)	236	267		261.8	230.6		238.4	245.4	
DRY SOIL + CONTAINER (gm)	209	227		230.7	198.5		205.2	215.2	
WEIGHT OF WATER (gm)	27.6	40.8			32.1		33.2	30.2	
WEIGHT OF CONTAINER (gm)	51.5	50.3		50.30	51.10		51.20	51.50	
WEIGHT OF DRY SOIL (gm)	157	176			147.4		154	163.7	
MOISTURE CONTENT (%)	17.55	23.14			21.78		21.56	18.45	
AVG. MOISTURE CONTENT (%)									

Factor =17.18N/div

PENETRATION TEST DATA

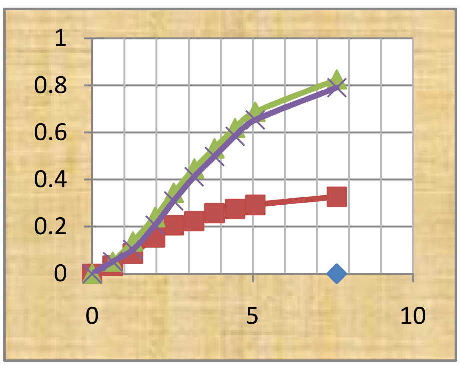
PENETRATION (mm)	**10 Blows**				**30 Blows**				**65 Blows**			
	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	**CBR %**	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	**CBR %**	DIAL RDG	LOAD (kn)	COR LOAD (kn))	**CBR %**
0	0	0	0		0	0			0	0		
0.64	45	0.773			50	0.86			58	0.99644		
1.27	56	0.962			61	1.05			66	1.13388		
1.96	64	1.1			76	1.31			74	1.27132		
2.54	75	1.289	1.29	9.7	83	1.43	1.43	10.7	86	1.47748	1.48	11.1
3.18	79	1.357			92	1.58			91	1.56338		
3.81	80	1.374			107	1.84			105	1.8039		
4.45	95	1.632			115	1.98			113	1.94134		
5.08	115	1.976	1.98	9.9	126	2.16	2.16	10.9	131	2.25058	2.25	11.3
7.62	126	2.165			134	2.3			140	2.4052		
10.16		0				0				0		

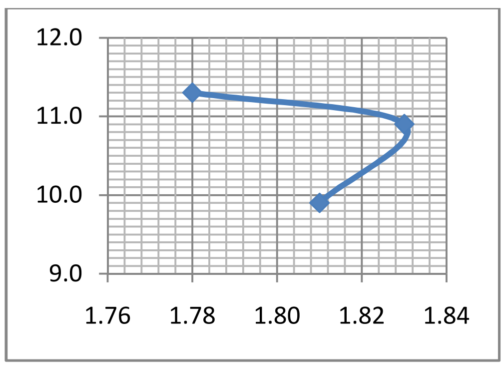
SWELL			
No. OF BLOWS	**10**	**30**	**65**
RDG (BEFORE SOAKING)	69.00	89.00	40.00
RDG (AFTER) SOAKING)	91.00	106.00	69.00
PERCENT SWELL	18.90	14.60	24.91
AVERAGE PERCENT SWELL :		19.47	



MODIFIED PROCTOR : T 180			
	MDD (g/cc) :		1.91
	OMC (%) :		17.72
	96% of MDD(g/cc)		1.83

	Before Soaking		After Soaking	
Blows	DD (g/cc)	Moisture (%)	DD (g/cc)	Moisture (%)
10	1.81	17.55	1.83	23.14
30	1.83	17.24	1.78	21.78
65	1.78	1.85	1.85	15.82





Blow	**LOAD (KN)**		**CBR(%)**		**Swell %**
	2.54mm	5.08mm	2.54mm	5.08mm	
10	1.01	1.98	7.6	10	2.47
30	1.12	2.08	8.4	10	2.13
65	1.37	2.23	10.3	11	1.68

Blow	**Dry density**	**CBR %**
10	1.38	10
30	1.49	10
65	1.62	11
CBR at 95 %		**9.82**

	JEREN SITE							
MDD (g/cc)	1.89		OMC (%)	12.6			REF No:	

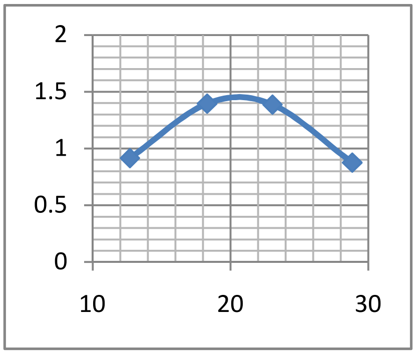
DENSITY DETERMINATION						
SOAKING CONDITION	10 Blows		30 Blows		65 Blows	
	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
MOLD NUMBER	3	3	8	8	C8	C8
WEIGHT OF SOIL + MOLD (gm)	10079	10103	10413	10443	10449	10482
WEIGHT OF MOLD (gm)	6098	6098	6465	6465	6491	6491
WEIGHT OF SOIL (gm)	3981	4005	3948	3978	3958	3991
VOLUME OF MOLD (cc)	2116	2116	2116	2116	2116	2116
WET DENSITY OF SOIL (g/cc)	1.88	1.89	1.87	1.88	1.87	1.89
DRY DENSITY OF SOIL (g/cc)	1.59	1.33	1.35	1.33	1.34	1.33

