Jimma University

School of Graduate Studies

Jimma Institute of Technology

Faculty of Civil and Environmental Engineering

Geotechnical Engineering Stream

Suitability assessment of selected material as subbase for road construction along Mazoriya to Limmu Genet in Limmu genet wereda quarry sites .

(A case study in Jimma zone)

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

By:

Habte Tamirat

March, 2018

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March, 2018

Jimma, Ethiopia

DECLARATION

I, the undersigned declare that this thesis entitled ‘‘Suitability assessment of selected material as subbase for road construction along Mazoriya to Limmu Genet in Limmu genet wereda quarry sites’’ is my original work and has not been presented by any other person for an award of a degree in this or any other University and all sources of material used for these have been duly acknowledged.

Candidate:

Habte Tamirat

Signature________________ Date_______________

ABSTRACT

Subbase is one of the unbound materials which enable traffic stresses to be reduced to acceptable levels in the subgrade. It acts as a working platform for the construction of the upper pavement layers, and it acts as a separation layer between the subgrade and base course. Selection of good quality materials for subbase is one of the problems facing during road construction, and such material should adhere the minimum requirements of the standard specifications. This study had been focused in assessing or identifying the suitability of the selected material as the subbase for road construction along mazoriya to limmu genet of Jimma zone. In the study area, it is predominantly covered with wide kilometer square of coffee beans which is the most exported product in Ethiopia (e.g., Horizon coffee plant) and some agricultural products. Three quarry sites were identified. Representative disturbed samples from the quarry sites of Limmu genet, Babu, and Ambuye (i.e. designated by site one, site two and site three, respectively) were collected from different parts. Laboratory tests of specific gravity, natural moisture content, Atterberg limits, Grain size analysis, compaction test, CBR strength test and in-situ test of unit weight by Sand cone replacement method, were conducted. Based on the laboratory tests results of the grain size distribution from the three quarry sites were gravelly soils. The gravel content and sand content have taken the significant percentage of the grain size analysis of the samples. Also, laboratory test results showed natural moisture content ranges from 10.95% to 40.45%. The average specific gravity of site one, site two and site three is 2.66, 2.58 and 2.68 respectively. This means the specific gravity of site three is the highest, and that of site two is the lowest specific gravity. It implies the construction of subbase layer by soil sample of site two is more prone to erosion and scour. Likewise, the CBR value of laboratory test of site one, site two and site three is 40%, 24% and 34% respectively. The sites in their previous order have a swelling of 0.00%-0.20%, 0.43%-0.50% and 0.26%-0.37% for 65 blows, 30 blows and 10 blows of compaction per layer *of CBR test. This implies that site one has less swelling than others. Therefore, it can be concluded that site one sample is more suitable for subbase material than the others.*

Keywords: Sub-base, Engineering properties of soil, CBR test and Swelling.

ACKNOWLEDGMENT

All Praise and thanks be to God almighty, the most gracious and the most merciful, for giving me the ability, the health, and the strength to finish this study.

I would like to express my deepest appreciation and thanks to my advisors, Prof. Emer T. Quezon, and Eng. Tadese Abebe for their endless encouragement, and valuable and continuous guidance through the preparation of this thesis and for the time in proofing the manuscript.

I also acknowledge JIT staffs and China Construction Corporation for allowing me to conduct the Laboratory tests by making suitable arrangement and also for their assistance in conducting the test in the laboratories.

Lastly, but not in the sense of the least, I would like to express my sincere thanks to my family for their constant encouragement, and moral support without which this study would not be finished.

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SYMBOLS AND ABBREVIATIONS

AASHTO	American Association of State Highway and Transport Officials
ASTM	American Society for Testing and Materials
CBR	California bearing ratio
C_C	Coefficient of curvature
C_u	Uniformity coefficient
D_{ii}	Diameter corresponding to percent's finer
DCP	Dynamic cone penetrometer
DCPI	Dynamic cone penetrometer index
ERA	Ethiopian Road Authority
GB	Granular base course
GC	Granular capping layer
GI	Group Index
GS	Granular sub-base
G_S	Specific gravity
K	Modulus of subgrade or subbase reaction
LL	Liquid Limit
MDD	Maximum dry density
NMC	Natural moisture content
OMC	Optimum moisture content
PI	Plasticity Index
PL	Plastic Limit
TRL	UK Transport Research Laboratory
USACE	United State Army Corps of Engineers
W_C	Water content
γ	Wet unit weight
γ_d	Dry unit weight

CHAPTER ONE

INTRODUCTION

1.1 Background

Limmu Genet is one of the towns in Southwestern Ethiopia, Oromia National Regional State. It is predominantly covered with huge kilometer square of coffee beans which is the most exported product in Ethiopia (e.g., Horizon coffee plant) and some agricultural products. It is true that the production of exported materials only is not enough for the growth of one town. Thus, the road that connects Limmu Genet wereda to Jimma town is until now serving with exhausting unbound materials cover defects which would be the major source of problems now and after the growth of this road to a pavement road. So assessing the strength and the engineering properties of these unbound materials is vital.

The performance of a pavement depends on the quality of its subgrade and subbase layers. As the foundation for the pavement's upper layers, the subgrade and subbase layers play a key role in mitigating the detrimental effects of climate and the static and dynamic stresses generated by traffic. Therefore, building a stable subgrade and a properly drained subbase is vital for constructing an effective and long-lasting pavement system (Vernon R. Schaefer et al. 2008).

It is known that some of the most important characteristics affecting the unbound aggregate materials are mineralogy, Particle size distribution (grading) and fines content, Particle shape, surface texture, and angularity and durability (soundness, abrasion resistance).These characteristics play out during construction by affecting the workability of the mixture and controlling the degree of compaction (density) and pore structure of the layer. These, in turn, impact the layer strength, stability (resistance to deformation), and modulus (stiffness) properties that are relevant to performance and design (Tutumluer, 2013).

The subgrade, the layer of soil on which the subbase or pavement is built, provides support to the remainder of the pavement system. It is crucial for highway engineers to develop a subgrade with a California Bearing Ratio (CBR) value of at least 10. Research has shown that if a subgrade has a CBR value less than 10, the subbase material will deflect under traffic loadings in the same manner as the subgrade and cause pavement deterioration.

The subbase, the layer of aggregate material immediately below the pavement, provides drainage and stability to the pavement. Undrained water in the pavement supporting layers can freeze and expand, creating high internal pressures on the pavement structure. Moreover, flowing water can carry soil particles that clog drains and, in combination with traffic, pump fines from the subbase or subgrade. It is therefore crucial that highway engineers develop a stable, permeable subbase with longitudinal subdrains (Vernon R. Schaefer et al. 2008).

In addition to stability and drainage requirements, the subgrade and subbase must be designed and constructed to exhibit a high level of spatial uniformity, measured using geotechnical engineering parameters such as shear strength, stiffness, volumetric stability, and permeability. Several environmental variables, such as temperature and moisture, must also be taken into account since these variables have both short-term and long-term effects on the geotechnical characteristics.

1.2 Statement of the Problem

The Nigerian Engineer Ajayi (1987) showed that from his field observations and laboratory experiments, road failures can arise from inadequate knowledge of the geotechnical characteristics and behavior of residual soils on which the roads are built and non-recognition of the influence of geology and geomorphology during the design and construction phases. Thus the treatment of troublesome materials like clays are not been considered by the construction engineers which may be problematic.

The required specifications for sub-base and road-base, given in the Technical Specification booklet of different road projects, are described by the following items: grading, Atterberg, limits, CBR, aggregate crushing value (ACV), ten percent fines value, bulk specific gravity, water absorption or swelling (Islam, R; 2004).

Harischandra (2004) found that potholes, cracks, edge defects, depressions and corrugation are significant road defects observed in the field. At the same time he emphasized that traffic, age, road geometry, weather, drainage, construction quality as well as construction material, maintenance policy play the major role as road deteriorate agents.

Those above potholes, poorly graded gravel and drainage problems are observed on the road this research was conducted. If through appropriate assessment or identification and gradation modification of the subbase materials is permitted, the problems can be mitigated for the future.

1.3 Objectives

1.3.1 General objective

The general objective of this study is to assess or identify the suitability of the selected material as the subbase for road construction along Mazoriya to Limmu Genet in Limmu genet wereda quarry sites.

1.3.2 Specific objectives

Based on the above general objective, the following key points were identified as the specific objectives of the study.

1) To determine and discuss the different engineering properties of the selected subbase materials
2) To evaluate the suitability of selected materials from the quarry sites by comparing test results of the different soil types which could be used as subbase material
3) To identify and discuss the factors that affect or change the behavior of the subbase material using laboratory aided tests.

1.4 Research question

Based on the above specific objectives; it can be drawn the following important research questions:

- What are the different engineering properties of the selected materials for subbase course preparation?
- Which site is suitable and how much deviation in values of the selected subbase materials from the standard specification?
- What are the factors or parameters that affect or change the behavior of the subbase materials?

1.5 Significance of the study

The research shall investigate some of the engineering properties and the strength of the excellent material for subbase quarry sites found in Limmu Genet wereda. In this wereda, there are many expansion projects of road construction and buildings.

Transverse cracking, potholes, surface defects, rutting, and drainage problems are among common road defect (problems) in which the main cause is the selection of qualified subgrade and sub-base material during construction. In this time, the road of study area is serving the community with many potholes, poorly graded gravel and drainage problems which are the causes for the delay of transportation, costly construction budget, and maintenance for every year. Although there is not yet constructed pavement road in this area, potholes and drainage problem are some currently occurring defects of the road. If through appropriate assessment or identification and gradation modification of the subbase materials is permitted, the problems can be mitigated in upgrading the construction of road to a flexible pavement.

It is true that, there is not yet enough research is done in this study area concerning to unbound materials. Besides that, in developing countries like Ethiopia, this research plays a vital role in selection of a good quality of sub-base material and it also enhances the development of the study area.

1.6 Scope of the study

This study identified the selected material quarry sites as subbase material along Mazoriya to Limmu Genet which covers about 76km length. A representative disturbed sample is taken from the identified three quarry sites to investigate the index properties of the granular soils such as water content, plastic index (PI), specific gravity and grain size distribution. CBR and swelling test is conducted to identify the strength of a select material. Finally, result analysis and discussion, conclusion and recommendation are discussed.

1.7 Structure of the thesis

Chapter 1 gives an introduction to study area and unbound materials, address about statement of the problem, objective, significance and scope of this research.

Chapter 2 address a literature review on unbound materials especially about subbase materials and factors that affect the characteristics of subbase materials.

Chapter 3 discuss about the methodology of this research, laboratory test procedure.

Chapter 4 presents the laboratory test result, discussion and summary comparison of the result with Era standard.

Chapter 5 discuss about the conclusions and recommendation obtained from the three sample test result and other related things to the study.

CHAPTER TWO

REVIEW OF RELATED LITERATURE

2.1 General

2.1.1 Unbound pavement materials

According to Ethiopian Roads Authority manual (2002), the selection of unbound materials for use as the base course, sub-base, capping and selected subgrade layers, the materials categorized with a summary of their characteristics are shown in Table 2.1.

Table 2. 1 Properties of Unbound materials

Code	Description	Summary of Specification
GB1	Fresh, crushed rock	Dense-graded, unweathered crushed stone, on-plastic parent fines
GB2 GB3	Crushed weathered rock, gravel or boulders Natural coarsely graded granular a material, including processed and modified gravels	Dense grading, PI < 6, soil or parent fines Dense grading, PI < 6 CBR after soaking > 80
GS	Natural gravel	CBR after soaking > 30
GC	Gravel or gravel-soil	Dense graded; CBR after soaking > 15

Notes: 1) These specifications are sometimes modified according to site conditions; material type and principal use (see text).

2) GB = Granular base course, GS = Granular sub-base, GC = Granular capping layer.

2.1.2 Sub-Bases (GS):

The sub-base is an important load spreading the layer in the completed pavement. It enables traffic stresses to be reduced to acceptable levels in the subgrade, it acts as a working platform for the construction of the upper pavement layers, and it acts as a separation layer between the subgrade and base course. Under special circumstances, it may also act as a filter or as a drainage layer. In wet climatic conditions, the most stringent requirements are dictated by the need to support construction traffic and paving equipment. In these circumstances, the sub-base material needs to be more tightly specified. In dry climatic conditions, in areas of good drainage, and where the road surface remains well sealed, unsaturated moisture conditions prevail, and sub-base specifications may be relaxed. The selection of sub-base materials will, therefore, depend on the design function of the layer and the anticipated moisture regime, both in service and at construction (ERA Manual, 2002).

2.1.3 Bearing capacity of sub-base

A minimum CBR of 30 percent is required at the highest anticipated moisture content when compacted to the specified field density. Usually, a minimum of 95 percent of the maximum dry density achieved in the ASTM Test Method D 1557 (Heavy Compaction). Under conditions of good drainage and when the water table is not near the ground surface, the field moisture content under a sealed pavement will be equal to or less than the optimum moisture content in the ASTM Test Method D 698 (Light Compaction). In such conditions, the sub-base material should be tested in the laboratory in an unsaturated state. Except in arid areas, if the base course allows water to drain into the lower layers, as may occur with unsealed shoulders and under conditions of poor surface maintenance where the base course is previous, saturation of the sub-base is likely. In these circumstances, the bearing capacity should be determined on samples soaked in water for four days. The test should be conducted on samples prepared at the density and moisture content likely to be achieved in the field. To achieve the required bearing capacity, and for consistent support to be provided to the upper pavement, limits on soil plasticity and particle size distribution may be required. Materials which meet the recommendations of Tables 2.2 and 2.3 will usually be found to have an adequate bearing capacity (ERA Manual, 2002).

2.1.4 Use as a construction platform

In many circumstances, the requirements of a sub-base are governed by its ability to support construction traffic without excessive deformation or traveling. A high-quality sub-base is therefore required where loading or climatic conditions during construction are severe. The suitable material should possess properties similar to those of a good surfacing material for unpaved roads. The material should be well graded and have a plasticity index at the lower end of the appropriate range for an ideal unpaved road wearing course under the prevailing climatic conditions. These considerations form the basis of the criteria given in Tables 2.2 and 2.3. Material meeting the requirements for severe conditions will usually be of higher quality than the standard sub-base (GS). If materials to these requirements are unavailable, trafficking trials should be conducted to determine the performance of alternative materials under typical site conditions. In the construction of low-volume roads, where cost savings at construction are particularly important, local experience is often invaluable, and a wider range of materials may often be found to be acceptable (ERA Manual, 2002).

In Ethiopia, laterite soil is one of the widely available materials and can be used as a sub-base material. Laterite meeting the graduation requirements of Table 2.3 can be used for traffic levels up to 3x10^6 ESA provided the following criteria are satisfied:

Plasticity Index (%) < 25

Plasticity Modulus (PM) < 500

CBR (%) > 30

Table 2. 2 Recommended plasticity characteristics for Granular Sub-bases (GS) (ERA Manual, 2002).

Climate	Typical Annual Rainfall	Liquid Limit	Plasticity Index	Linear Shrinkage
Moist tropical and wet tropical	>500mm	<35	<6	<3
Seasonally wet tropical	>500mm	<45	<12	<6
Arid and semi-arid	<500mm	<55	<20	<10

Table 2. 3 Typical particle size distribution for Sub-base (GS) (ERA Manual, 2002).

Test Sieve (mm)	Percentage by mass of total aggregate Passing test sieve (%)
50	100
37.5	80-100
20	60-100
5	30-100
1.18	17-75
0.3	30-100
0.075	5-25

2.2 Soil grouping based on their genetic basis and soil forming factors

2.2.1 Soil grouping based on their genetic basis

It was found that most of the tropically weathered soils of Africa could be divided in to three groups on a genetic basis, determined by the soil-forming factors. The three major groups of significance have been defined by D'Hoore 1964, (Lyon, 1971). These are:

I) Ferruginous Soils: These occur in semi-arid to moist sub-humid conditions for lateritic soils, in areas with pronounced dry seasons. Ferruginous soils are common they are hard and durable. Marked separation of iron oxide is frequently observed which may be leached or precipitated with the profile. Kaolinite is the predominant clay mineral in this type. It requires an average annual rainfall of 600 to 1800mm for its formation.

ii) Ferallitic Soils: These occur in moist sub-humid to very humid areas for lateritic soils and in areas with dense vegetation cover. Gibbsite is the most common clay mineral observed and other hydrated forms of alumina occur as well as hydrated iron minerals. Halloysite is fairly common over volcanic rocks. The annual average rainfall requirement for its formation is 1500 to 4000mm. Both of the above soils have SiO_2/R_2O_2 ratio of less than 2.0 and are classified either as lateritic or laterite soils.

iii) Ferrisols: Those are formed over all types of rocks in intermediate to high rainfall areas where erosion has kept the place with profile development. They have similar profiles to ferallitic soils, but with few weatherable minerals remaining. The entire clay fraction comprises Kaolinite and amorphous oxides of iron and aluminum. These are developed at deeper levels due to the surface erosion, and occur in regions of annual average rainfall of 1250 to 2750mm. According to Morine W.J. and Todor P.C. (1976), Ethiopian laterites fall under this group (Blight, 1997).

2.2.2 Soil forming factors

The Factors of Soil Formation, in which Jenny (1941) sought mathematical expressions of soil formation, based on the variables he referred to as cl, o, r, p, and t (climate, organisms, relief, parent material, and time).

i. Parent Material

The initial material from which soils form is considered the parent material. In the case of Histosols (organic soils), the parent material is plant debris, but for most soils, it is mineral matter. The parent material may be solid rock that weathers in place to form soil, or it may be transported and deposited before having soils form in it. Transported parent materials include those deposited by running water (alluvium), gravity (colluvium), glaciers (till), wind (loess, aeolian sand), and volcanic eruptions (tephra).

ii. **Climate**

The influence of climate on soil formation is largely through the combined effects of water and temperature, although wind and solar radiation also play important roles.

iii. **Topography**

The shape of the land's surface, or topography, influences how water flows onto and off of the soil, as well as how it moves into and through the soil. Consequently, it exerts a strong control on the balance between soil organic matter additions and decomposition, erosion and deposition, leaching and accumulation, and even oxidation and reduction. Topography is formed by depositional features, such as lava flows, moraines, alluvial fans, and dunes, and by geologic erosion, such as stream incision, glaciation, and mass wasting.

iv. Organisms

Soils provide habitats for a multitude of organisms, and these plants, animals, and microbes strongly impact the soils as well. The integrated effects of organisms can be observed by comparing surface soil horizons developed under coniferous forest with those developed under grass. Much of the organic matter in coniferous forest soils is derived from foliage that falls from the trees.

v. Time

The environmental factors of climate, organisms, topography, and parent material must interact over a period of time to produce soil. The longer these factors are able to act together, the more developed and vertically differentiated and distinctive the soil will become. Thus, the stability of the soil landscape governs the duration of soil formation, and the character of the soil reflects how long the other environmental factors have exerted their influence.