MOISTURE DETERMINATION								
SOAKING CONDITION	10 Blows			30 Blows			65 Blows	
	BEFORE	AFTER		BEFORE	AFTER		BEFORE	AFTER	
			AVG.		TOP 1 in.	AVG.		TOP 1 in.	AVG.
CONTAINER NUMBER	9	6		3A	5		A5	2	
WET SOIL + CONTAINER (gm)	221.1	234.2		275.5	249.2		220.2	240.4	
DRY SOIL + CONTAINER (gm)	195.1	179.5		213.3	191.9		173	184.8	
WEIGHT OF WATER (gm)	26.0	54.7		62.2	57.3		47.2	55.6	
WEIGHT OF CONTAINER (gm)	52.2	51.4		51.30	51.90		52.90	50.20	
WEIGHT OF DRY SOIL (gm)	142.9	128.1		162	140		120.1	134.6	
MOISTURE CONTENT (%)	18.19	42.70		38.40	40.93		39.30	41.31	
AVG. MOISTURE CONTENT (%)									

Ring Factor = 17.18N/div

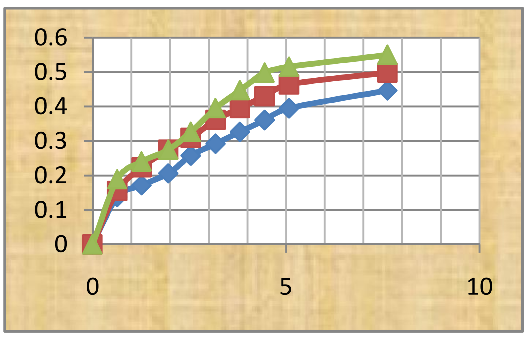
PENETRATION TEST DATA												
PENETRATION	**10 Blows**				**30 Blows**				**65 Blows**			
(mm)	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	**CBR %**	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	**CBR %**	DIAL RDG	LOAD (kn)	COR LOAD (kn))	**CBR %**
0	0	0	0		0	0			0	0		
0.64	8	0.13744			9	0.1546			11	0.18898		
1.27	10	0.1718			13	0.2233			14	0.24052		
1.96	12	0.20616			16	0.2749			16	0.27488		
2.54	15	0.2577	0.26	1.9	18	0.3092	0.31	2.3	19	0.32642	0.33	2.4
3.18	17	0.29206			21	0.3608			23	0.39514		
3.81	19	0.32642			23	0.3951			26	0.44668		
4.45	21	0.36078			25	0.4295			29	0.49822		
5.08	23	0.39514	0.40	2.0	27	0.4639	0.46	2.3	30	0.5154	0.52	2.6
7.62	26	0.44668			29	0.4982			32	0.54976		
10.16		0				0			0	0		
12.7												

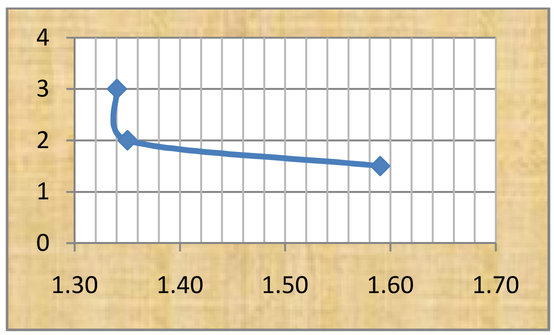
SWELL			
No. OF BLOWS	**10**	**30**	**65**
RDG (BEFORE SOAKING)	54.00	69.00	59.00
RDG (AFTER) SOAKING)	54.00	69.00	62.00
PERCENT SWELL	0.00	0.00	2.58
AVERAGE PERCENT SWELL :	0.86		



MODIFIED PROCTOR : T 180			
	MDD (g/cc) :		1.49
	OMC (%) :		26.25
	95% of MDD(g/cc)		1.42

	Before Soaking		After Soaking	
Blows	DD (g/cc)	Moisture (%)	DD (g/cc)	Moisture (%)
10	1.59	18.19	1.33	42.70
30	1.35	38.40	1.25	40.93
65	1.34	39.30	1.20	41.31





Blow	**LOAD (KN)**		**CBR(%)**		**Swell %**
	2.54mm	5.08mm	2.54mm	5.08mm	
10	0.26	0.40	1.9	2	2.47
30	0.31	0.46	2.3	2	2.13
65	0.33	0.52	2.4	3	1.68

Blow	**Dry density**	**CBR %**
10	1.59	2
30	1.35	2
65	1.34	3
CBR at 95 %		**2.22**