2.3 Index properties of soils

2.3.1. Moisture contents

'Water content' or 'moisture content' of a soil has a direct bearing on its strength and stability. The water content of a soil in its natural state is termed its 'Natural moisture content', which characterizes its performance under the action of load and temperature. The water content may range from a trace quantity to that sufficient to saturate the soil or fill all the voids in it. If the trace moisture has been acquired by the soil by absorption from the atmosphere, then it is said to be 'hygroscopic moisture' (C.Venkartmain, 2006).

Vernon R. Schaefer et al. (2008) stated that number of subgrade properties, including loadbearing capacity, shrinkage, and swelling can be affected by moisture content. By itself moisture content can be influenced by a number of factors, such as drainage, groundwater table elevation, infiltration, or pavement porosity.

Regular water content used to express the consistency about mud soil previously, its characteristic state. Consistency is a term used to show that level about solidness of durable soils. The consistency from claiming regular durable clay is communicated qualitatively by such terms as exact soft, soft, stiff, precise firm furthermore diligent. Those physical properties from claiming clays extraordinarily contrast toward distinctive water substance. A clay which précised delicate in a higher rate of water content turns into Verwoerd diligent with diminish on water substance. However, it need been found that toward those same water content, two tests for clay from claiming diverse sources might have distinctive consistency. Clay might be generally delicate same time alternate might a chance to be hard. Further, A diminishing over water content might bring little impact for one example of clay in any case might change the opposite example starting with Just about An fluid should a firm state. Water substance alone, therefore, will be not a sufficient list for consistency to building furthermore large portions of other purposes (J. Jibril, 2014).

2.3.2 Atterberg limits

Atterberg Limits are conceptual boundaries between various states of real behavior involving mixes of soil particles and water. They were developed by Swedish scientist Dr. A. Atterberg in 1911 to classify agricultural soils. The limits are represented by water content values corresponding to specific observations of behavior (J.T Germaine 2009).

Figure 2.1 illustrates a scale of increasing water content along with the various material behaviors possible for a particulate system.

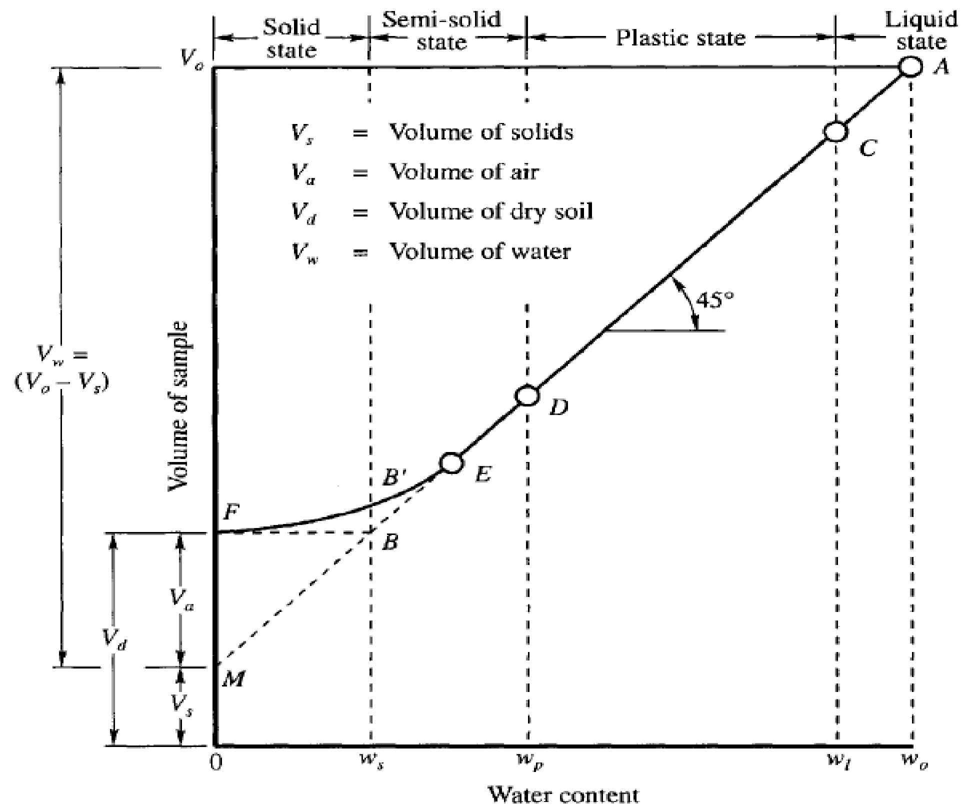

Figure 2. 1 Curve showing transition stages from the liquid to solid state (Murthy, 1994)

The scientist Albert Atterberg originally defined seven "limits of consistency" to classify fine-grained soils, but in current engineering practice only two of the limits, the liquid and plastic limits, are commonly used (A third limit, called the shrinkage limit, is occasionally used) (Jibril. J, 2014).

The limits are determined on specimens of remolded soils on the portion of particles finer than the 0.425 mm (No. 40) sieve (i.e., grains the size of fine sand and smaller). This is an arbitrary choice that makes processing the soil to eliminate the coarse particles relatively easy without including too many large, non-plastic particles in the measurement.

LL = Liquid Limit.

The liquid limit marks the boundary between plastic and fluid-like behavior. Once again, this is a clear state rather than an absolute boundary. At the water content corresponding to the liquid limit, the soil becomes fluid under a standard dynamic shear stress. ASTM D4318 also covers the determination of the liquid limit.

PL = Plastic Limit: The plastic limit marks the boundary between semi-solid and plastic mechanical behavior; however, in reality, the material slowly transitions between the two. At the water content corresponding to the plastic limit, the soil crumbles when rolled into a 3.2 mm (1/8 in.) diameter string. The plastic limit can be determined using ASTM D4318 Liquid Limit, Plastic Limit, and Plasticity Index of Soils.

SL = Shrinkage Limit

The shrinkage limit marks the boundary between a solid and a semi-solid. At this water content, the soil volume is at a minimum while maintaining a full state. In theory, the shrinkage limit is a well - defined condition, but it does depend to some extent on the first fabric of the material. The shrinkage limit can be determined by ASTM D4943 Shrinkage Factors of Soils by the Wax Method.

Plasticity Index (PI)

The plastic and liquid limit values are further used to define parameters referred to as the Plasticity Index (PI) and Liquidity Index (LI). The PI is the difference in water content between the liquid and plastic limits, as presented in Equation 9.1.

$$PI = LL - PL \tag{9.1}$$

Plasticity index PI indicates the degree of plasticity of soil. The greater the difference between liquid and plastic limits, the greater is the plasticity of the soil. The cohesionless soil has zero plasticity indexes. Such soils are termed as non-plastic. Soils possessing large values of LL and PI are said to be highly plastic or fat. Those with low values are described as slightly plastic or lean. Organic clays possess liquid limits greater than 50. The plastic limits of such soils are equally higher. Therefore soils with organic content have low plasticity indices corresponding to comparatively high liquid limits (J. Jibril, 2014). Atterberg classifies the soils according to their plasticity indices as in Table 2.4

Table 2. 4 Soil classifications according to plasticity index (Murthy, 1994)

Plasticity Index	Plasticity
0	Non- plastic
0<7	Low plastic
7<17	Medium plastic
>17	High plastic

2.3.3 Unit weight of the soils

The in-situ unit weight refers to the unit weight of soil in the undisturbed condition or of a compacted soil in-place. Determination of in-situ unit weight is made on borrow-pit soils to estimate the quantity of soil required for placing and compacting a certain fill or embankment. During the construction of compacted fills, it is standard practice to make the in-situ determination of a unit weight of the soil after it is placed to ensure that the compaction effort has been adequate (C.Venkatramaiah, 2006).

Important methods for the determination of the in-situ unit weight are being given:

(i) Sand-replacement method.

(ii) Rubber Balloon Method (ASTM Designation D-2167)

2.3.3.1 Sand-replacement Method

The principle of the sand replacement method consists in obtaining the volume of the soil excavated by filling in the hole in-situ from which it is excavated, with sand, previously calibrated for its unit weight, and after that determining the weight of the sand required to fill the hole.

The test procedure consists of calibration of the cylinder and sand and later, the measurement of the unit weight of the soil.

2.3.3.1.1 Calibration of the Cylinder and Sand:

This consists in obtaining the weight of sand required to fill the pouring cone of the cylinder and the bulk unit weight of the sand. Uniformly graded, dry, clean and is used. The cylinder is filled with sand almost to be top and the weight of the cylinder with the sand is taken (W1). The sand is run out of the cylinder into the conical portion by pulling out the shutter. When no further sand runs out, the shutter is closed. The weight of the cylinder with the remaining sand is found (W2). The weight of the sand collected in the conical portion may also be found separately for a check (Wc), which should be equal to (W1 – W2). The cylinder is placed centrally above the calibrating container such that the bottom of the conical portion coincides with the top of the container. There sand is allowed to run into the container as well as the conical portion until both are filled, as indicated by the fact that no further sand runs out; then the shutter is closed. The weight of the cylinder with the remaining sand is found (W3). The weight of the sand filling the calibrating container (Wcc) may be found by deducting the weight of sand filling the conical portion (Wc) from the weight of sand filling this and the container (W2 – W3). Since the volume of the cylindrical calibrating container (Vcc) is known precisely from its dimensions, the unit weight of the sand may be obtained by dividing the weight Wcc, by the volume Vcc. (Wcc may also be found directly by striking-off the sand level with the top of the container and weighing it).

∴ Unit weight of the sand: $\gamma = \dfrac{W_{cc}}{V_{cc}}$

2.3.3.1.2 Measurement of Unit Weight of the Soil:

The site at which the in-situ unit weight is to be determined is cleaned and leveled. A test hole, about 10 cm diameter and for about the depth of the calibrating container (15 cm), is made at the site, the excavated soil is collected and its weight is found (W). The sand pouring cylinder is filled with sand to about 3/4 capacity and is placed over the hole, after having determined its initial weight with sand (W4), and the sand is allowed to run into it. The shutter is closed when no further movement of sand takes place. The weight of the cylinder and remaining sand is found (W5). The weight of the sand occupying the test hole and the conical portion will be equal to (W4 – W5). The weight of the sand occupying the test hole, Ws, will be obtained by deducting the weight of the sand occupying the conical portion, Wc, from this value. The volume of the test hole, V, is then got by dividing the weight, Ws, by the unit weight of the sand. The in-situ unit weight of the soil, γ, is then obtained by dividing the weight of the soil, W, by its volume, V. If the moisture content, w, is also determined, the dry unit weight of the soil, γ_d, is obtained as $\gamma/(1+w)$.

2.3.3.2 Rubber Balloon Method (ASTM Designation D-2167)

The procedure for the rubber balloon method is similar to that for the sand cone method; a test hole is made, and the moist weight of the soil removed from the hole and its moisture content are determined. However, the volume of the hole is determined by introducing a rubber balloon filled with water from a calibrated vessel into the hole, from which the volume can be read directly. The dry unit weight of the compacted soil can be determined by using $\gamma/(1+w)$.

2.3.4 Specific gravity of soils

Soil is a three-phase system comprising solid, liquid and gas. Many soil parameters like unit weight void ratio, porosity and water content relates the proportion of these phases with each other or to the total soil mass/volume but specific gravity of a soil is a property of soil solids only. Specific gravity of a soil is defined as the ratio of the mass in air of a given volume of soil solid to the mass in air of an equal volume of distilled water at stated temperature. (Bowles, 1996).

The specific gravity of the soil grains is of some value in computing the void ratio when the unit weight and water content are known. The test is of moderate difficulty with the major source of error deriving from the presence of entrapped air in the soil sample. Since G_S does not vary widely for most soils, the values indicated here are commonly estimated without performing a test. (Bowles, 1996).

Table 2. 5 Specific gravity of different soils (Bowles, 1996)

Soil	G_S
Gravel	2.65-2.68
Sand	2.65-2.68
Silt, inorganic	2.62-2.68
Clay, organic	2.58-2.65
Clay, inorganic	2.68-2.75

A value of Gs = 2.67 is commonly used for cohesion fewer soils and a value of 2.70 for inorganic clay. Where any uncertainty exists of a reliable value of G_S, one should perform a test on a minimum of three small representative samples and average the results. Values of G_S as high as 3.0 and as low as 2.3 to 2.4 are not uncommon (Bowles, 1996).

2.3.5 Grain size Determination.

A sieve analysis is conducted by taking a measured amount of air dry, well-pulverized soil and passing it through a stack of progressively finer sieves with a pan at the bottom. The amount of soil retained on each sieve is measured, and the cumulative percentage of soil passing through each is determined. This percentage is referred to as percent finer. The percent finer of each sieve, determined by a sieve analysis, is plotted on semi-logarithmic graph paper, as shown in Figure 2.2. Note that the grain diameter, D, is plotted on the logarithmic scale and the percent finer is plotted on the arithmetic scale (Braja M. Das, 2011).

Hazen (1893) has shown that the permeability of clean filter sands in a free state can be correlated with numerical values designated D_{10}, the effective grain size. The effective grain size corresponds to 10 percent finer particles. Hazen found that the sizes smaller than the effective size affect the functioning of filters more than did the remaining 90 percent of the sizes. Hazen proposed two parameters (equation 2.1 and 2.2) to determine whether a material is uniformly graded or well graded.

The two parameters can be determined from the grain-size distribution curves of coarse-grained soils: (1) the uniformity coefficient and (2) the coefficient of gradation or coefficient of curvature C_c. These coefficients are:

$$C_u = \frac{D_{60}}{D_{10}} \quad \ldots\ldots 2.1$$

$$C_c = \frac{(D_{30})^2}{D_{60} x D_{10}} \quad \ldots\ldots 2.2$$

Where:

D_{10}, D_{30}, and D_{60} are the diameter corresponding to percent's finer than 10, 30, and 60%, respectively.

The soil is said to be well graded if Cc lies between 1 and 3 for gravels and sands. For all practical purposes, we can consider the following values for granular soils (Murthy, 1994).

Cu >4 for well-graded gravel

Cu >6 for well-graded sand

Cu <4 for uniformly graded soil containing particles of the same size

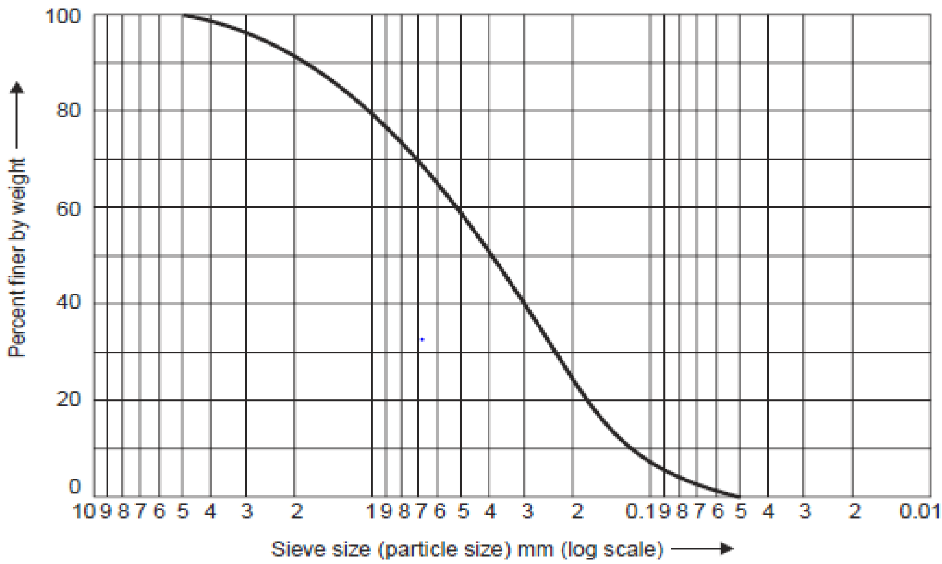

Figure 2. 2 Grain-size distribution curve of a coarse-grained soil obtained from sieve analysis

2.3.6 Soil classification system

Soils in nature rarely exist separately as gravel, sand, silt, clay or organic matter, but are usually found as mixtures with varying proportions of these components. Grouping of soils by certain definite principles would help the engineer to rate the performance of a given soil either as a sub-base material for roads and airfield pavements, foundations of structures, etc. (Murthy, 1994).

Soil classification systems divide soils into groups and subgroups based on common engineering properties such as the grain-size distribution, liquid limit, and plastic limit. The two major classification systems presently in use are and (1) the Unified Soil Classification System (also ASTM) and (2) the American Association of State Highway and Transportation Officials System (AASHTO). The AASHTO system is used mainly for the classification of highway subgrades (Braja M. Das, 2011).

2.3.6.1 Unified Soil Classification System

The Unified soil classification system was originally developed by A. Casagrande and adopted by the U.S. Corps of Engineers in 1942 as 'Airfield Classification'. It was later revised for universal use and designated as the ''Unified Soil Classification'' in 1957.It has since been adopted by the American Society for Testing and Materials (ASTM) as the standard classification of soils for engineering purposes (C.Venkatrmaiah, 2006).

The USCS uses symbols for the particle size groups. These symbols and their representations are G-gravel, S-sand, M-silt, and C-clay. These are combined with other symbols expressing gradation characteristics-W for well graded and P for poorly graded and plasticity characteristics-H for great and L for low, and a symbol, O, indicating the presence of organic material. A typical classification of CL means a clay soil with low plasticity, while SP means poorly graded sand (Muni Budhu, 2000).

The flowcharts shown in Figures below provide systematic means of classifying a fin-grained soil according to the USCS.