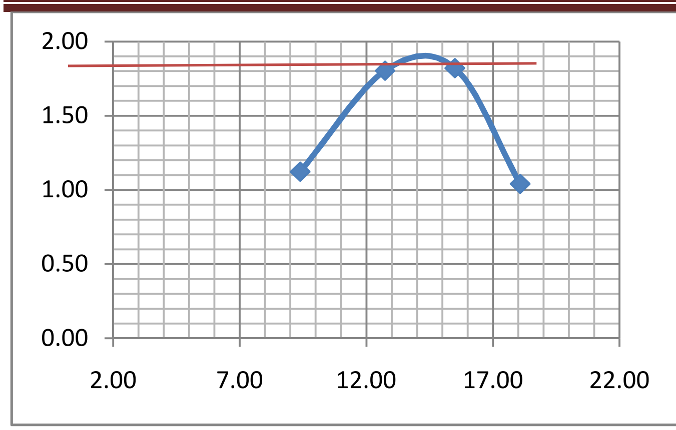
APPENDIX 3

SETO SITE

PROCTER COMPACTION

DENSITY	VISUAL DESCRIPTION **ENBANKMENT**		DATE TESTED			
	PURPOSE		SOURCE STATION			
	TRIAL NUMBER	1	2	3	4	5
	WEIGHT OF SOIL + MOLD g	8500	10120	10260	8500	
	WEIGHT OF MOLD g	6025	6025	6025	6025	
	WEIGHT OF SOIL g	2,475	4,095	4,235	2,475	
	VOLUME OF MOLD cc	2014	2014	2014	2014	
	WET DENSITY OF SOIL g/cc	1.23	2.03	2.10	1.23	
MOISTURE	CONTAINER NUMBER	5	3	10	4	
	WET SOIL + CONTAINER g	330.61	188.12	445.55	206.94	
	DRY SOIL + CONTAINER g	318.62	173.14	400.11	185.33	
	WEIGHT OF WATER g	11.99	14.98	45.44	21.61	
	WEIGHT OF CONTAINER g	190.95	55.6	107.02	75.25	
	WEIGHT OF DRY SOIL g	127.67	117.59	293.09	119.5	
	MOISTURE CONTENT %	9.39	12.74	15.50	18.08	
DRY DENSITY OF SOIL g/cc		1.12	1.80	1.82	1.04	

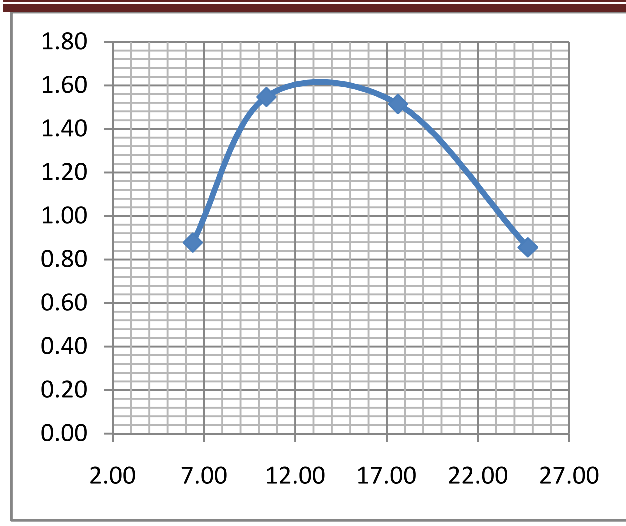
	MDD :	1.82	%
	OMC :	15.5	%



Dedo site

	VISUAL DESCRIPTION		DATE TESTED		
	PURPOSE		SOURCE STATION		45KM
DENSITY	TRIAL NUMBER	1	2	3	4
	WEIGHT OF SOIL + MOLD g	7900	9460	9605	8170
	WEIGHT OF MOLD g	6020	6020	6020	6020
	WEIGHT OF SOIL g	1,880	3,440	3,585	2,150
	VOLUME OF MOLD cc	2014	2014	2014	2014
	WET DENSITY OF SOIL g/cc	0.93	1.71	1.78	1.07
MOISTURE	CONTAINER NUMBER	1	6	8	4
	WET SOIL + CONTAINER g	249.9	161.82	233..30	170.89
	DRY SOIL + CONTAINER g	238.84	148.87	215.08	145.92
	WEIGHT OF WATER g	11.06	12.95	19.22	24.97
	WEIGHT OF CONTAINER g	65.62	24.5	105.96	45.00
	WEIGHT OF DRY SOIL g	173.22	124.42	109.12	100.92
	MOISTURE CONTENT %	6.38	10.41	17.61	24.74
DRY DENSITY OF SOIL g/cc		0.88	1.55	1.51	0.86

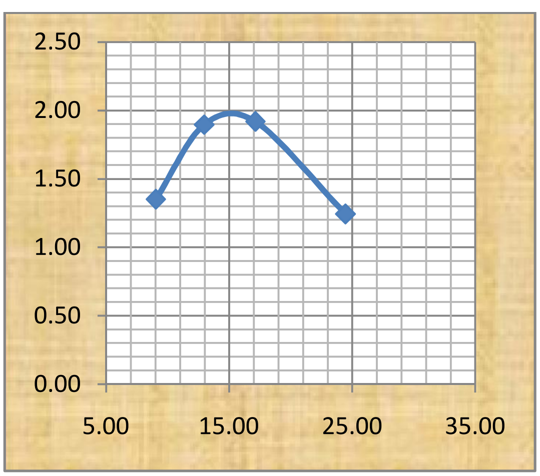
	MDD :	1.55 g/cc
	OMC :	10.4 %


1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
2.00
7.00
12.00
17.00
22.00
27.00

Seka site

	VISUAL DESCRIPTION			DATE TESTED			
	PURPOSE			SOURCE STATION		70+000 LHS	
	TRIAL NUMBER		1	2	3	4	5
DENSITY	WEIGHT OF SOIL + MOLD	g	9100	10450	10660	9250	
	WEIGHT OF MOLD	g	6135	6135	6135	6135	
	WEIGHT OF SOIL	g	2,965	4,315	4,525	3,115	
	VOLUME OF MOLD	cc	2014	2014	2014	2014	
	WET DENSITY OF SOIL	g/cc	1.47	2.14	2.25	1.55	
MOISTURE	CONTAINER NUMBER		C5	1	3A	2	
	WET SOIL + CONTAINER	g	332.83	320.24	482.04	316.19	
	DRY SOIL + CONTAINER	g	316.13	299.03	434.32	275.69	
	WEIGHT OF WATER	g	16.7	21.21	47.72	40.5	
	WEIGHT OF CONTAINER	g	131.41	135.6	155.72	110.12	
	WEIGHT OF DRY SOIL	g	184.72	163.4	278.60	165.57	
	MOISTURE CONTENT	**%**	9.04	12.98	17.13	24.46	
DRY DENSITY OF SOIL		**g/cm^3**	1.35	1.90	1.92	1.24	

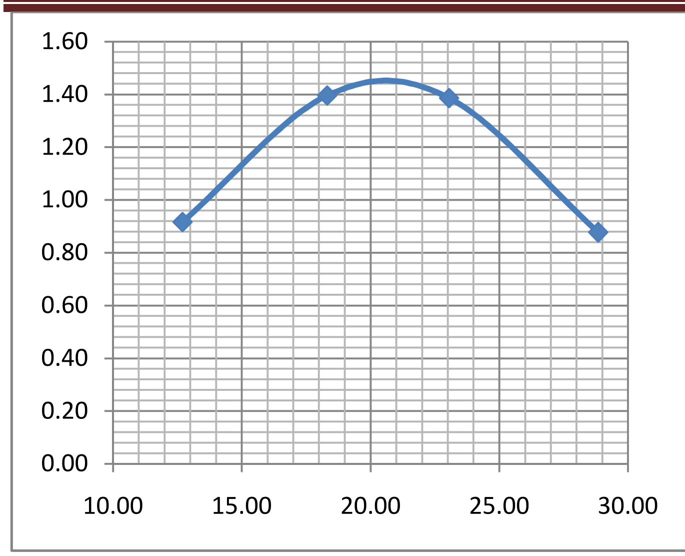
	MDD :	1.92	g/cm^3
	OMC :	17..13	%

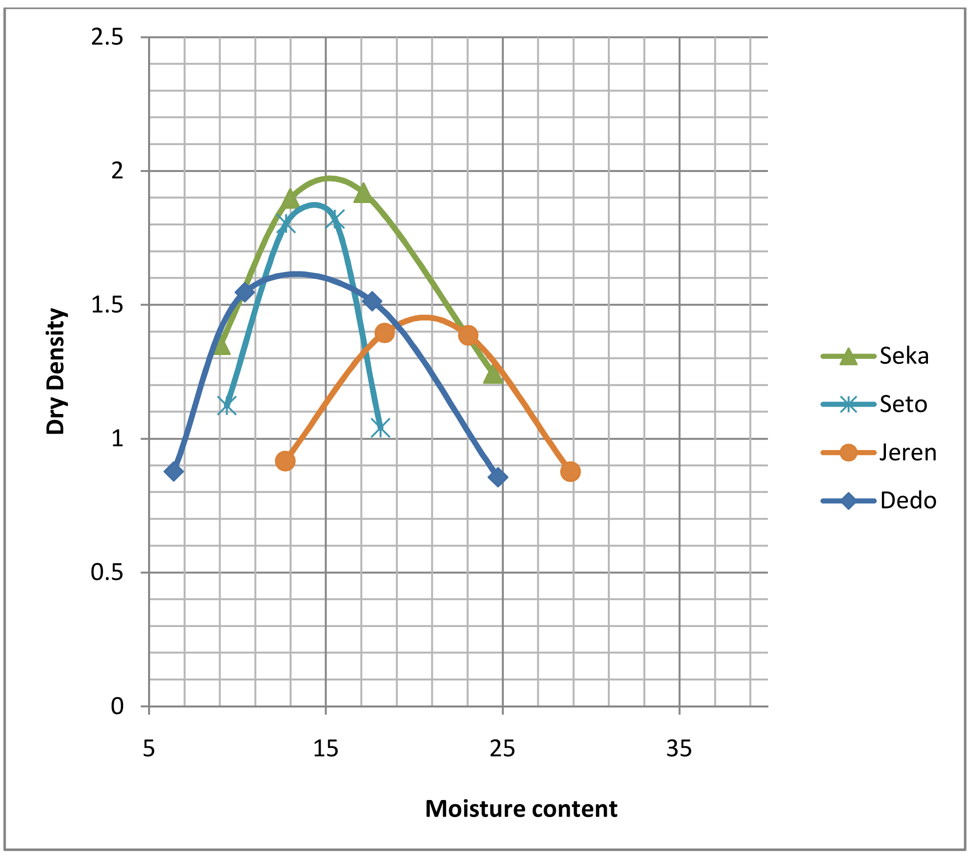


Jerensite

	VISUAL DESCRIPTION		DATE TESTED		
	PURPOSE		SOURCE STATION		
	TRIAL NUMBER	1	2	3	4
DENSITY	WEIGHT OF SOIL + MOLD g	8105	9350	9460	8300
	WEIGHT OF MOLD g	6025	6025	6025	6025
	WEIGHT OF SOIL g	2,080	3,325	3,435	2,275
	VOLUME OF MOLD cc	2014	2014	2014	2014
	WET DENSITY OF SOIL g/cc	1.03	1.65	1.71	1.13
MOISTURE	CONTAINER NUMBER	6	5	C4	4
	WET SOIL + CONTAINER g	280.79	322.56	326.42	335.21.
	DRY SOIL + CONTAINER g	264.88	294.5	290.08	307.61
	WEIGHT OF WATER g	15.91	28.06	36.34	54.6
	WEIGHT OF CONTAINER g	139.62	141.31	132.42	118.27
	WEIGHT OF DRY SOIL g	125.26	153.19	157.66	189.34
	MOISTURE CONTENT %	12.70	18.32	23.05	28.84
DRY DENSITY OF SOIL g/cc		0.92	1.40	1.39	0.88

	MDD : 1.4 g/cc
	OMC : 18.3 %


1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
10.00
15.00
20.00
25.00
30.00