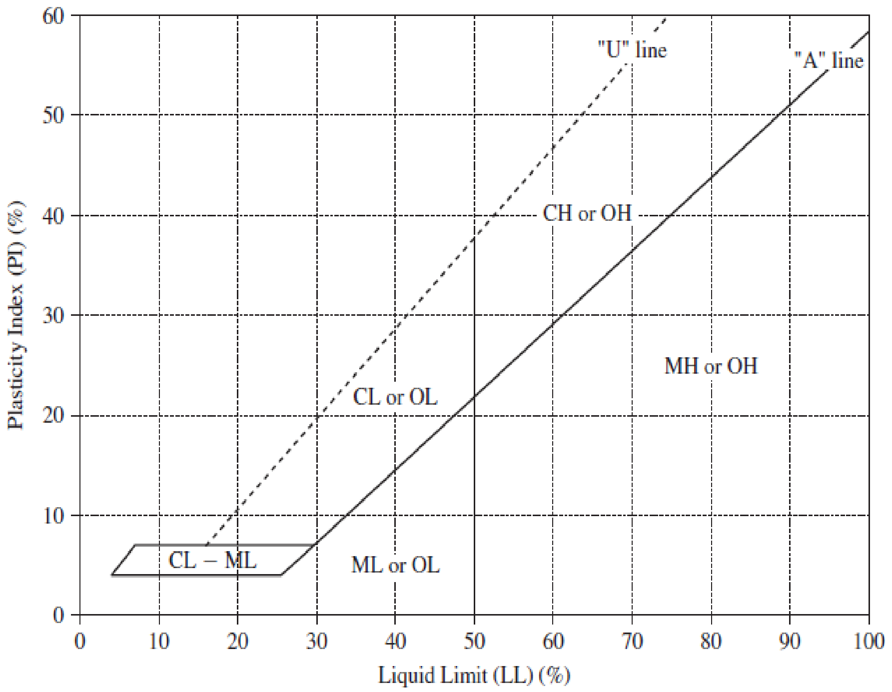

Figure 2. 3 Casagrande Plasticity Chart fine-grained soils and the fine-grained fraction, of course, grained soils.

2.3.6.2 AASHTO Classification System

The AASHTO Soil Classification System was originally proposed by the Highway Research Board's Committee on Classification of Materials for Subgrades and Granular Type Roads (1945). It is used to determine the suitability of soils for earthworks, embankments, and roadbed materials (subgrade-natural material below a constructed pavement; the subbase-a layer of soil above the subgrade; and the base-a layer of soil above the subbase that offers high stability to distribute wheel loads). According to AASHTO, granular soils are soils in which 35% or less are finer than the No. 200 sieve (0.075 mm). Silt-clay soils are soils in which more than 35% are finer than the No. 200 sieve (Muni Budhu, 2000).

The classification is more specific than the USC system in the limits placed on size ranges and amounts and ranges of liquid limits and plasticity indexes for fines. As with the USC system, these limits are placed on groups within both the granular (coarse-grained) and silty/clay (fine-grained) soils as required by soil gradations. Rather than using the No. 4 sieve (4.75 mm) of the USC system as the upper limit of the sand-size range, the AASHTO classification uses the No. 10 sieve (2.0mm) as the upper size limit of sand. However, the No. 200 sieve (0.075 mm) used in the USC system is retained to separate the finer fractions from sand.

This system classifies soils into eight groups, A-1 through A-8. Table 2.6 illustrates the current AASHTO soil classification system. Peat, muck, and other highly organic soils are classified under

A-8. They are identified by visual inspection. The soils shown in table 2.6 are groups A-1 through A-7 with two subgroups in A-7 in a total of 12 subgroups. The soil with the lowest number A-1 is the most suitable for a highway material or subgrade and subbase. The lower in the number of soil category; the more suitable is the soil. For example, the soil A-4 is better than the soil A-5 (J. Jibril, 2014).

Fine-grained soils are further rated for their suitability for highways by the group index (GI), determined as follows:

$$GI = (F_{200}-35)*[0.2+0.005(LL-40)] +0.01(F_{200}-15) (PI-10)$$

Where: F_{200} = percentage by weight passing through sieve No.200 (size 0.075 mm), expressed as whole number; LL = liquid limit; and PI = plasticity index.

Table 2. 6 AASHTO soil classification (Murthy, 1994)

General classification	Granular Materials (35 percent or less of total sample passing No. 200)							Silt-clay Materials (more than 35 percent of total sample passing NO.200)			
Groop classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve analysis percent passing											
No.10	50 max										
No.40	30 max	50 max	51 min								
No.200	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing No.40											
Liquit limit				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
plastic index	6 max		N.P	10 max	10 max	11 min	11 max	10 max	10 max	10 min	10 min
Usual types of significant constituent materials	Stone fragments gravel and sand		Fine sand	Silty or clayey gravel and sand				Silt soils		Clayey soils	
General rating as subgrade	Excellent to good					Fair to poor					

The group index is developed to evaluate the desirability of soil as highway subgrade materials. The higher the value of the group index for a given soil, the weaker will be the soil's performance as a subgrade. A group index of 20 or more indicates a very poor subgrade material. If the computed value of group index is negative, the group index is reported as zero. The group index for soils which fall in groups A-1-a, A-1-b, A-3, A-2-4, and A-2-5 is always zero (AASHTO, 2006).

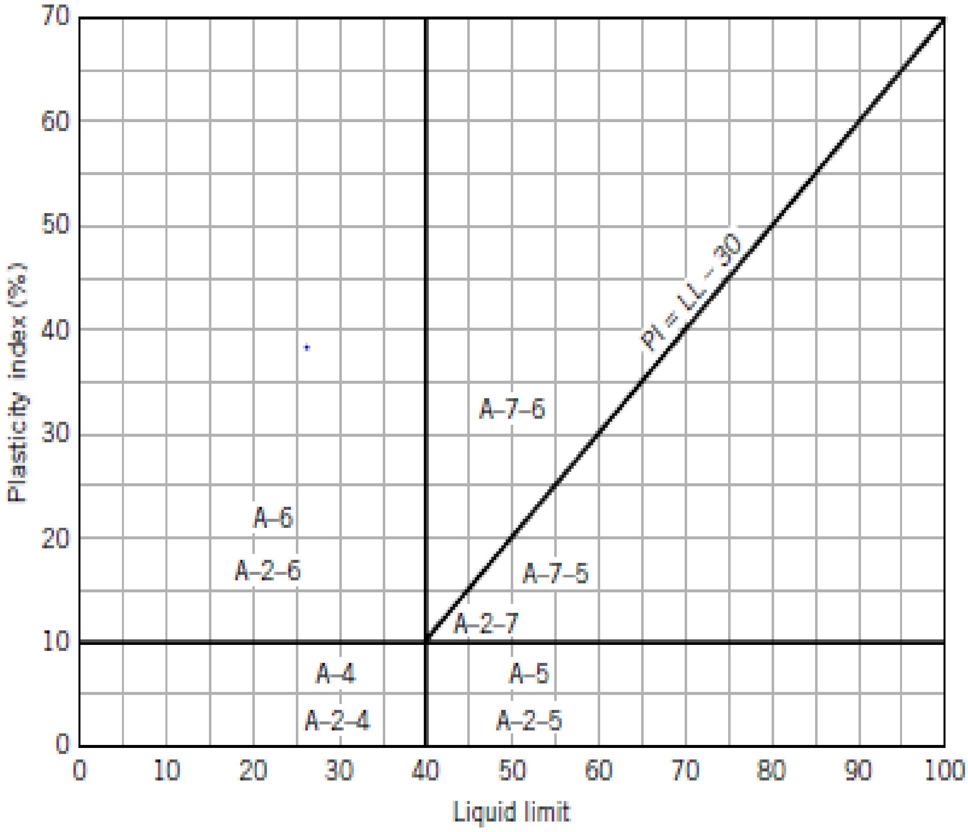

Figure 2. 4 AASHTO classifications of silt and clay within the plasticity chart. (Muni Budhu, 2000)

2.4 Moisture-density relation (compaction) test

2.4.1 General

Compaction is the reduction in a void ratio (or increase in density) due to the application and removal of a static or dynamic force. The process happens at constant water content, and only air is expelled from the material. As such, the process occurs very quickly for all types of soils. The dry density achieved by imparting energy on the soil is dependent upon the initial water content, termed the molding water content. Starting from the dry condition, an increase in the molding water content will result in a higher compacted dry density. This trend will continue up to the maximum dry density, which occurs at the optimum molding water content. Further increases in water will result in a continuous reduction in the dry density. This relationship is referred to as the compaction curve (John T.Germaine and Amy V.Germaine (2009)).

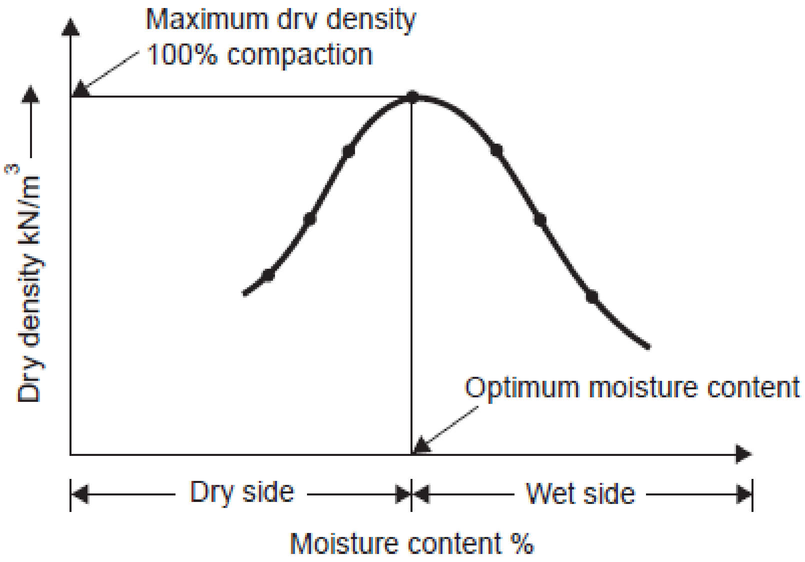

Figure 2. 5 Moisture content versus dry density at a particular compactive effort.

(Source: C.Venkartmain, 2006)

Different methods are used to compact soil in the field, and some examples include tamping, kneading, vibration, and static load compaction. This is controlled in a laboratory by employing the tamping or impact compaction method using the type of equipment and methodology developed by R. R. Proctor in 1933. Therefore, the test is also known as the Proctor test. Two types of compaction tests are routinely performed: Standard Proctor Test (lower compaction effort), and Modified Proctor Test (higher compaction effort). Most of the time standard Proctor test is used but, the modified Proctor test may be used when greater soil unit weight is required.

Table 2. 7 Summary of Standard Proctor and Modified Proctor Compaction Test Specifications (ASTM D 698 and ASTM D-1557 respectively)

Procedure	**Procedure A**	**Procedure B**	**Procedure C**
Material	≤20% Retained on No.4 Sieve	>20% Retained on No.4 ≤20% Retained on 9.5mm Sieve	>20% Retained on 9.5mm <30% Retained on 19mm Sieve
Material to be used	Passing 4.75 mm Sieve	Passing 9.5 mm sieve	Passing 19.0 mm sieve
Mold diameter	10.16cm(4 inch)	10.16cm(4 inch)	15.25cm(6 inch)

Volume of mold		944 cm^3	944 cm^3	2123 cm^3
Blow per layer		25	25	56
Layer	Standard Proctor	Three	Three	Three
	Modified Proctor	Five	Five	Five
Height of drop	Standard Proctor	30.5cm	30.5cm	30.5cm
	Modified Proctor	45.7cm	45.7cm	45.7cm
Weight of rammer	Standard Proctor	2.45Kg	2.45Kg	2.45Kg
	Modified Proctor	4.54Kg	4.54Kg	4.54Kg

2.4.2 Factors Affecting Compaction

Water acts as the lubricant between solid particles during the soil compaction process (see Fig 2.5). Because of this, in the initial stages of compaction, the dry unit weight of compaction increases (Braja M. Das, 2007).

For any compactive effort, the dry density of soil will vary with its water content. A soil compacted dry will reach a certain dry density. If compacted again with the same compactive effort, but this time with water in the soil, the dry density will be higher, since the water lubricates the grains and allows them to slide into a denser structure. Air is forced out of the soil, leaving more space for the soil solids, as well as the added water. With even higher water content, a still greater dry density may be reached since more air is expelled. However, when most of the air in the mixture has been removed, adding more water to the mixture before compaction results in a lower dry density, as the extra water merely takes the place of some of the soil solids (Vernon R. Schaefer et al. 2008).

However, another factor that will control the dry unit weight of compaction of soil at given moisture content is the energy of compaction (see Fig 2.6).

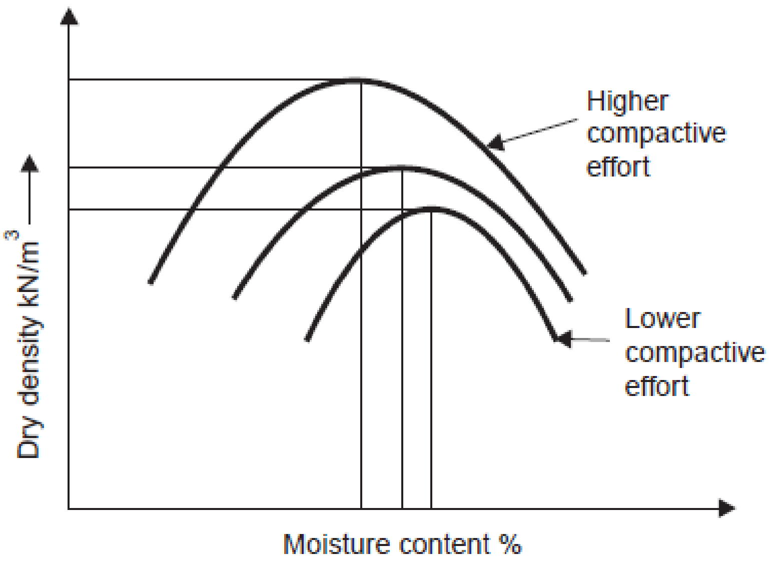

Figure 2. 6 Effect of compaction effort on compaction characteristics

(Source: C.Venkartmain, 2006)

The energy imparted by the hammer is given by:

$$E_{COMP} = Mh * g * \frac{h}{V} Nb * Nl$$

Where:

Mh is the mass of the hammer, *g* is the acceleration due to gravity, *h* is the height of fall of the hammer, *V* is the volume of compacted soil, *Nb* is the number of blows, and *Nl* is the number of layers.

Thus, the compaction energy of the standard Proctor test is

$$E_{COMP} = 2.5 * 9.81 * \frac{0.305}{9.44 * 10\text{^}-4} 25 * 3 * 10\text{^}-3 = 594 kJ / m\text{^}3$$

In modified Proctor test, the hammer has a mass 4.54 kg and falls freely from a height of 457 mm. The soil is compacted in five layers with 25 blows per layer in the modified Proctor mold. The compaction energy of the modified Proctor test compaction test is 2695 kJ/m3, about 4.5 times the energy of the standard Proctor test (Muni Budhu, 2000).

2.5 Strength and stiffness of subbase

2.5.1 California Bearing Ratio (CBR).

The CBR test is a simple strength test that compares the bearing capacity of a material with that of pavements subgrade, subbase and a well-graded crushed stone (thus, a high-quality crushed stone material should have a CBR of 100%). It is primarily intended for, but not limited to, evaluating the strength of cohesive materials having maximum particle sizes less than 19 mm (AASHTO T 193-93).

The CBR method is probably the most widely used method for designing pavement structures. This method was developed by the California Division of Highways around 1930 and has since been adopted and modified by numerous states, the U.S. Army Corps of Engineers (USACE), and many countries around the world. Their test procedure was most generally used until 1961, when the American Society for Testing and Materials (ASTM) adopted the method as ASTM D 1883, CBR of Laboratory-Compacted Soils. The ASTM procedure differs in some respects from the USACE procedure and AASHTO T 193. The ASTM procedure is the easiest to use (Vernon R. Schaefer et al. 2008).

The CBR is a comparative measure of the shearing resistance of soil. The test consists of measuring the load required to cause a piston of the standard size to penetrate a soil specimen at a specified rate. This load is divided by the load required to force the piston to the same depth in a standard sample of crushed stone. The result, multiplied by 100, is the value of the CBR. Usually, depths of 2.54 to 5.08 mm are used, but depths of 7.62, 10.16, and 12.71 mm may be used if desired. Penetration loads for the crushed stone have been standardized. This test method is intended to provide the relative bearing value, or CBR, of subbase and subgrade materials. Procedures are given for laboratory-compacted swelling, non-swelling, and granular materials. These tests are usually performed to obtain information that will be used for design purposes (AASHTO T 193-93).

According to AASHTO T 193-93 CBR values are obtained in percent by dividing the corrected load value of stress-strain curve at 2.54 and 5.08mm penetration by the standard loads of 6.9 and 10.3MPa respectively and multiplying these ratios by 100.

$$\mathrm{CBR} = \frac{\text{Corrected load value}}{\text{Standard load}} \mathrm{x}100$$

The relationship of dry density and CBR values for 10 blows, 30 blows and 65 blows per layers plotted on graph to determine the 95 percent of dry density CBR value for three point CBR test method.

The CBR value for a soil will depend upon its density, molding moisture content, and moisture content after soaking. Since the product of laboratory compaction should closely represent the results of field compaction, the first two of these variables must be carefully controlled during the preparation of laboratory samples for testing. Unless it can be ascertained that the soil being tested will not accumulate moisture and be affected by it in the field after construction, the CBR tests should be performed on soaked samples (Vernon R. Schaefer et al. 2008).