Dry Density
2.5
2
1.5
1
0.5
0
5
15
25
35
Moisture content
Seka
Seto
Jeren
Dedo

APPENDIX 4

DYNAMIC CONPENETERATION TEST ANALYSES

SETOSITE

Tril1

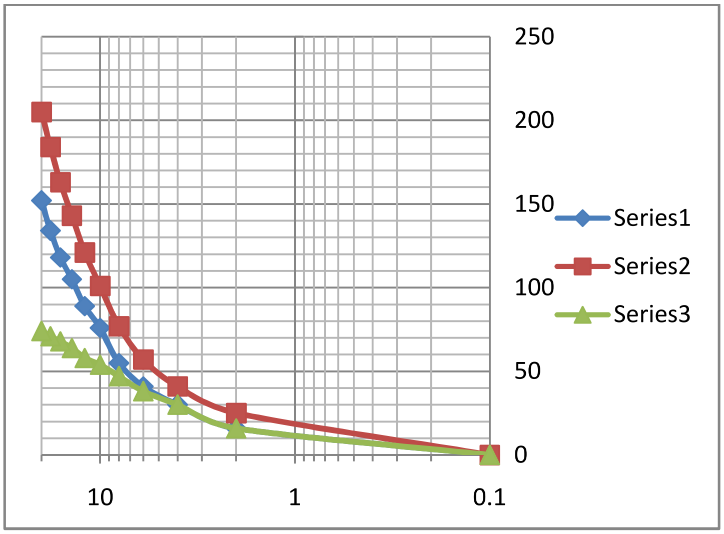
NO	TOTAL	READING
BLOWS	BLOWS	MM
0	0	0
2	2	16
2	4	30
2	6	41
2	8	55
2	10	76
2	12	89
2	14	105
2	16	118
2	18	134
2	20	152

Tril2

NO OF	TOTAL	READING
BLOWS	BLOWS	MM
0	0	0
2	2	25
2	4	41
2	6	57
2	8	77
2	10	101
2	12	121
2	14	143
2	16	163
2	18	184
2	20	205

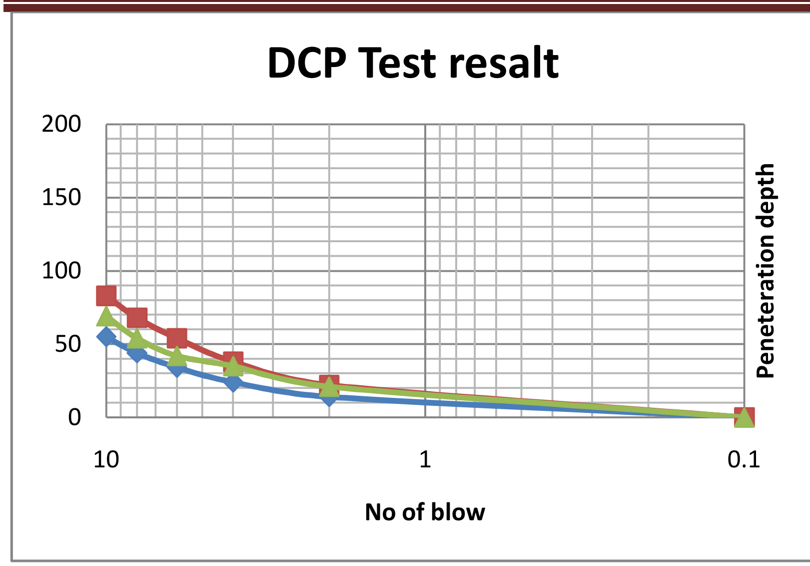
Tril 3

NOOF	TOTAL	READING
BLOWS	BLOWS	MM
0	0	0
2	2	16
2	4	30
2	6	38
2	8	47
2	10	54
2	12	58
2	14	64
2	16	68
2	18	71
2	20	74



seka site

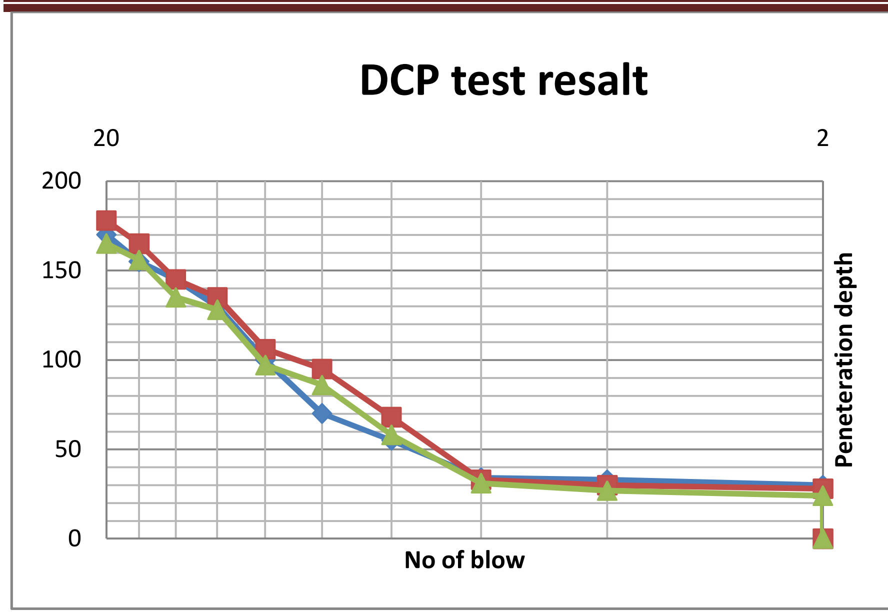
Tril 1			Tril 2			Tril 3		
NO	TOTAL	READING	NO OF	TOTAL	READING	NOOF	TOTAL	READING
BLOWS	BLOWS	MM	BLOWS	BLOWS	MM	BLOWS	BLOWS	MM
0	0	0	0	0	0	0	0	0
2	2	14	2	2	22	2	2	21
2	4	24	2	4	38	2	4	35
2	6	34	2	6	54	2	6	42
2	8	44	2	8	68	2	8	54
2	10	55	2	10	83	2	10	69
2	12	67	2	12	98	2	12	83
2	14	75	2	14	112	2	14	100
2	16	87	2	16	127	2	16	115
2	18	102	2	18	141	2	18	131
2	20	116	2	20	155	2	20	145
2	22	137	2	22	171	2	22	156
2	24	147	2	24	188	2	24	186