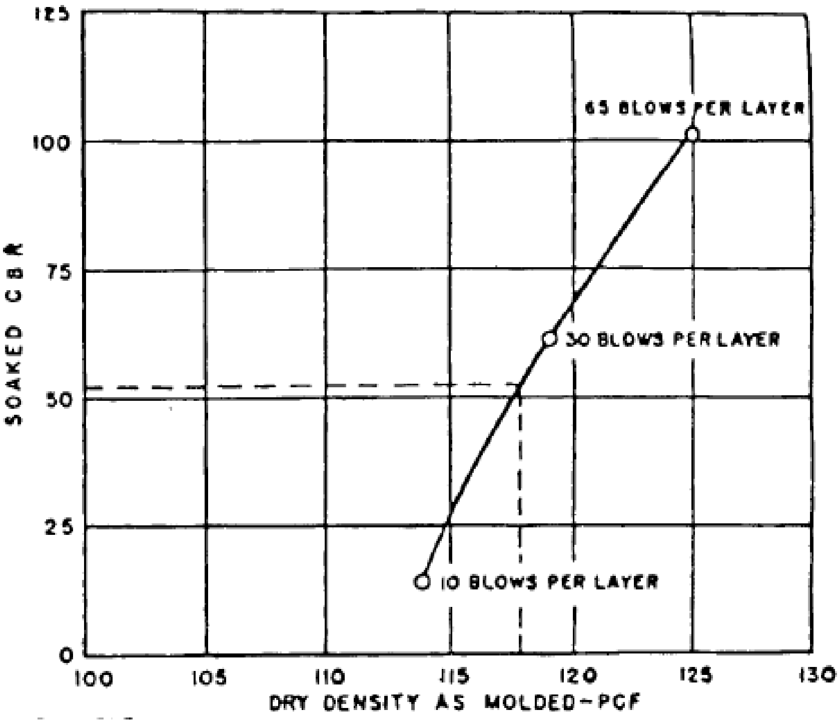

Figure 2. 7 Dry density versus CBR (AASHTO T 193-93)

Relative ratings of supporting strengths as a function of CBR values are given in Table 2.8

Table 2. 8 Relative CBR values for subbase and subgrade soils

(Vernon R. Schaefer et al. 2008).

CBR (%)	Material	Rating
>80	Subbase	Excellent
50 to 80	Subbase	Very good
30 to 50	Subbase	Good
20 to 30	Subgrade	Very good
10 to 20	Subgrade	Fair-good
5 to 10	Subgrade	Poor-good
<5	Subgrade	very poor

Table 2. 9Summary of the suitability of different soils for subgrade and sub-base applications.

Subgrade and subbase Soils for Design	Unified Soil Classifications	Load Support and Drainage Characteristics	Modulus of Subgrade Reaction (k), psi/inch	Resilient Modulus (MR), psi	CBR Range
Crushed Stone	GW, GP, and GU	Excellent support and drainage characteristics with no frost potential	220 to 250	Greater than 5700	30 to 80
Gravel	GW, GP, and GU	Excellent support and drainage characteristics with very slight frost potential	200 to 220	4500 to 5700	30 to 80
Silty gravel	GW-GM, GP-GM, and GM	Good support and fair drainage, characteristics with moderate frost potential	150 to200	4000 to 5700	20 to 60
Sand	SW, SP, GP-GM, and GM	Good support and excelle nt drainage characteristics with very slight frost potential	150 to 200	4000 to 5700	10 to 40
Silty Sand	SM, non-plastic(NP), and >35% silt (minus #200)	Poor support and poor drainage with very high frost potential	100 to150	2700 to 4000	5 to 30
Silty Sand	SM, Plasticity Index (PI) <10, And <35 % silt	Poor support and fair to poor drainage with moderate to high frost potential	100 to 150	2700 to 4000	5 to 20

Silt	ML, >50% silt, liquid limit <40, and PI <10	Poor support and impervious drainage with very high frost value	50 to 100	1000 to 2700	1 to 15
Clay	CL, liquid limit >40 and PI >10	Very poor support and impervious drainage with high frost potential	50 to 100	1000 to 2700	1 to 15

Source: American Concrete Pavement Association; Asphalt Paving Association; State of Ohio; State of Iowa; Rollings and Rollings 1996.

The higher the CBR value of a particular soil, the more strength it has to support the pavement. This means that a thinner pavement structure could be used on a soil with a higher CBR value than on a soil with a low CBR value. Clays have a CBR value of 6 or less. Silty and sandy soils are next, with CBR values of 6 to 8. The best soils for road-building purposes are the sands and gravels whose CBR values normally exceed 10.

The change in pavement thickness needed to carry a given traffic load is not directly proportional to the change in CBR value of the subgrade soil. For example, a one-unit change in CBR from 5 to 4 requires a greater increase in pavement thickness than does a one-unit change in CBR from10 to 9 (Vernon R. Schaefer et al. 2008).

2.5.2 Field strength and compaction quality control mechanism

2.5.2.1 Dynamic Cone Penetrometer (DCP) test

The TRL Dynamic Cone Penetrometer (DCP), shown in figure 2.7, is an instrument designed for the rapid in-situ measurement of the structural properties of existing road pavements with unbound granular materials. Continuous measurements can be made to a depth of 800 mm or to 1200 mm when an extension rod is fitted (ERA manual 2002).

The cone penetration is inversely related to the strength of the material. DCP test is conducted according to ASTM D 6951 (Standard Test Method for the use of Dynamic Cone Penetrometer in Shallow Pavement Applications), which was first released in 2003. This test involves measurement of penetration rate per each blow of a standard 17.6-pound or 8kg hammer, through undisturbed and compacted materials. Primary advantages of this test are its availability at lower costs and ease to collect and analyze the data rapidly (Vernon R. Schaefer et al. 2008).

It is recommended that a reading should be taken at increments of penetration of about 10 mm. However, it is usually easier to take readings after a set number of blows. It is therefore necessary to change the number of blows between readings according to the strength of the layer being penetrated. For good quality granular road bases, readings every ten blows are normally satisfactory, but for weaker sub-base layers and subgrade readings, every one or two blows may be appropriate. Little difficulty is normally experienced with the penetration of most types of granular or weakly stabilized materials. It is more difficult to penetrate strongly stabilized layers, granular materials with large particles and very dense, high-quality crushed stone. The TRL instrument has been designed for strong materials, and therefore the operator should persevere with the test. Penetration rates as low as 0.5 mm/blow are acceptable, but if there is no measurable penetration after 20 consecutive blows, it can be assumed that the DCP will not penetrate the material. Under these circumstances, a hole can be drilled through the layer using either an electric or pneumatic drill or by coring. The lower layers of the pavement can then be tested in the normal way (ERA manual 2002).

2.5.2.1.1 Test procedure

The DCP needs three operators; one to hold the instrument, one to raise and drop the weight and a technician to record the results. The instrument is held vertical and the weight carefully raised to the handle. Care should be taken to ensure that the weight is touching the handle, but not lifting the instrument, before it is allowed to drop and that the operator lets it fall freely and does not lower it with his hands. If, during the test, the DCP tilts from the vertical, no attempt should be made to correct this, as contact between the shaft and the sides of the hole will give rise to erroneous results. If the angle of the instrument becomes worse, causing the weight to slide on the hammer shaft and not fall freely, the test should be abandoned. The following steps are followed:

1. One operator holds the device vertical by the handle on the top shaft and "sealing" the cone tip by dropping the hammer until the widest part of the cone is just below the testing surface. The second person records the height at the bottom of the anvil about the ground; this is recorded as initial penetration as "below zero".
2. Again the operator lifts the hammer from the anvil to the handle, and then releases the hammer. The second person records the new height at the bottom of the anvil.
3. Step 2 is repeated until the desired depth of testing is reached or the full length of the lower rod is buried. The rod is 1m high and since there is unavailability of extension rod the test is done by excavating the soil every 1m for in-situ soil strength investigation. The soil is less confined near the surface and during excavation the upper soil is disturbed so that the DCP is able to penetrate further per drop thus making the initial drops unreliable hence the first two reading are taken as seating blows. The test is done up to 4m based on the soil condition. At that time, a specially adapted jack is used to extract the device. If the tip is disposable (i.e., not fastened to the lower shaft and left in the soil after test is complete), hitting the hammer lightly on the handle is acceptable.

2.5.2.1.2 Description of some parts of the device

Complete drawings of the DCP are given in figure 2.8. The DCP is comprised of the following elements.

a) Handle: The handle is located at the top of the device. It is used to hold the DCP shafts plumb and to limit the upward movement of the hammer.

b) Hammer: The 8-kg hammer is manually raised to the bottom of the handle and then allowed to fall freely to transfer energy through the lower shafts to the cone tip. It is guided by the upper shaft.

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

d) Anvil: The anvil serves as the lower stopping mechanism for the hammer. It also serves as a connector between the upper and the lower shaft. This allows for disassembly which reduces the size of the instrument for transport.

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

f**) The cone:** measures 20 mm in diameter and has a 600 cone.

The combined mass of the upper shaft, anvil, lower shaft and cone is approximately 3.1kg. The DCP (except the hammer) is usually constructed of stainless steel to prevent corrosion. But, if the ordinary or mild steel is employed, the instrument has to be cleaned and dried after each use to prevent rusting. The cone tip should be replaced when the diameter of its widest section is deformed by more than 10% (2-mm) (Gedeyon A.; 2015).

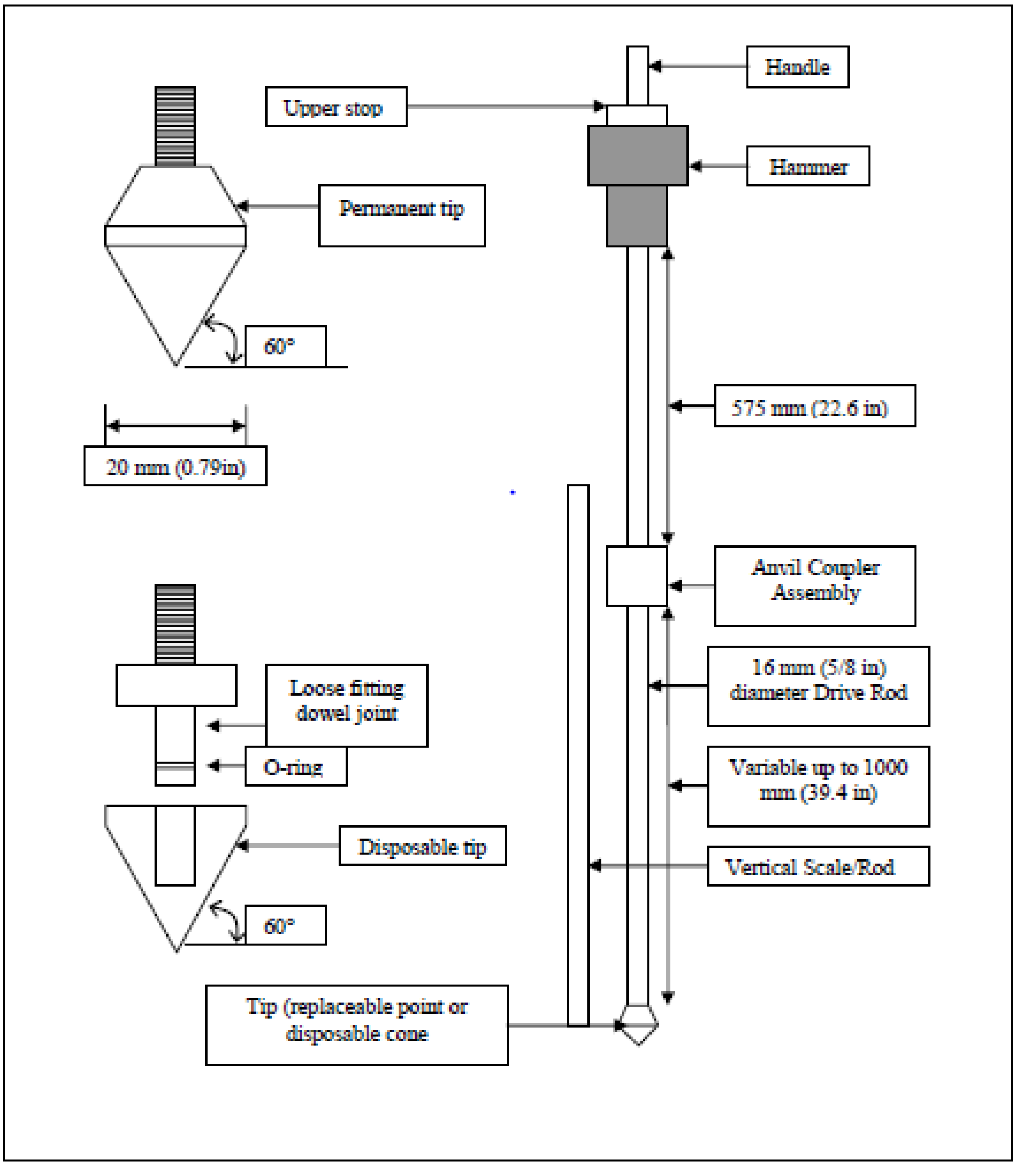

Figure 2. 8 DCP design and cone tip details

2.5.2.1.3 Benefits and Limitations

The DCP offers many benefits compared to other similar hand-held testing devices. Its benefits make the device not only inexpensive, portable and easy to operate and understand but also the most versatile among other similar equipment. Some of these benefits are listed below:

a) **Easy to Use:** It does not take the extensive experience to interpret results. An operator can be trained in a matter of minutes. Its light weight makes it preferable for field exploration for lightweight structures.

b) **Large Penetration Depth:** Data can be collected to a depth of 6m using extension rods compared to a maximum of 0.3m for other hand-held testing devices like the vane shear test.

c) **Fast**: A large amount of data can be taken quickly, and the DCPI values are easily converted into other indices which are used to determine the bearing properties and performance of the underlying soil.

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

e) **Versatility**: The device has found many applications in the construction field for construction control, supervision and design parameter determination.

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

Other than manpower errors there are also other limitations:

- Adhesion between the rod and the soil for highly plastic soil and collapsible granular soils.
- It is difficult to penetrate hard and granular materials.
- As in most dynamic tests, the DCP does not give reliable result in saturated fine graded soils. This is because the dynamic load from the equipment is carried by a developed pore water pressure rather than the soil grains in these type soils.

The maximum depth suggested for this test is about 6m using extension rod. If tests have to be conducted beyond 6m depth, one has to use lubrication between the hole and the rod throughout the test (Gedeyon A. 2015).

2.5.2.2 Selection of appropriate type of compaction equipment

Several compaction devices are available in modern earthwork, and selection of the proper equipment is dependent on the material intended to be densified. Generally, compaction can be accomplished using pressure, vibration, and/or kneading action. Different types of field compaction equipment are appropriate for different types of soils. Steel-wheel rollers, the earliest type of compaction equipment, are suitable for cohesion less soils. Vibratory steel rollers have largely replaced static steel-wheel rollers because of their higher efficiency. Sheep foot rollers, which impart more of a kneading compaction effort than smooth steel wheels, are most appropriate for plastic cohesive soils. Vibratory versions of sheep foot rollers are also available. Pneumatic rubber-tired rollers work well for both cohesion less and cohesive soils. A variety of small equipment for hand compaction in confined areas is also available (Vernon R. Schaefer et al. 2008).

Table 2. 10 Recommended field compaction equipment (Rollings and Rollings 1996)

Soil	First Choice	Second Choice	Comment
Rock fill	Vibratory	Pneumatic	…..
Plastic soil, CH,MH	Sheets foot or pad foot	Pneumatic	Thin lifts usually needed
Low-Plastic soil CL,ML	Sheets foot or pad foot	Pneumatic or Vibratory	Moisture control often critical for silty soils
Plastic sands and gravels GC, SC	Pneumatic or Vibratory	Pad foot	…..
Silty sands and gravel SM, GM	Vibratory	Pneumatic or Pad foot	Moisture control often critical
Clean sand SW,SP	Vibratory	Impact or Pneumatic	…..
Clean gravel GW,GP	Vibratory	Pneumatic, Impact or Grid	grid useful for over-size particle

2.5.2.3 Controlling dry density and optimum moisture content of field compaction

Laboratory compaction testing performed on subbase layers according to AASHTO T 99; Standard Proctor density shows a significant change in density and optimum water content with change in gradation in similar aggregate types. Therefore, it is recommended to use relative density values correlated to gradation for compaction control of aggregate materials in the field to avoid inadequate compaction. A relative density of at least 70% is recommended (Vernon R. Schaefer et al. 2008). During the construction of compacted fills, it is standard practice to make the in-situ determination of a unit weight of the soil after it is placed to ensure that the compaction effort has been adequate (C.Venkatramaiah, 2006).

2.6 Swelling potential of soil

Swelling of soils is recognized as problematic soils that undergo significant volume changes when their moisture content is changed. Principally, swelling occurs when water infiltrates between the clay particles, causing them to separate (Okagbue; 1990).

Many lightly-loaded structures have undergone severe damages when they were founded over such soils due to differential heaving of the under lying soils. Volume change of these soils is a major cause of concern, since it causes extensive damage to the structures and the allied services (Ferber, V. et al; 2009).

The response of expansive soils in the form of swelling and shrinkage due to changes in water content is frequently expressed superficially as heaving and settlement of lightly loaded structures such as pavements, walkways, railways, roadways, foundations, channel linings, etc. (Shi, B.et al ,2002). Even when mitigating measures such as drain systems have been provided to prevent these soils from reacting to changes in their moisture condition, the soils still exhibit inherent low shear strength and undergo large secondary compression. Expansive or swelling soils are highly plastic soils that typically contain clay minerals such as Montmorillonite that attract and absorb significant amount of water (Muntohar, A.S; 2000).

This property is associated with the presence of varieties of minerals such as Kaolinite, Illite and Montmorillonite group of minerals (Ene and Okagbuea; 2009).

The type and amount of mineral and the percentage of clay fraction play a vital role in controlling the index properties such as liquid limit, plasticity index and activity as well as the swelling characteristics including swell potential and swell pressure of such soils. Based on the mineral present, the swell potential of the soil varies; the Montmorillonite group minerals have the maximum swell potential and the Kaolinite family minerals have the least swelling properties (Pak. J. Engg. & Appl. Sci, 2014)

Many empirical models have been proposed by various researchers for predicting the swelling properties of such soils on the basis of physical and index properties. The evaluation of swelling parameters (swell potential and swell pressure) of such soils includes both direct as well as indirect measurements. The direct methods involve the physical measurements of swell potential and swell pressure through laboratory tests; however, the indirect methods involve the use of empirical models and correlations formulated on the basis of basic soil properties (Pak. J. Engg. & Appl. Sci, 2014).

A number of correlations between index properties and the swelling characteristics have been developed in the past for a variety of expansive soils, some of which are presented in Table 2.11.

Table 2. 11 Empirical correlation for predicting the swelling parameters by various researchers (Pak. J. Engg. & Appl. Sci, 2014).

Correlation	Author	Reference
S_p (%) = $Be^{A(PI)}$	Chen	Chen, F.H; 1975
S_p (%) = 7.518+0.323 (Cf)	Muntohar	Israr, J; 2012
S_p (%) = $60K(PI)^{2.44}$	Holtz et al.	Ferber, V. et al; 2009

Note: Sp: swell potential; PI: plasticity index; A: activity; Cf: clay fraction; B and K are empirical constants.

CHAPTER THREE

STUDY AREA, MATERIALS AND RESEARCH METHODOLOGY

3.1 Study area

The study area is found in Jimma zone on the road that connects Jimma city to Limmu genet wereda which passes through Mazoriya (in Mana wereda). Limmu genet (also known as Limmu Inariya and formerly as Limmu suntu) is one of the towns in Southwestern Ethiopia, Oromia National Regional State which founded in 1952 (Socio-economic profile of Jimma, Google, 2017). It is predominantly covered with huge kilometer square of coffee beans which is the most exported product in Ethiopia (e.g., Horizon coffee plant) and some agricultural products.

3.1.1 Location of the study area

Limmu Genet is one of the town in Southwestern Ethiopia; Oromia National Regional State. It is the administrative center of Limmu Kosa wereda which is 76 km far from Jimma town. Part of Jimma zone, Limmu Kosa wereda is bordered on the south by Kersa, on the southwest by Mana, on the west by Gomma, on the northwest by the Didessa River which separates it from the Illubabor Zone, on the north by Limmu Sakka, on the northeast by Gibe River which separates it from west Shewa zone and the Southern Nations, Nationalities and peoples of Region, on the east by Sokoru, and on the southeast by Tiro Afeta. The administrative center of this wereda includes Ambuye and Babu. (Socio-economic profile of Jimma, Google, 2017)

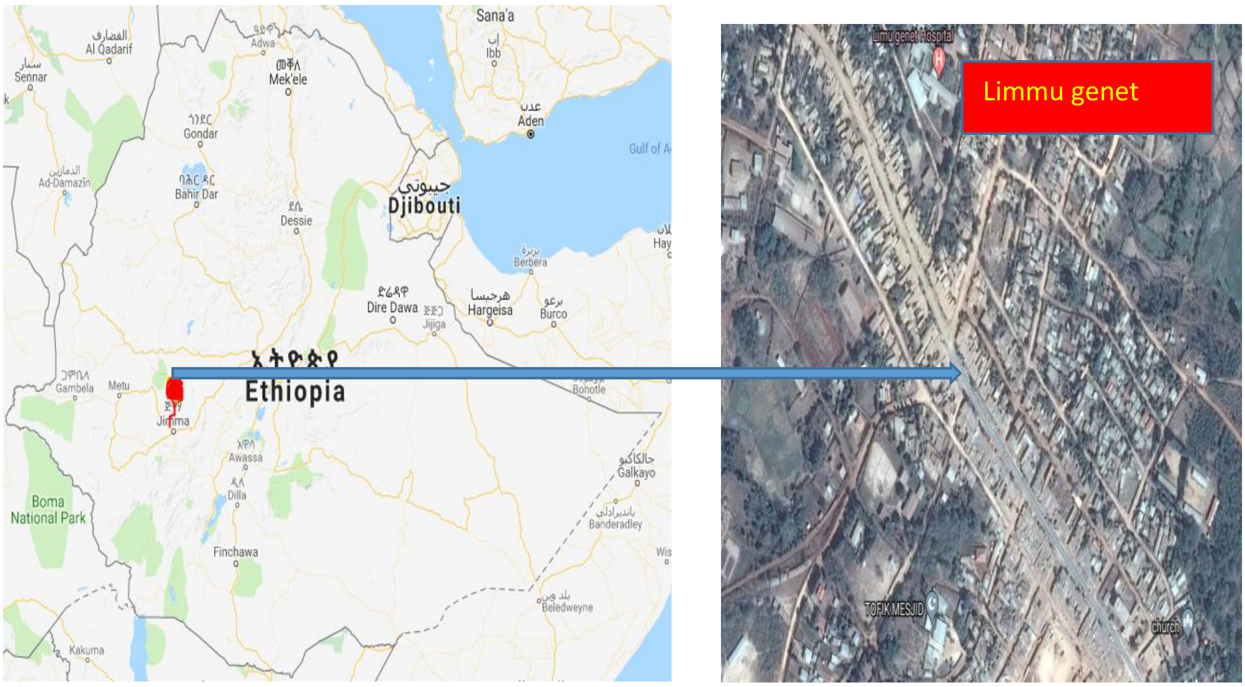

Figure 3. 1 Location of study area (Google map, 2018)

Figure 3. 2 Location of the quarry sites (Google map, 2018)

3.1.2 Topography, terrain and relief

The shape of the land's surface, or topography, influences how water flows onto and off of the soil, as well as how it moves into and through the soil. Consequently, it exerts a strong control on the balance between soil organic matter additions and decomposition, erosion and deposition, leaching and accumulation, and even oxidation and reduction. Topography is formed by depositional features, such as lava flows, moraines, alluvial fans, and dunes, and by geologic erosion, such as stream incision, glaciation, and mass wasting.

The administrative center of Limmu genet has a latitude and longitude of $08^{0}06'$N $36^{0}57'$E/ 8.100^{0}N 36.950^{0}E with the elevation of 1773 meters above sea level. The wereda predominantly covered with huge kilometer square of forest and coffee beans which is the most exported product in Ethiopia (e.g., Horizon coffee plant) and some agricultural products. The terrain and topography of the study area is rolling with significant flat area. This is due to the slopes generally rise and fall moderately and where occasional steep slopes are encountered.

3.1.3 Climate

The climate condition of Limmu Genet wereda is categorized under "Wayna Dega" with the minimum, mean and maximum temperature of 8^{o}c, 20^{o}c, and 35^{o}c respectively. It has maximum rainfall of laying between Junes to September.

3.1.3.1 Rainfall

According to meteorological data of worldweatheronline.com the rainfall of limu Genet wereda ranges approximately in between 10mm to 425mm throughout the year.

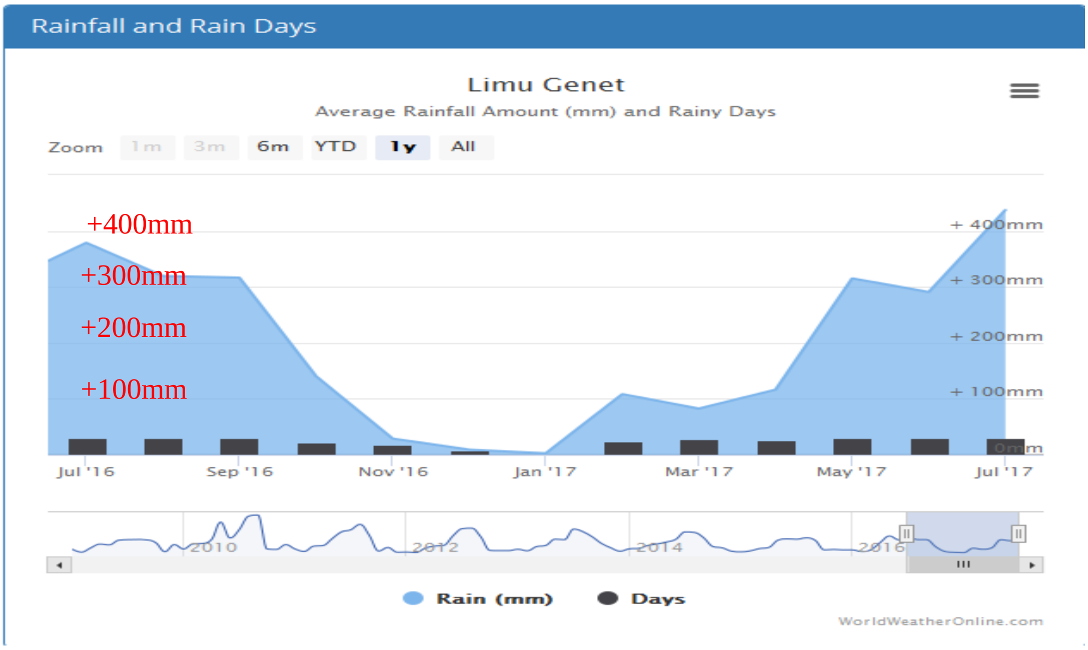

Figure 3. 3 Average rainfall amounts (mm) of study area (worldweatheronline.com, 2017)

3.1.3.2 Temperature

The temperature of this study area ranges between 8°c and 35°c approximately.

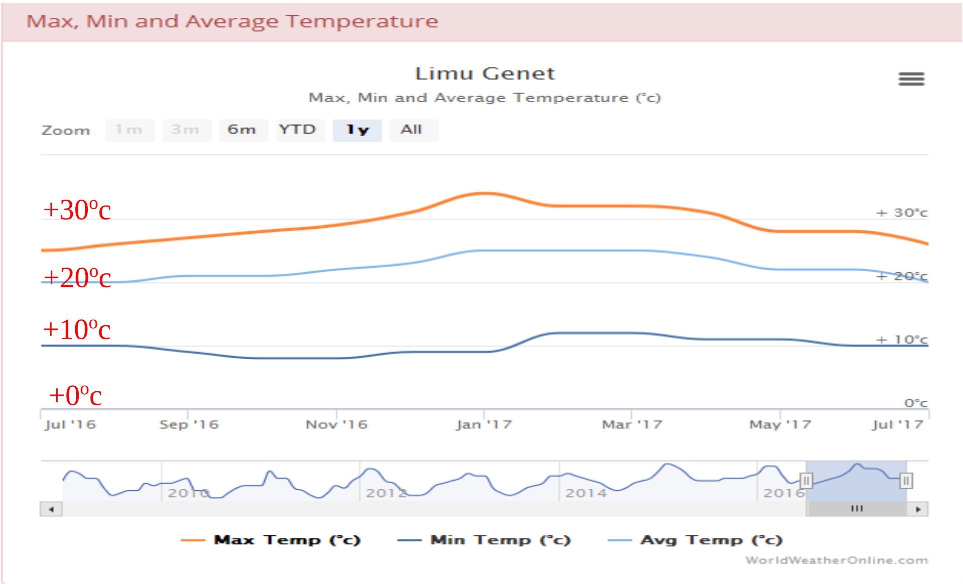

Figure 3. 4 Temperature distribution of Limmu Genet Wereda (worldweatheronline.com, 2017)

3.1.4 Seismicity

The wereda is found under Jimma zone which has seismicity magnitude of 5.1. According to the website below many houses are damaged and injuries are recorded without any death.

Table 3. 1 Seismicity magnitude in Ethiopia

Date	Region	Magnitude
2010-12-16	Jimma, Hosaena, Shenk'ola, Wenjela	5.1
1973-04-01	Ethiopia, Djibouti	5.9
1969-03-29	Sardo	6.2
1961-06-01	Karakore	6.5
1921-08-14	Massawa Subregion	5.9
1875-11-02	Tigray Region	6.2
1845-02-12		
1842-12-08	Ankober	

(https://en.wikipedia.org/wiki/List_of_earthquakes_in_Ethiopia)

3.2 The functional classification of the road corridor

The choice of design controls and criteria is influenced by the functional classification of the road; the nature of the terrain; the design vehicle; the traffic volumes expected on the road; the design speed; the density and character of the adjoining land use; and economic and environmental considerations (ERA Manual, 2002). The road of study area can be classified under Link Roads (Class II). The road corridor connects Limmu genet wereda to Addis Ababa passing through Jimma town.

3.3 Materials

3.3.1 Selected materials

Gravel material to be used for sub-base shall be obtained from approved sources in borrow areas, cuts or existing pavement layers. The complete sub-base shall contain no material having a maximum dimension exceeding two-thirds of the completed layer thickness. Gravel Sub-base material shall, unless otherwise stated, conform to the following requirements (ERA Manual, 2002).

(a) Grading Limits

The sub-base material shall comply with one of the gradings shown in Table 4.1 as described in the Contract. The material shall have a smooth continuous grading within limits for grading A, B, or C given below.

Table 3. 2 Grading Requirements for Sub-base Material (ERA Manual, 2002)

Sieve Size (mm)	Mass Percent Passing			
	A	B	C	D
63	100	-	-	-
50	90-100	100	100	-
37.5			80-100	
25	51	55-85	-	100
20			60-100	
9.5	-	40-70		51-85
5			30-100	
4.75	35-70	30-60		35-65
2	-	20-51		25-51
1.18			17-75	
0.425	-	10-30		15-30
0.3			9-50	
0.075	5-15	5-15	5-25	5-15

(b) Plasticity Index

All sub-base materials shall have a maximum Plasticity Index of 6 or 12, as described in the Contract, and when determined by AASHTO T-90. The plasticity product (PP = PIx percentage passing the 0.075mm sieve) shall not be greater than 75.

(c) Californian Bearing Ratio (CBR)

The minimum soaked Californian Bearing Ratio (CBR) shall be 30% when determined by the requirements of AASHTO T-193. The Californian Bearing Ratio (CBR) shall be determined at a density of 95% of the maximum dry density when determined by the requirements of AASHTO T-180 method D.

3.4 Research Methodology

As the main objective of this study is assessing or identifying a good quality of selected material as subbase for road construction along Mazoriya to Limmu genet, the study designed how to collect, analyze and interpret observations from the laboratory tests and field tests. The first task is reconnaissance survey of the study area to identifying a query site for collection of sample for further study.

In the minimization of the cost of construction and maintenance, the suitability and easy accessibility of subbase material plays a great role. To assess the suitability of identified quarry sites, the subbase materials were checked to be used as–it is by laboratory tests and ERA standard specification manual.

From the recovered samples laboratory tests:

- Natural moisture content,
- Specific gravity test,
- Grain size analysis
- Atterberg limit tests
- Compaction test (Modified Proctor test) and
- Three point method CBR test

And a field test of:

- Sand-replacement density test

If the material passes the standard specification, the material will be recommended as good material. Otherwise, the material is not suitable for the road construction and identifying other quarry sites are recommended, or chemical treatment (stabilization) should be performed for the modification of the material in further study.

3.4.1 Sampling, sample description and sample preparation

The samples were collected from available three quarry sites in the wereda after reconnaissance survey of the study area. The quarries are located in Babu Kebele around Kebena forest, between Kosa and Ambuye kebele in “Horizon coffee plant”, and the entrance of Limmu Kosa town through the road goes to Gumer about 5km far from the main road. A representative disturbed sample was collected from different places of each site by random selection and visual inspection. From those samples, a sample of air dried for grain size distribution is prepared by quartering and ruffling method. The physical properties of the samples which taken from the three sites are different in grain size having a gravely soils. It is clear seen that from the figure 3.5 below, the color of the disturbed samples that obtained from Ambuye entrance in the Horizon coffee plant, Babu Kebele near Kebena forest and Limmu genet entrance through the road goes to Gumer is red, white and gray respectively. For all these samples, the proposed laboratory tests were done.

Figure 3. 5 Visual descriptions of the samples.

3.4.2 Index properties

To distinguish the soil type index properties plays a great role. Index properties may be divided into two main categories namely, soil grain properties and soil aggregate properties. The soil grain properties are the properties of the individual grains as expressed by size, shape and mineralogical characteristics (Alemayehu T and Mesfin L; 1999).

3.4.2.1 Moisture content

The test is performed to determine the water (moisture) content of soils. The water content is the ratio, expressed as a percentage, of the mass of “pore” or “free” water in a given mass of soil to the mass of the dry soil solids.

Test procedure

ASTM D 2216 - Standard Test Method the moisture content of soil, rock and soil-aggregate mixture is determined as follow:

I. Record the moisture can and lid number. Determine and record the mass of empty, clean, and dry moisture can with its lid (M_C)
II. Place the moist soil in the moisture can and secure the lid. Determine and record the mass of the moisture can (now containing the moist soil) with the lid (MCMS).
III. Remove the lid and place the moisture can (containing the moist soil) in the drying oven that is set at 105 °C. Leave it in the oven overnight.
IV. Remove the moisture can. Carefully but securely, replace the lid on the moisture can using gloves, and allow it to cool to room temperature. Determine and record the mass of the moisture can and lid (containing the dry soil) (MCDS).

V. Empty the moisture can and clean it.

Data Analysis:

(1) Determine the mass of soil solids.

M_S = MCDS −MSC

(2) Determine the mass of pore water.

M_W = MCMS −MCDS

(3) Determine the water content.

$$w(\%) = \frac{Mw}{Ms} x100$$

3.4.2.2 Grain size analysis

This test is performed according to ASTM D 422 - Standard Test Method to determine the percentage of different grain sizes contained within a soil. The mechanical or sieve analysis is performed to determine the distribution of the coarser, larger-sized particles and fine particles. The distribution of different grain sizes affects the engineering properties of soil. Grain size analysis provides the grain size distribution, and it is required in classifying the soil.

Test procedure (dry preparation)

I. Write down the weight of each sieve as well as the bottom pan to be used in the analysis.

II. Record the weight of the given dry soil sample.

III. Make sure that all the sieves are clean, and assemble them in the ascending order of sieve numbers sieve #200 at bottom. Place the pan below 200 sieve number. Carefully pour the soil sample into the top sieve and place the cap over it.

IV. Place the sieve stack in the mechanical shaker and shake for 10 minutes.

V. Remove the stack from the shaker and carefully weigh and record the weight of each sieve with its retained soil. In addition, remember to weigh and record the weight of the bottom pan with its retained fine soil.

3.4.2.3 Specific gravity

Specific gravity is the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature. The test is performed according to ASTM D 854-00 to determine the specific gravity of soil by using a pycnometer. It is used in the phase relationship of air, water, and solids in a given volume of the soil.

Test procedure

I. Determine and record the weight of the empty clean and dry pycnometer, WP.

II. Place 10g of a dry soil sample (passed through the sieve No. 10) in the pycnometer. Determine and record the weight of the pycnometer containing the dry soil, WPS.
III. Add distilled water to fill about half to three-fourth of the pycnometer. Soak the sample for 10 minutes.
IV. Apply a partial vacuum to the contents for 10 minutes, to remove the entrapped air.
V. Stop the vacuum and carefully remove the vacuum line from pycnometer.
VI. Fill the pycnometer with distilled (water to the mark), clean the exterior surface of the pycnometer with a clean, dry cloth. Determine the weight of the pycnometer and contents, WB.
VII. Empty the pycnometer and clean it. Then fill it with distilled water only (to the mark). Clean the exterior surface of the pycnometer with a clean, dry cloth. Determine the weight of the pycnometer and distilled water, WA.
VIII. Empty the pycnometer and clean it.

3.4.3 Soil classification

3.4.3.1 Unified soil classification system (USCS)

The USCS uses symbols for the particle size groups. These symbols and their representations are G-gravel, S-sand, M-silt, and C-clay. These are combined with other symbols expressing gradation characteristics-W for well graded and P for poorly graded and plasticity characteristics-H for great and L for low, and a symbol, O, indicating the presence of organic material. A typical classification of CL means a clay soil with low plasticity, while SP means poorly graded sand (Muni Budhu, 2000).