Dedo site

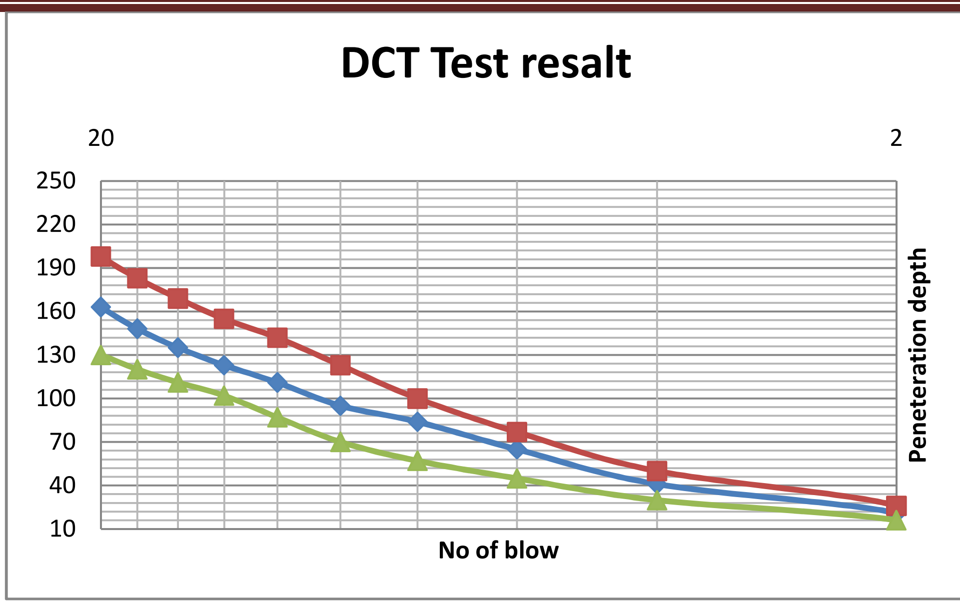
Tril 1 | Tril 2 | Tril 3

NO	TOTAL	READING	NO OF	TOTAL	READING	NOOF	TOTAL	READING
BLOWS	BLOWS	MM	BLOWS	BLOWS	MM	BLOWS	BLOWS	MM
0	0	0	0	0	0	0	0	0
2	2	30	2	2	28	2	2	24
2	4	33	2	4	30	2	4	27
2	6	34	2	6	33	2	6	31
2	8	55	2	8	68	2	8	58
2	10	70	2	10	95	2	10	86
2	12	100	2	12	106	2	12	97
2	14	130	2	14	135	2	14	128
2	16	145	2	16	145	2	16	135
2	18	155	2	18	165	2	18	156
2	20	170	2	20	178	2	20	165
2	22	175						
2	24	180						



Jeren site

Tril 1			Tril 2			Tril 3		
NO	**TOTAL**	**READING**	**NO OF**	**TOTAL**	**READING**	**NOOF**	**TOTAL**	**READING**
BLOWS	BLOWS	MM	BLOWS	BLOWS	MM	BLOWS	BLOWS	MM
0	0	0	0	0	0	0	0	0
2	2	21	2	2	26	2	2	16
2	4	41	2	4	50	2	4	30
2	6	65	2	6	77	2	6	45
2	8	84	2	8	100	2	8	57
2	10	95	2	10	123	2	10	70
2	12	111	2	12	142	2	12	87
2	14	123	2	14	155	2	14	102
2	16	135	2	16	169	2	16	111
2	18	148	2	18	183	2	18	120
2	20	163	2	20	198	2	20	130
2	22	179	2	22	215	2	22	137
2	24	196	2	24	230	2	24	147


DCT Test resalt
20
2
250
220
190
160
130
100
70
40
10
Peneteration depth
No of blow

APPENDIX 5

Determination of coefficient of permeability
Falling Head test
SETO SITE

Initial head of water =	186	cm
Final head of water =	181.3	cm
Duration of test =	4152	s
Sample length =	11	cm
Sample diameter =	8.2	cm
Stand-pipe diameter =	0.58	cm

Area of stand-pipe =	0.26	cm^2
Area of sample =	52.81	cm^2
Coefficient of permeability, k =	**3.39E-07**	cm/s

DEDO SITE

Initial head of water =	186	cm
Final head of water =	180.8	cm
Duration of test =	3618.2	s
Sample length =	11	cm
Sample diameter =	8.2	cm
Stand-pipe diameter =	0.58	cm

Area of stand-pipe =	0.26	cm^2
Area of sample =	52.81	cm^2
Coefficient of permeability, k =	**4.31E-07**	cm/s

SEKE SITE

Initial head of water =	186	cm
Final head of water =	176.6	cm
Duration of test =	1735	s
Sample length =	11	cm
Sample diameter =	8.2	cm
Stand-pipe diameter =	0.58	cm

Area of stand-pipe =	0.26	cm^2
Area of sample =	52.81	cm^2
Coefficient of permeability, k =	**1.64E-06**	cm/s

JEREN SITE

Initial head of water =	186	cm
Final head of water =	144	cm
Duration of test =	5400.03	s
Sample length =	11	cm
Sample diameter =	8.2	cm
Stand-pipe diameter =	0.58	cm