3.4.3.2 AASHTO classification system

The classification is more specific than the USC system in the limits placed on size ranges and amounts and ranges of liquid limits and plasticity indexes for fines. As with the USC system, these limits are placed on groups within both the granular (coarse-grained) and silty/clay (fine-grained) soils as required by soil gradations. Rather than using the No. 4 sieve (4.75 mm) of the USC system as the upper limit of the sand-size range, the AASHTO classification uses the No. 10 sieve (2.0mm) as the upper size limit of sand.

3.4.4 California Bearing Ratio (CBR) test

This test is done according to AASHTO T 193-93. CBR values are obtained in percent by dividing the corrected load value of stress-strain curve at 2.54 and 5.08mm penetration by the standard loads of 6.9 and 10.3MPa respectively and multiplying these ratios by 100.

$$\mathrm{CBR} = \frac{\text{Corrected load value}}{\text{Standard load}} \mathrm{x}100$$

The relationship of dry density and CBR values for 10 blows, 30 blows and 65 blows per layers plotted on graph to determine the 95 percent of dry density CBR value in three point method.

3.4.5 The swelling potential of the materials

Soil can be classified according to their swelling potential to low, medium, high and very high or inactive, medium and active.

3.4.5.1 Direct estimation of swell potential by CBR swell test

According to AASHTO T 193-93 to evaluate the water absorption or the swell potential of the subbase material, soaking or immersion of the specimen for 96 hours (4 days) is done. During soaking the level of water in thank is maintained in the soaking thank approximately 25.4mm above the top of the specimen to allow free access of water to the specimen. At the end of 96 hours the final dial gauge reading on the soaked specimen was made and the swell as a percentage of the initial sample length is calculated.

$$\text{Percent swell} = \frac{\text{change in length in mm after soaking}}{\text{Initial sample length in mm}} \mathrm{x}100$$

3.4.5.2 Indirect estimation of swell potential

An indirect prediction of swell potential includes correlations based on index properties, swell, physical indicator and a combination of them. Some of such classification systems are:-

I. Skempton's method (Mckeen, 1976)

This method is developed, by combining Atterberg limits and clay content into a single parameter called Activity. Activity is defined as the ratio of the plastic index to percent of clay fraction finer than 2µm. Skempton suggested three classes of clays according to their activity.

Activity	Potential of expansion
Ac < 0.75	low (inactive)
0.75 < Ac<1.25	medium (normal)
Ac > 1.25	high (active)

II. Seed, Woodward and Lundgreen

According to Seed, Woodward and Lundgreen (1974), plasticity index is a parameter which can be used as a preliminary indicator of the swelling characteristics of a soil.

Seed, Woodward and Lundgreen suggested three classes of clays according to their plasticity index.

Plasticity index	Swell potential
0-15	low
10-35	medium
20-55	high
55 and above	very high

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 Natural moisture content

The natural moisture content of each soil sample is different from the other. From table 4.1 the natural moisture content of soils of the study area ranges from 10.89%- 41.20%. The dry density and bulk density of the sites range from 1.142 g/cm^3 – 2.021 g/cm^3 and 1.604 g/cm^3 - 2.243 g/cm^3 respectively.

Table 4. 1 Natural moisture content, dry density and Bulk Unit weight of the samples

Ser. No:	Location	Sample designation	Test trial	Nature of sample	NMC %	Average NMC%	Dry density (g/cm3)	Bulk Density (g/cm3)
1	Limmu Genet	Site one	1	Disturbed	41.20	40.45	1.142	1.604
			2	Disturbed	39.96			
			3	Disturbed	40.20			
2	Babu	Site two	1	Disturbed	11.06	10.95	2.022	2.243
			2	Disturbed	10.89			
			3	Disturbed	10.89			
3	Ambuye	Site three	1	Disturbed	36.29	36.09	1.349	1.835
			2	Disturbed	35.74			
			3	Disturbed	36.25			

4.2 Atterberg's limit

This test was done according to the Standard Reference: ASTM D 4318-98 standard test method. The Atterberg Limits for the soil samples in these three sites summarized in Table 4.3 and from this it is noticeable that liquid limit ranges from 40.25% – 50.92 %, plastic limit ranges from 32.04% – 41.81% and plastic index from 8.21% – 11.77%.

Table 4. 2 Atterberg's limit laboratory test result

Ser. No:	Location	Sample designation	Nature of sample	Liquid limit LL %	Plastic limit PL%	Plastic Index PI %
1	Limmu Genet	Site one	Disturbed	50.92	41.81	8.70
2	Babu	Site two	Disturbed	40.25	32.04	8.21
3	Ambuye	Site three	Disturbed	48.60	36.83	11.77

4.3 Specific gravity

Specific gravity is the ratio of the mass of the unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature. From table 4.3 the average specific gravity of site one, site two and site three is 2.66, 2.58 and 2.68 respectively, i.e., the specific gravity of site three is the maximum and that of site two is the minimum. According to Bowles (1996), the soil sample of site one and site three is a gravel material. The soil sample of site two has a fine clay content.

Table 4. 3 Summary of specific gravity of the study area

Ser. No:	Location	Sample designation	Test trial	Nature of sample	Specific Gravity	Average specific Gravity	Remark
1	Limmu Genet	Site one	1	Disturbed	2.694	2.66	Medium
			2	Disturbed	2.641		
			3	Disturbed	2.658		
2	Babu	Site two	1	Disturbed	2.608	2.58	Is the lowest of the three
			2	Disturbed	2.567		
			3	Disturbed	2.558		
3	Ambuye	Site three	1	Disturbed	2.676	2.68	Is the highest of the three
			2	Disturbed	2.702		
			3	Disturbed	2.652		

4.4 Grain size distribution

This test is performed to determine the percentage of different grain sizes contained within a soil. To determine the gradation of soil uniformity coefficient and coefficient of curvature is determined from the semi-logarithmic graph of percent finer versus sieve size. These coefficients are summarized in table 4.5.

Table 4. 4 Percentage passing of each sieve size

Sieve size (mm)	Percent passing		
	Site one	Site two	Site three
75.00	94.50	100.00	100.00
63.00	94.50	83.42	100.00
37.50	86.33	72.58	90.25
19.00	68.33	56.83	78.75
13.20	58.92	49.42	70.83
9.50	53.33	45.67	64.33
4.75	39.83	37.58	50.08
2.00	23.00	30.00	34.25
0.850	12.75	24.50	23.92
0.425	7.58	19.67	17.92
0.300	6.33	17.83	16.00
0.150	3.67	15.17	13.58
0.075	3.00	9.92	8.25

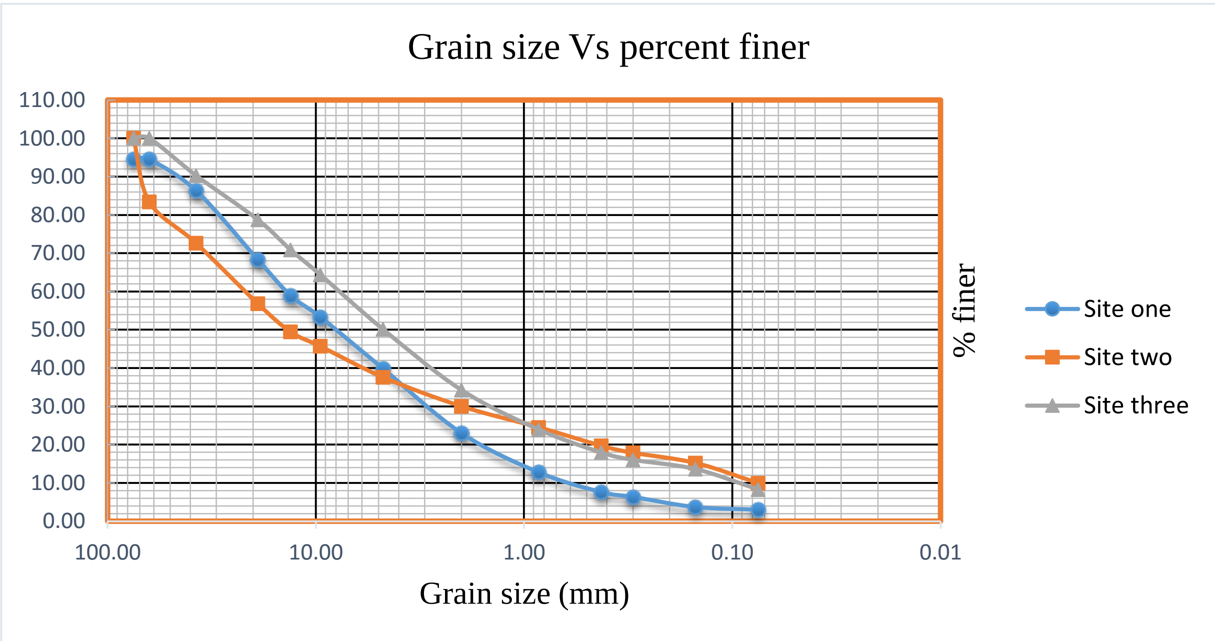

Figure 4. 1 Grain size distribution curve

4.5 Soil classification

4.5.1 Classification of soils based on Unified soil classification system (USCS)

The classification of USCS depends on the value uniformity coefficient and coefficient of curvature, a percentage of gravel, sand and little content of the sample. From flow chart of USCS classification of appendix F it can realized that, since the sample from site one has a percentage of gravel in the soil is greater than the percentage of sand with less than five percent of the fine content of sieve no.200 and the value of a coefficient of curvature is not in between 1 and 3; the soil is poorly graded gravel with sand (GP). The soil sample from site two also has more percent of gravel than sand with approximately ten percent of fines. Its little content categorized under silt with low plasticity (see figure 4.2). The uniformity coefficient and coefficient of curvature of this sample are in the permissible range, and then the soil is well-graded gravel with silt and sand which designated by GW-GM. Accordingly, the sample from site three is poorly graded gravel with silt and sand (GP-GM). Table 4.5 summarizes the result.

Table 4. 5 Classification of the soil sample according to USCS

Ser. No:	Location	Uniformity coefficient (C_U)	Coefficient of curvature (C_C)	Percentage amount of particle size % for USCS classification			Percentage finer than 0.075mm sieve	Classification
				Gravel	Sand	Silt and clay		
1	Limmu Genet	23.08	0.923	60.17	36.83	3.00	3.00	Poorly graded gravel with sand (GP)
2	Babu	306.67	2.320	62.42	27.66	9.92	9.92	Well graded gravel with silt & sand (GW-GM).
3	Ambuye	88.88	4.014	49.92	41.83	8.25	8.25	Poorly graded gravel with silt & sand(GP-GM)

4.5.2 AASHTO classification system

The AASHTO soil classification system is used to determine the suitability of soils for earthworks, embankments, and roadbed materials (subgrade—natural material below a constructed pavement; subbase—a layer of soil above the subgrade; and base—a layer of soil above the subbase that offers high stability to distribute wheel loads). According to AASHTO, granular soils are soils in which 35% or less are finer than the No. 200 sieve (0.075 mm) (Muni Budhu, 2000). Thus all the soil sample from study area are a granular material, i.e., suitable for highway roadbed material.

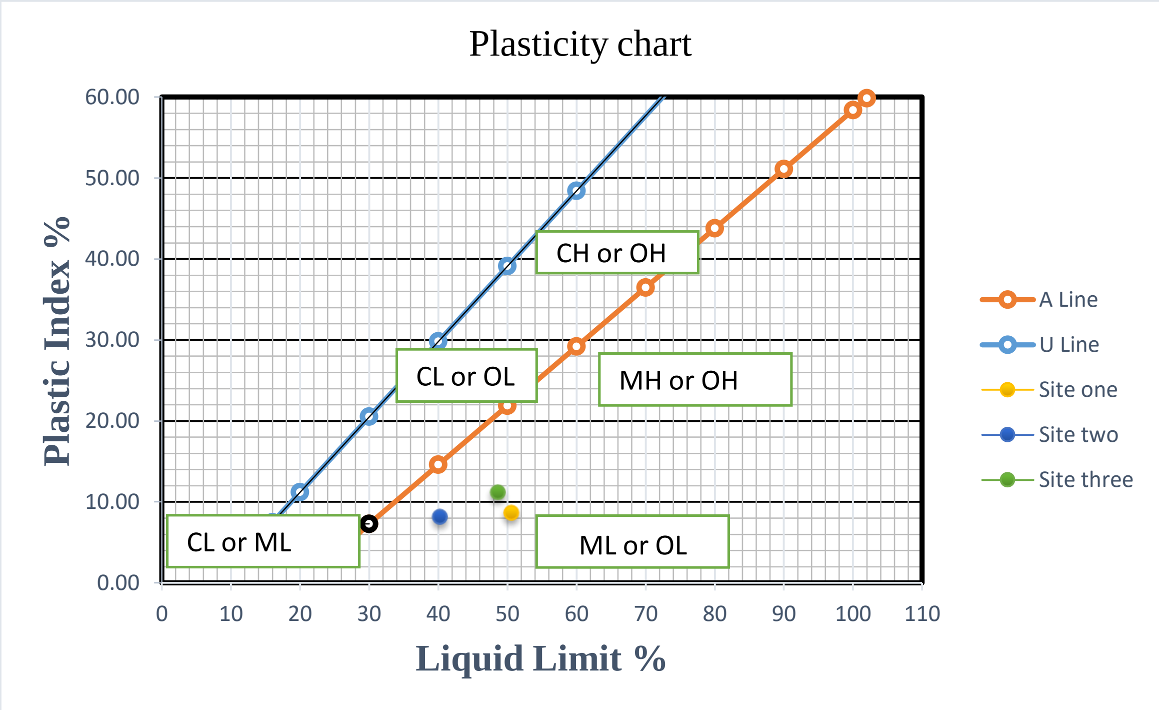

Figure 4. 2 Plasticity Chart of the fine-grained fraction of the coarse-grained soil of the samples by USCS.

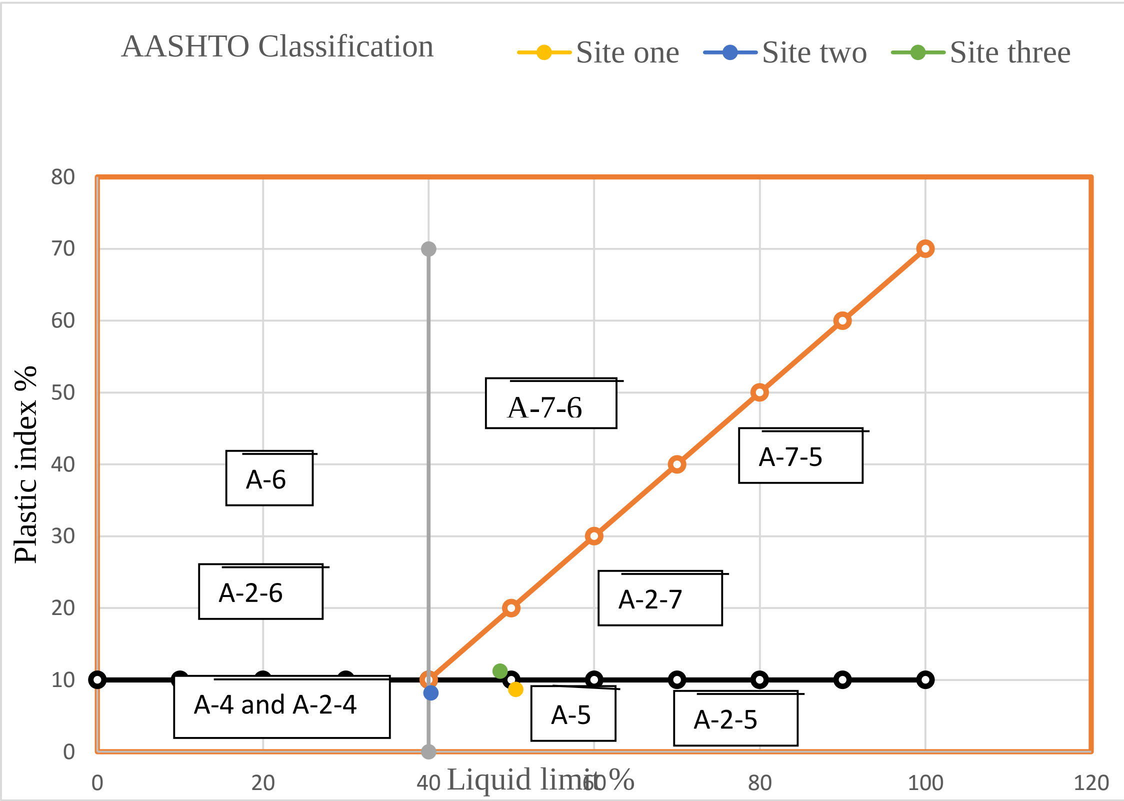

Figure 4. 3 Plasticity Chart of the fine-grained fraction of the coarse-grained soil of the samples by AASHTO classification system.

Table 4. 6 Classification of the soils according to AASHTO

Ser. No:	Location	Sample designation	Nature of sample	GI value	Liquid Limit LL %	Plastic Index PI %	AASHTO Classification	Remark
1	Limmu Genet	Site one	Disturbed	0(-7.99)	50.92	8.7	A-2-5	Silty or clayey gravel and sand
2	Babu	Site two	Disturbed	0(-4.95)	40.25	8.21	A-2-5	Silty or clayey gravel and sand
3	Ambuye entrance	Site three	Disturbed	0(-6.61)	48.6	11.77	A-2-7	Silty or clayey gravel and sand

4.6 Compaction test result

This test was performed by automatic compaction machine for each soil sample of five different 5kg sample. Modified Proctor test (Procedure C) is selected depending on the grain size distribution criteria listed in table 2.7. From table 4.3 one can realize the maximum dry density of site one, site two and site three is 1.39 g/cm^3, 1.83 g/cm^3, and 1.48 g/cm^3 respectively. Figure 4.5 shows the relationship between the dry density, moisture content and saturation line (zero air void line).

Table 4. 7 Compaction test results of the modified method test.

Compaction test results								
Site one			Site two			Site three		
W_c (%)	γd g/cm^3	γd Sat. line	W_c (%)	γd g/cm^3	γd Sat. line	W_c (%)	γd g/cm^3	γd Sat. line
26.64	1.31	1.56	8.33	1.75	2.12	22.26	1.45	1.68
27.94	1.31	1.53	12.00	1.78	1.97	24.97	1.47	1.61

30.39	1.39	1.47	14.59	1.83	1.87	27.22	1.48	1.55
34.27	1.34	1.39	18.57	1.73	1.74	29.85	1.47	1.49
37.68	1.27	1.33	20.13	1.65	1.70	31.17	1.42	1.46

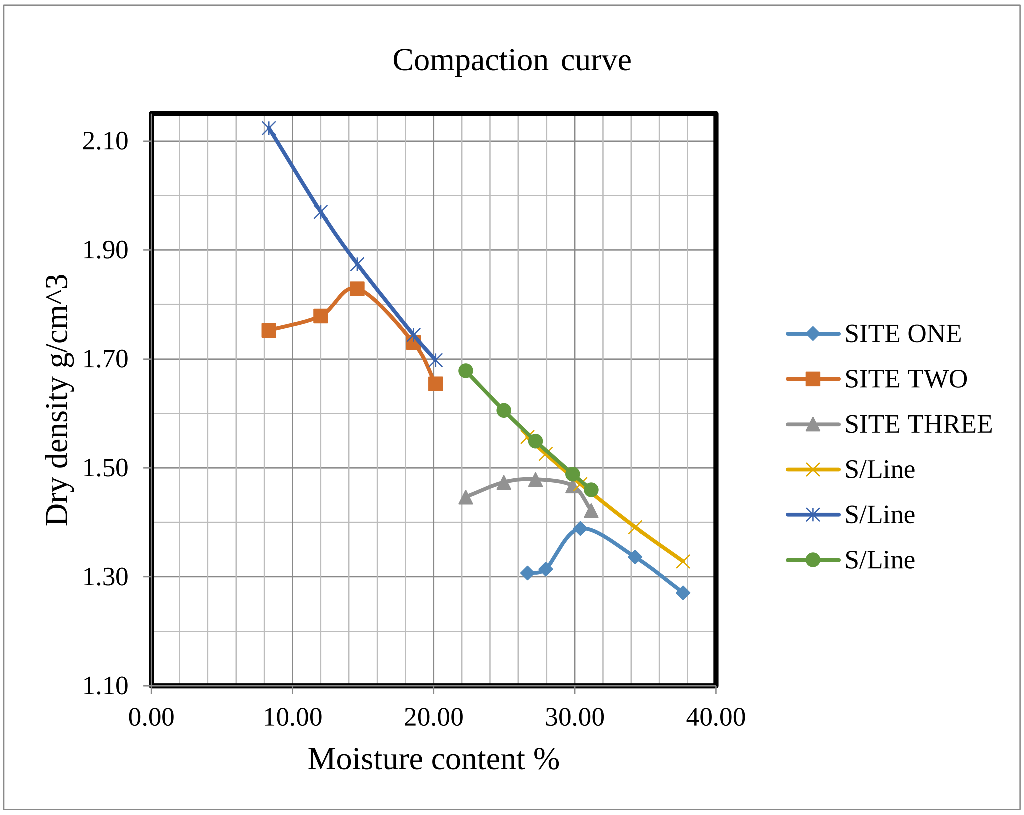

Figure 4. 4 Compaction curve of density- moisture relation.

4.7 California Bearing Ratio (CBR) test

The CBR value is determined by an empirical penetration test devised by the California State Highway Department (U.S.A.), and derives its name thereof. The results obtained by these tests are used in conjunction with empirical curves, based on experience, for the design of flexible pavements. It is defined as the rate of the force per unit area required to penetrate a soil mass with a standard circular plunger of 50 mm diameter at the rate of 1.25 mm/min to that required for the corresponding penetration of a standard material. The standard material is crushed stone, and the load which has been obtained from a test on it is the standard load, this material is considered to have a CBR of 100%. The CBR value is usually determined for penetrations of 2.5 mm and 5 mm.

Where the ratio at 5 mm is consistently higher than that at 2.5 mm, the value at 5 mm is used. Otherwise, the value at 2.5 mm is used, which is more common (C.Venkatrmaiah, 2006).

From figure 4.6 it can be concluded that at the 95% of the maximum dry density of each sample, the respective value of CBR is obtained from three point method. Accordingly site one has a CBR value of 40% at 1.32 g/cm^3, site two has a CBR value of 24% at 1.74 g/cm^3, and site three has a CBR value of 34% at 1.41 g/cm^3 after four-day soaking. ERA manual recommends a minimum 30% of CBR value for selected material as a subbase. Therefore site two failed to fulfill the criteria. It can be used as a good subgrade material.

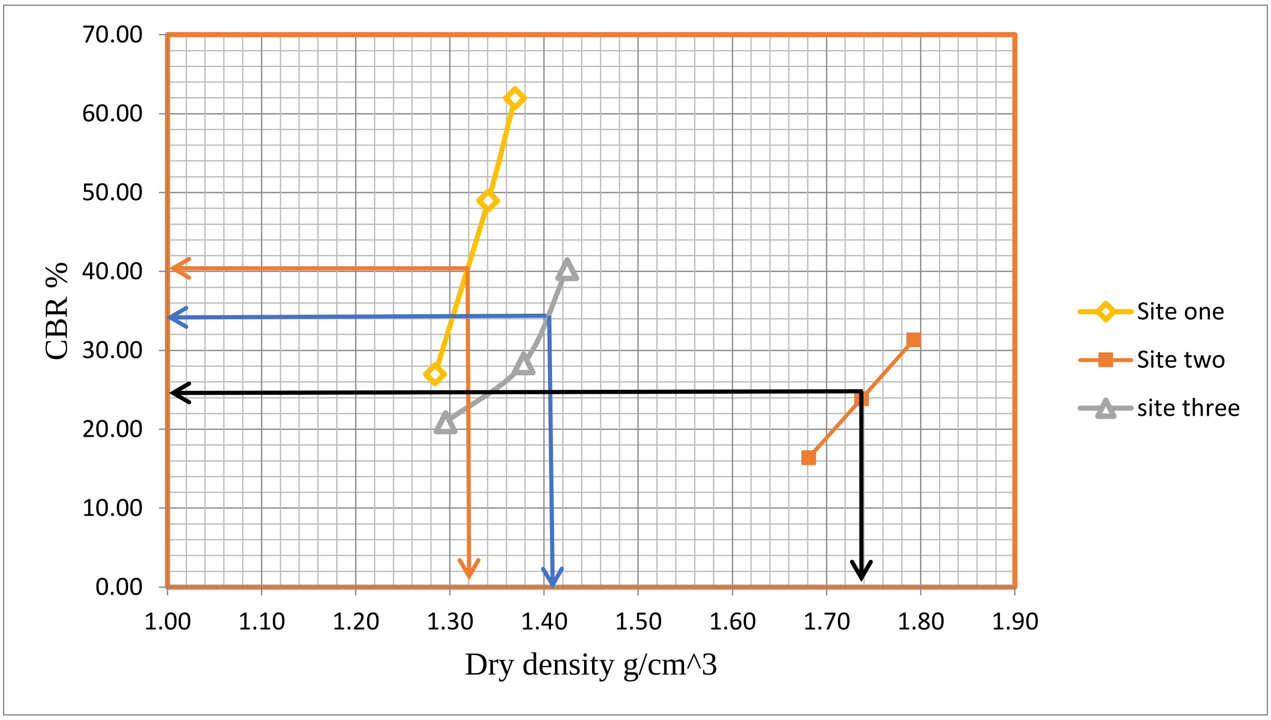

Figure 4. 5 The relationship between CBR and dry density.

4.7.1 Load-penetration curve

The load versus penetration curve is plotted above. This curve will be mainly convex upwards although the initial portion of the curve may be concave upwards due to surface irregularities. A correction shall then be applied by drawing a tangent to the upper curve at the point of contra-flexure. The corrected curve shall be taken to be this tangent plus the convex portion of the original curve with the origin of penetrations shifted to the point where the tangent cuts the horizontal penetration axis.

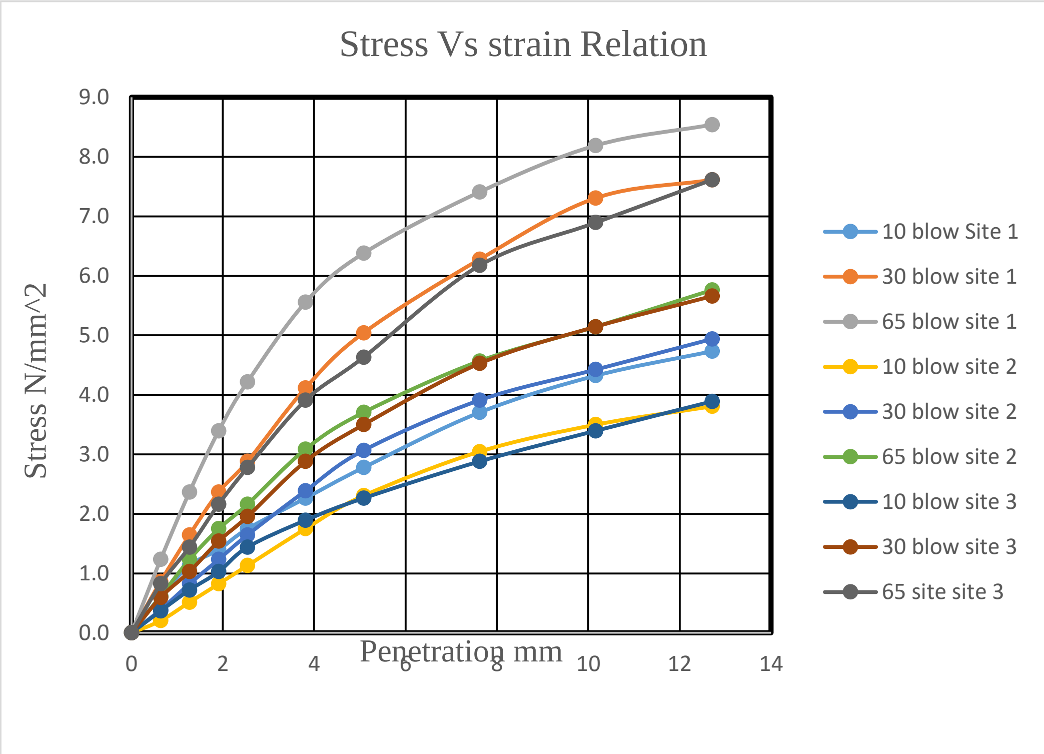

Figure 4. 6 Load-penetration curves.

Table 4. 8 Experimental results of CBR penetration

Penetration mm	Stress Vs. strain relation of the three site after soaking for four days.								
	Site one			Site two			Site three		
	Stress of 10 blows	Stress of 30 blows	Stress of 65 blows	Stress of 10 blows	Stress of 30 blows	Stress of 65 blows	Stress of 10 blows	Stress of 30 blows	Stress of 65 blows
0	0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.64	0.62	0.86	1.23	0.21	0.39	0.62	0.37	0.60	0.82
1.27	1.13	1.65	2.37	0.51	0.82	1.23	0.72	1.03	1.44
1.91	1.40	2.37	3.40	0.82	1.23	1.75	1.03	1.54	2.16
2.54	1.75	2.88	4.22	1.13	1.65	2.16	1.44	1.96	2.78
3.81	2.26	4.12	5.56	1.75	2.39	3.09	1.89	2.88	3.91
5.08	2.78	5.04	6.38	2.31	3.07	3.70	2.26	3.50	4.63
7.62	3.70	6.28	7.41	3.05	3.91	4.57	2.88	4.53	6.17
10.16	4.32	7.31	8.19	3.50	4.42	5.15	3.40	5.15	6.89
12.71	4.73	7.61	8.54	3.81	4.94	5.76	3.89	5.66	7.61

4.8 The swelling potential of the materials

4.8.1 Direct determination of swelling potential from CBR test.

The swell percentage is investigated after soaking for four days during laboratory CBR test. After the fourth day, the swell of site one, site two and site three ranges between 0.00%-0.20%, 0.43%-0.50% and 0.26%-0.37% for 65 blows, 30 blows and 10 blows of compaction per layer. The following tables are the summary of swell laboratory test from CBR test.

Table 4. 9 Swelling test result for site one or Limmu Genet entrance

Day of month	Elapse time (Day)	Mould 1(10 blows)			Mould 2 (30 blows)			Mould 3 (65 blows)		
		Guage reading	Swell		Guage reading	Swell		Guage reading	Swell	
			mm	%		mm	%		mm	%
22/09/2017	0	0	0.23	0.20	0	0.07	0.06	0	0	0.00
26/09/2017	4	23			7			0		

Table 4. 10 Swelling test result for site two or Babu near Kebena forest

SWELL DATA OF SITE TWO										
Day of month	Elapse time (Day)	Mould 1 (10 blows)			Mould 2 (30 blows)			Mould 3 (65 blows)		
		Guage reading	Swell		Guage reading	Swell		Guage reading	Swell	
			mm	%		mm	%		mm	%
22/09/2017	0	0	0.585	0.50	0	0.5252	0.45	0	0.5	0.43
26/09/2017	4	58.5			52.52			50		

Table 4. 11 Swelling test result for site three or Ambuye in Horizon coffee plant.

Day of month	Elapse time (Day)	Mould 1 (10 blows)			Mould 2 (30 blows)			Mould 3 (65 blows)		
		Guage reading	Swell		Guage reading	Swell		Guage reading	Swell	
			mm	%		mm	%		mm	%
22/09/2017	0	5	0.37	0.32	1	0.31	0.27	0	0.3	0.26
26/09/2017	4	37			30.99			30		

4.8.2 Indirect determination of swelling potential

According to Seed, Woodward and Lundgreen (1974), plasticity index is a parameter which can be used as a preliminary indicator of the swelling characteristics of a soil. The indirect estimation of swelling characteristics shows that all samples have low degree of swelling or all samples are inactive since the clay content in the materials is insignificant.

Table 4. 12 Degree of swelling using plastic index test result

Ser. No:	Location	Sample designation	Liquid limit LL %	Plastic limit PL%	Plastic Index PI %	Degree of swelling
1	Limmu Genet	Site one	50.92	41.81	8.70	Low
2	Babu	Site two	40.25	32.04	8.21	Low
3	Ambuye	Site three	48.60	36.83	11.77	Low

4.9 Discussions of the Laboratory Test Results

The natural moisture content of soils of the study area ranges from 10.89%- 41.20%. The dry density and bulk density of the sites range from 1.142 g/cm^3 – 2.021 g/cm^3 and 1.604 g/cm^3 - 2.243 g/cm^3 respectively.

The average specific gravity of site one, site two and site three is 2.66, 2.58 and 2.68 respectively, i.e., the specific gravity of site three is the maximum, and that of site two is the minimum specific gravity. This implies construction of subbase layer by soil sample of site two is more prone to erosion and scour. According to Bowles (1996), the soil sample of site one and site three is a gravel material whereas soil sample from site two has a fine clay content.

From the research, the result of Atterberg Limit test of the soil samples of three different quarry sites shows the samples are medium plastic material since the plastic index ranges from 7 to 17% according to Murthy (1994). This plastic index value lies between 6 and 12% which ERA manually recommends for sub base material. The soil sample in the research area has liquid limit ranging from 40.25% – 50.92 %, plastic limit ranges from 32.04% – 41.81% and plastic index from 8.21% – 11.77%. This means the consistency of the soil is medium plastic.

After grain size distribution test analysis, uniformity coefficient (C_u) and coefficient of curvature (C_c) are determined from the semi-logarithmic graph of percent finer versus sieve size (mm). Site one, site two and site three has the uniformity coefficient (C_u) of 23.08, 306.67 and 88.88 and coefficient of curvature (C_c) of 0.923, 2.32 and 4.018 respectively. For these respective sites, the gradation classification is GP, GW-GM, and GP-GM according to USCS soil classification system and also the samples classified under silty or clayey gravel and sand according to AASHTO classification system. Finally, this test has a great role in the selection of the procedure for compaction test which stated in table 2.7. Procedure C of modified Proctor test was selected due to more than 20% of the grain size distribution of all samples is retained on sieve size 19mm.

The compaction test result showed that maximum dry density (MDD) of the study area ranges from 1.39 to 1.83g/cm^3 and the optimum moisture content (OMC) ranges from 14.5%to 31%. This implies soil samples have a moderate dry density to be classified under the granular material. At 95% of the maximum dry density of each sample, the respective soaked CBR value is obtained.

Accordingly site one has a CBR value of 40% at 1.32 g/cm^3, site two has a CBR value of 24% at 1.74 g/cm^3, and site three has a CBR value of 34% at 1.41 g/cm^3 after four-day soaking. ERA manual recommends a minimum 30% of CBR value for selected material as a subbase. Therefore, site two failed to fulfill the criteria. It can be used as a good subgrade material.

The indirect estimation of swelling characteristics shows that all samples have low degree of swelling or all samples are inactive since the clay content in the materials is insignificant.

4.9 Comparison of Test Results of the three sites with ERA Standard specification

A comparison is made between the soil sample Index property tests and strength tests (CBR) under the investigation. These results were summarized in table 4.13. From the table, it can be summarized the natural moisture content of the study sample ranges from 10.95% to 40.45%. The moisture content in descending order is of site one (40.45%), site three (36.09%) and site two (10.95%). The specific gravity of site three and site two is the maximum (2.68) and minimum (2.58) specific gravity of the study area. This indicates the probability of flood scouring is greater in the construction of subbase layer by soil sample of site two.

The main parameters that affect the suitability of unbound material especially subbase layer are the plastic index and the CBR value from both laboratory test and gradation. Accordingly, through the investigation of the research, it was found that the plastic index (PI) of samples lies between six and twelve. For the most suitable subbase layer plastic index should be less than six. But ERA standard allows a maximum PI value of 12%. The material till can be used as subbase based the PI value.

The CBR value of site two (in Babu Kebele) is 24% which did not pass the standard (ERA manual recommends at least the CBR value from laboratory test after four soaking should 30%). The swelling potential of this site was also larger than the others. Therefore the material from this is not suitable for subbase to be used as the select material for road construction. The material can be used as it is for subgrade improvement as capping layer without stabilization and for subbase layer after stabilization.