Area of stand-pipe =	0.26	cm^2
Area of sample =	52.81	cm^2
Coefficient of permeability, k =	**2.61E-06**	cm/s

APPENDIX 6

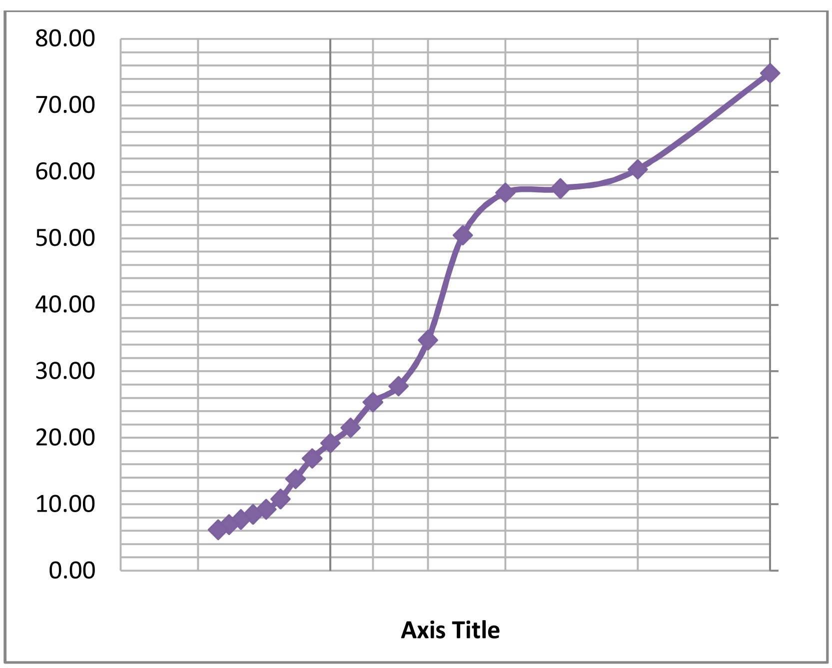
DETERMINATION OF GRAIN SIZE DISTRIBUTION

SETO SITE

Sample mass,

M_o : 5000g

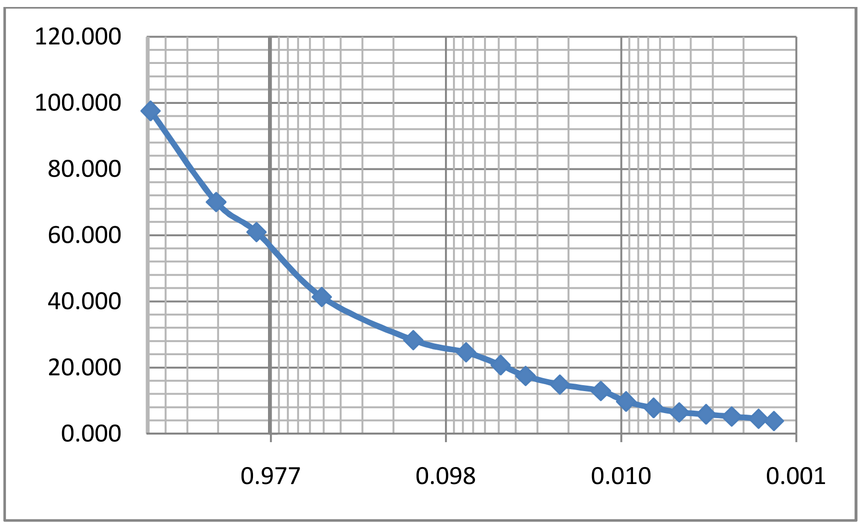
Sive Size(mm)	Wt Retained(kg)	Percentage retained	Cummulative percentage retained	Percentage passing
4.75	2.525	50.50	50.50	49.50
2. 0	0.725	14.50	65.00	35.00
1.18	0.145	2.90	67.90	32.10
1	0.03	0.60	68.50	31.50
0.5	0.32	6.40	74.90	25.10
0.15	0.79	15.80	90.70	9.30
0.075	0.345	6.90	97.60	2.40
pan	0.12	2.40	100.00	0.00
Total mass	**5**	kg		
From plot				



DEDO SITE

sample mass5000gm(depth)

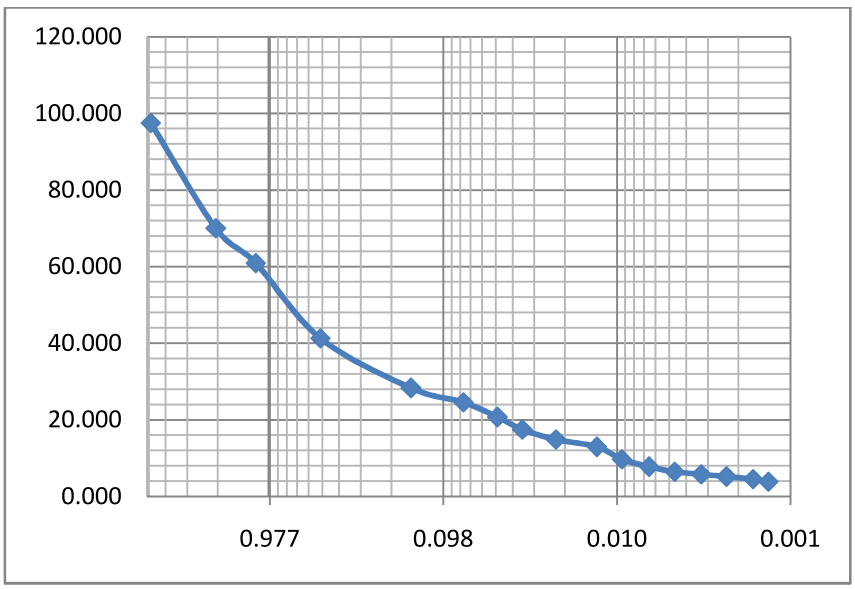
Sive Size(mm)	Percentage retained	Percentage retained	Cummulative percentage retained	Percentage passing
4.75	0.875	17.50	17.50	82.50
2.00	1.66	33.20	50.70	49.30
1.18	0.455	9.10	59.80	40.20
0.5	0.98	19.60	79.40	20.60
0.15	0.65	13.00	92.40	7.60
0.075	0.185	3.70	96.10	3.90
Pan	0.195	3.90	100.00	
Total mass	**5**	gm		



JERENSITE

sample mass 5000gm(depth)

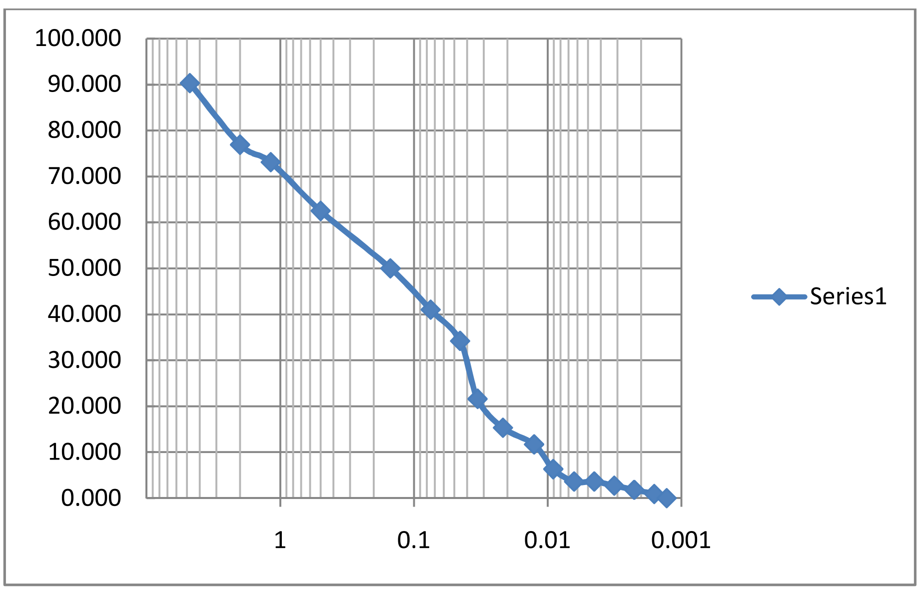
Sive Size(mm)	Wt Retained(gm)	Percentage retained	Cummulative percentage retained	Percentage passing
4.75	1.395	27.90	27.90	72.10
2.00	0.725	14.50	42.40	57.60
1.18	0.85	17.00	59.40	40.60
0.5	0.41	8.20	67.60	32.40
0.15	0.72	14.40	82.00	18.00
0.075	0.25	5.00	87.00	13.00
Pan	0.65	13.00	100.00	
Total mass	**5**	gm		

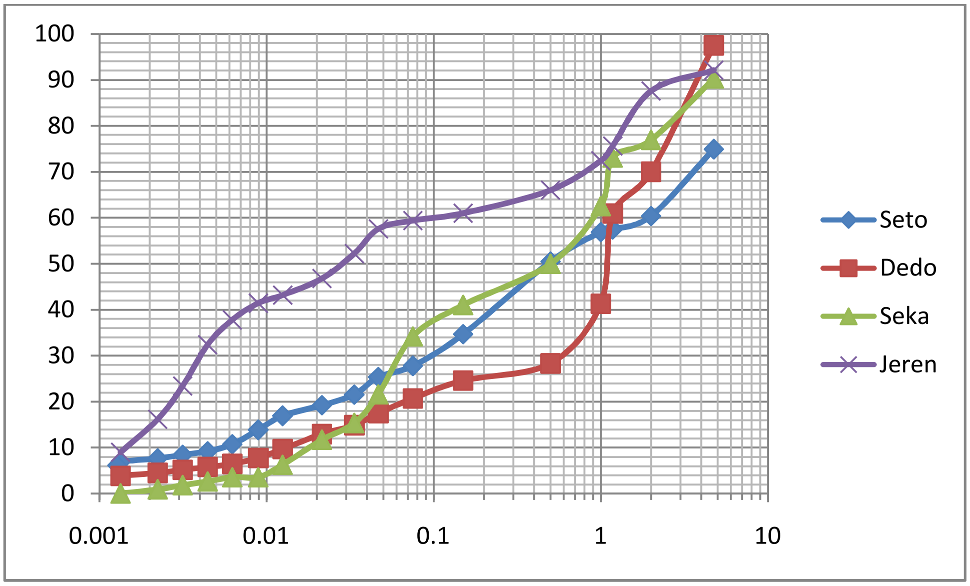


SEKASITE

sample mass 5000gm(depth)

Sive Size(mm)	Wt Retained(gm)	Percentage retained	Cummulative percentage retained	Percentage passing
4.75	1.485	29.70	29.70	70.30
2.00	1.17	23.40	53.10	46.90
1.18	0.19	3.80	56.90	43.10
0.5	0.53	10.60	67.50	32.50
0.15	0.625	12.50	80.00	20.00
0.075	0.45	9.00	89.00	11.00
Pan	0.55	11.00	100.00	
Total mass	**5**	gm		




100
90
80
70
60
50
40
30
20
10
0
0.001
0.01
0.1
1
10
Seto
Dedo
Seka
Jeren