Table 4. 13 Comparison of test result of the three sites with ERA manual

Soil properties	**Comparison of Test Results of the three sites with ERA Manual**				
	Site one	**Site two**	**Site three**	**ERA specification for subbase**	**Remark**
Average NMC %	40.45	10.95	36.09	-	
Bulk unit weight	1.604	2.243	1.836	-	
Dry unit weight	1.142	1.021	1.348	-	
Specific gravity	2.66	2.58	2.68	-	
Liquid Limit %	50.92	40.25	48.60	-	
Plastic Limit %	41.81	32.04	36.83	-	
Plastic Index %	8.70 (PP = 26.1)	8.21 (PP =81.44)	11.77 (PP = 97.10)	6-12	Medium plastic (Murthy,1994)
Classification					
USCS method	poorly graded gravel with sand (GP)	well-graded gravel with silt and sand (GW-GM)	poorly graded gravel with silt and sand (GP-GM)	The AASHTO classification system	The groups have GI of zero which shows the material is good (not sufficient criteria)
AASHTOO method	A-2-5 (Silty or clayey gravel and sand)	A-2-5 (Silty or clayey gravel and sand)	A-2-7 (Silty or clayey gravel and sand)		
Compaction					
MDD (g/cm3)	1.39	1.83	1.48	-	Sample from site two has high MDD
OMC %	31.00	14.50	27.20	-	Sample from Site two highly affect by moisture than others
CBR TEST %					
Lab. CBR %	40.00	24.00	34.00	CBR should be >30	Site two failed the criteria
CBR test Swell %	0.00-0.20	0.43-0.50	0.26-0.37	-	Site two has high swelling potential than others
Indirect swell degree	low	low	low		All samples are good

CHAPTER FIVE
CONCLUSION AND RECOMMENDATION

5.1 Conclusion

From the laboratory test, it is observed that the soils from these quarry sites in the study area are gravel soil. The gravel content and sand content takes the significant percentage of the grain size analysis of the samples.

The plastic product (PP = plastic index * percent finer than 0.0075 mm sieve size should be less 75 according to ERA specification. This did not fulfilled except for only site one sample which indicates the sample is more suitable than the others.

Site one to site three, the gradation classification is GP, GW-GM, and GP-GM according to USCS soil classification system respectively and also the samples classified under silty or clayey gravel and sand (A-2-5 and A-2-7) according to AASHTO classification system.

According Rollings and Rollings (1996), for subbase material of silty gravel ranging a CBR value of between 20 and 60 percent the load Support and Drainage Characteristics of the material is classified under good support and fair drainage, characteristics with moderate frost potential.

The average specific gravity of site one, site two and site three is 2.66, 2.58 and 2.68 respectively, i.e., the specific gravity of site three is the maximum, and that of site two is the minimum specific gravity. This implies construction of subbase layer by soil sample of site two is more prone to erosion and scour.

The CBR value of laboratory test of site one, site two and site three is 40%, 24% and 34% respectively. The CBR value implies that site one sample is more suitable for subbase material than the others according to ERA standard.

The swell percentage is investigated after soaking for four days during laboratory CBR test. After the fourth day, the swell of site one, site two and site three ranges between 0.00%-0.20%, 0.43%-0.50% and 0.26%-0.37% for 65 blows, 30 blows and 10 blows of compaction per layer. This implies that the swelling of sample site one is less than the other which minimize defect of flexible pavement like cracking, pothole and others. But the indirect estimation of swelling characteristics shows that all samples have low degree of swelling i.e. all samples are inactive since the clay content in the materials is insignificant.

5.2 Recommendations

Based on the findings of this research, the following recommendations are forwarded:

The research strongly recommends that the selected material from site two is not an effective material for selected subbase material due to the CBR value of the sample is under ERA's specification requirement and the swelling potential of this sample also is more than the others from direct CBR test. Therefore this material cannot be used for selected subbase material as it is. The material can be used as subgrade improvement capping layer without any stabilization and for subbase layer after stabilizing by other gravel material or chemical stabilizers.

In-situ CBR by DCP test should be conducted according to ERA specification during the construction period. ERA standard specifications recommends 70% of in-situ CBR and a minimum of DCP test result of 4mm penetration of per blow for selected subbase material.

From the factors that affect the strength of subbase material during construction period is compaction effort. The research found that the soil samples are well-graded and poorly graded gravel material. According Rollings and Rollings 1996, for well-graded and poorly graded gravel (GW, GP) vibratory field compaction equipment is primarily recommended and pneumatic and impact field compaction equipment are used as choice. Grid is useful for oversized particles. And also the materials are low plastic silty gravel especially site two and site three. Accordingly moisture control often critical for silty soils.

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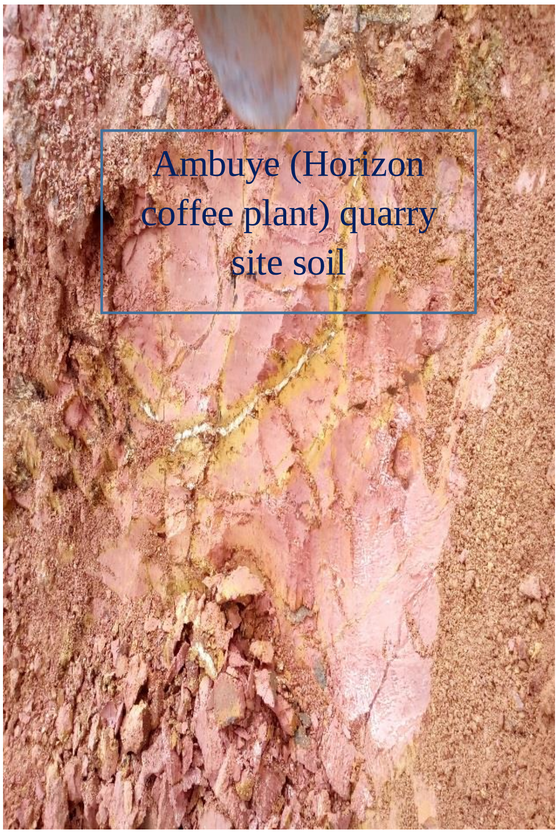
Appendix-A

Natural moisture content and Unit weight test result

Site	Site one		
Trial No.	1	2	3
Weight of Empty container(g)	17.996	18.212	17.483
Weight of container + wet soil(g)	84.183	101.124	96.594
Weight of container + Oven dry soil(g)	64.868	77.451	73.91
Weight of water(W_w) (gm)	19.315	23.673	22.684
Weight of Oven dry soil(W_s)gm	46.872	59.239	56.427
Water content (W)(%)	41.208	39.962	40.201
Average water content (W)(%)	40.457		

Site	Site two		
Trial No.	1	2	3
Weight of Empty container(g)	17.593	17.2431	17.9513
Weight of container + wet soil(g)	112.589	113.818	119.300
Weight of container + Oven dry soil(g)	103.125	104.349	109.346
Weight of water(W_w) (gm)	9.464	9.469	9.954
Weight of Oven dry soil(W_s)gm	85.532	87.1059	91.3947
Water content (W)(%)	11.065	10.871	10.891
Average water content (W)(%)	10.942		

Site	Site three		
Trial No.	1	2	3
Weight of Empty container(g)	17.5	17.758	17.366
Weight of container + wet soil(g)	75.756	98.74	98.808
Weight of container + Oven dry soil(g)	60.023	77.405	77.136
Weight of water(W_w) (gm)	15.733	21.335	21.672
Weight of Oven dry soil(W_s)gm	42.523	59.647	59.77
Water content (W)(%)	36.999	35.769	36.259
Average water content (W)(%)	36.342		



Ambuye (Horizon coffee plant) quarry site soil

Unit weight of the soil sample

Calibration of density of sand	
Mass of cylinder (g)	870
Mass of cylinder + sand (g)	3740
Mass of sand (g)	2870
Volume of cylinder cm3	1869.64
Density of sand (g/cm3)	1.535

Site	1	2	3
Mass of sand In filled jar (g)	8425	9725	9550
Mass of sand in partially filled jar (g)	3795	5105	4895
Mass in hole + cone + baase plate (g)	4630	4620	4655
Mass of sand in cone + base plate (g)	1845	1845	1845
Mass of sand in hole (g)	2785	2775	2810
Mass of soil extracted from the hole (g)	2910	4055	3360
Water content (%)	40.45	10.95	36.09
Wet density of soil (g/cm3) $= \left(\frac{Density\ of\ sand * mass\ of\ soil}{mass\ of\ sand}\right)^1$	1.604	2.243	1.836
Dry density of soil (g/cm3) = wet density/(1+w)	1.142	2.022	1.349

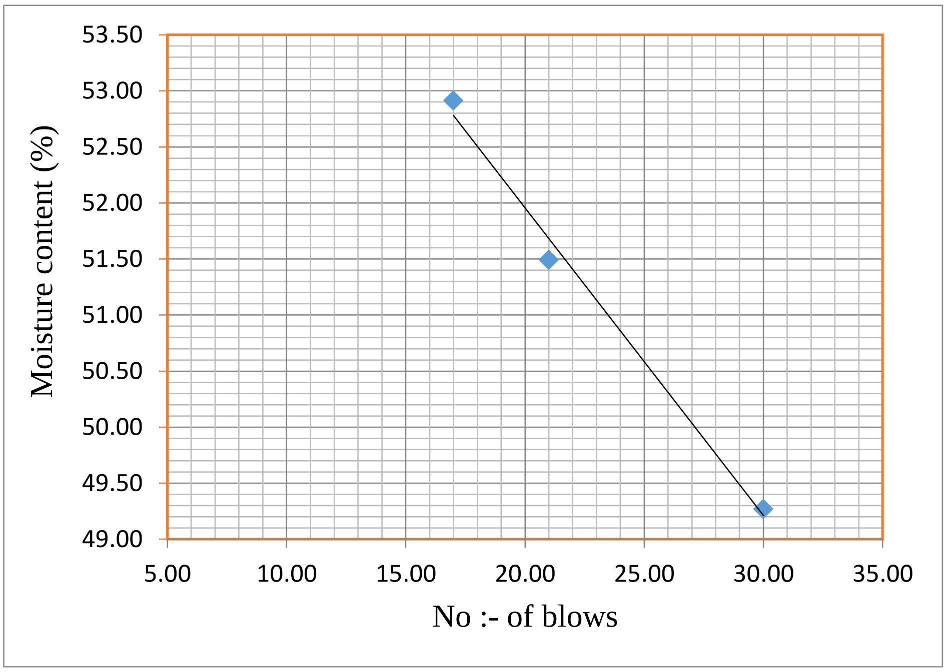
Appendix - B

Index properties test results

Atterberg's limit

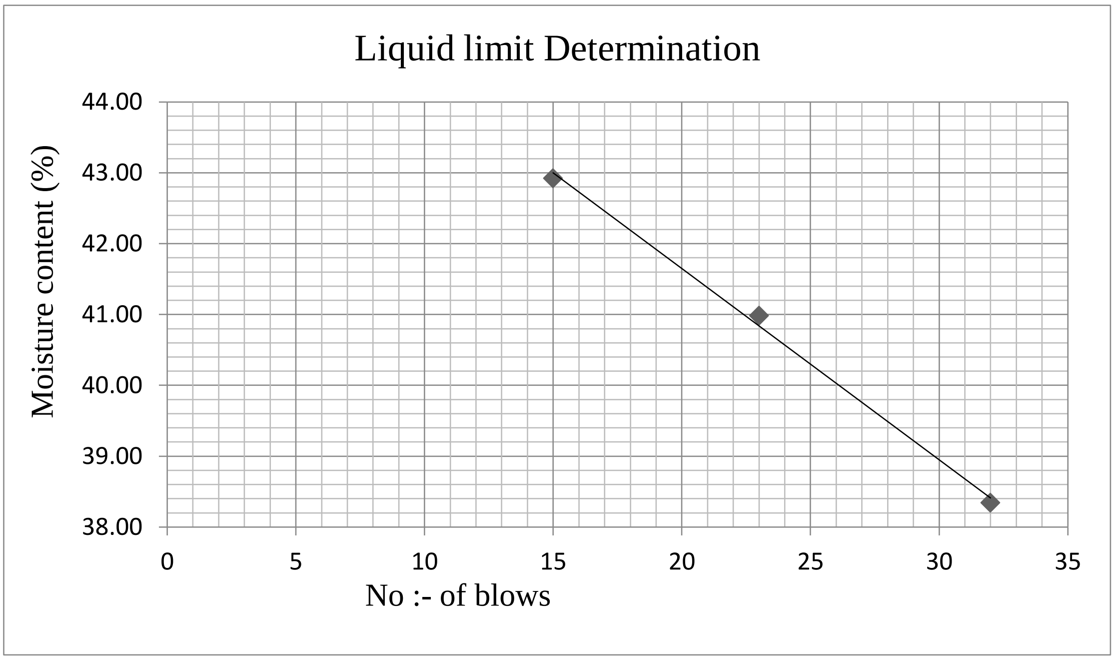
The sample of Site one (Limmu Genet):

	Liquid limit			Plastic limit		
Determination No.	1	2	3		1	2
Number of drops	30	21	17			
Can No.	A	B	C		A	B
Mass of can + Moist Soil, g	45.51	36.85	37.9		16.72	13.99
Mass of can + dry soil, g	36.73	30.29	31.18		13.76	11.69
Mass of can, g	18.91	17.55	18.48		6.68	6.19
Mass of water	8.78	6.56	6.72		2.96	2.3
Mass of dry soil, g	17.82	12.74	12.7		7.08	5.5
Moisture content, w (%)					41.80	41.82
From the flow curve , LL @ 25 blow is = 50.52%				Average water content= 41.81%		
				Plastic limit, PL= 41.81%		
Plastic index, LL-PL= 8.7%						



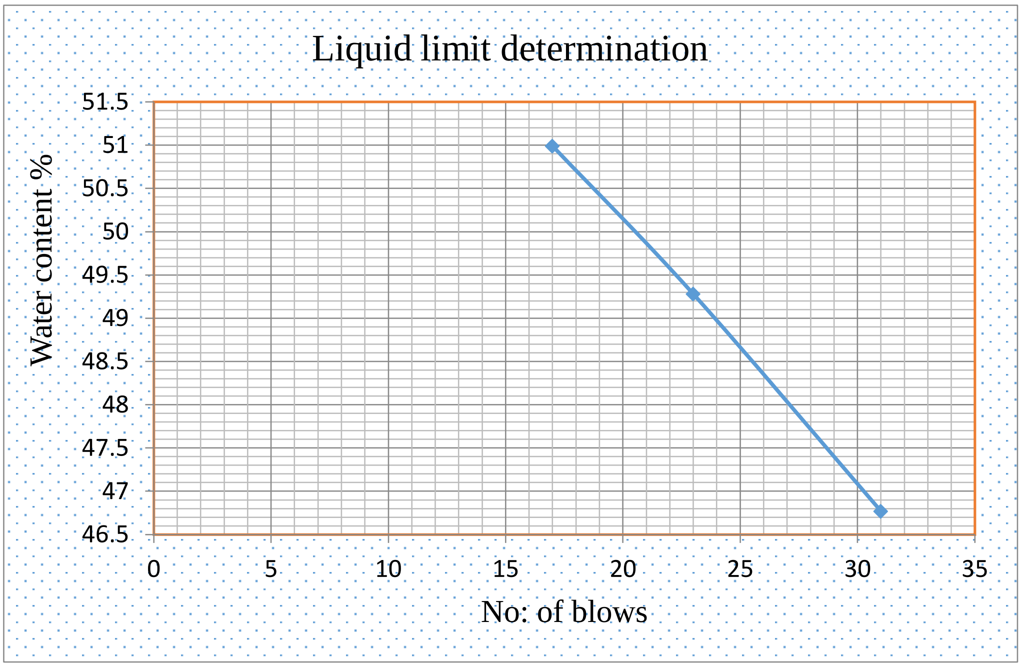
The sample of Site two (Babu):

	Liquid limit			Plastic limit		
Determination No.	1	2	3		1	2
Number of drops	30	21	17			
Can No.	A	B	C		A	B
Mass of can + Moist Soil, g	51.08	46.14	43.02		12.33	12.52
Mass of can + dry soil, g	41.82	37.89	35.65		10.69	10.75
Mass of can, g	17.67	17.76	18.48		5.55	5.5
Mass of water	9.26	8.25	7.37		1.64	1.77
Mass of dry soil, g	24.15	20.13	17.17		5.14	5.5
Moisture content, w (%)	38.34	40.98	42.92		31.91	32.18
From the flow curve , LL @ 25 blow is = 40.25%				Average water content= 32.04%		
				Plastic limit, PL= 32.04%		
Plastic index, LL-PL= 8.21%						



The sample of Site three (Ambuye):

	Liquid limit			Plastic limit		
Determination No.	1	2	3	1	2	3
Number of drops	31	23	17			
Can No.	A	B	C	A	B	C
Mass of can + Moist Soil, g	61.06	56.60	64.20	12.55	12.43	13.42
Mass of can + dry soil, g	47.64	43.9	48.80	10.76	10.55	11.18
Mass of can, g	18.95	18.13	18.60	5.83	5.60	5.26
Mass of water	13.42	12.70	15.40	1.80	1.88	2.24
Mass of dry soil, g	28.69	25.77	30.2	4.93	4.95	6.22
Moisture content, w (%)	46.77	49.28	50.99	36.51	37.97	36.01
From the flow curve , LL @ 25 blow is = 48.60%				Average water content = 36.83%		
				Plastic limit, PL= 36.83%		
Plastic index, LL-PL= 11.77%						



Specific gravity determination

The sample of Site one (Limmu Genet):

S. No.	Observation Number	1	2	3
1	Weight of density bottle (W_1 g)	28.912	27.515	28.224
2	Weight of density bottle + dry soil (W_2 g)	39.115	37.552	38.218
3	Weight of bottle + dry soil + water at temperature $T_x{}^0$C (W_3 g)	86.33	84.577	85.956
4	Tx ^{o} C	24	24	24
5	Weight of bottle + water (W_4 g) at temperature $T_i{}^0$ C	79.936	78.362	79.743
6	T	22	22	22
7	Mass of soil Ms (W_2-W_1 in g)	10.203	10.037	9.994
8	Density of water @ Ti(ρw@ti)	0.9978	0.9978	0.9978
9	Density of water @ Tx(ρw@tx)	0.99732	0.99732	0.99732
10	k	0.9991	0.9991	0.9991
11	ρw@tx/ρw@ti	0.9995	0.9995	0.9995
12	Mpw@tx	79.911	78.338	79.718
13	Specific gravity G at $T_x{}^0$ C	2.694	2.641	2.658
14	Average specific gravity at $T_x{}^0$ C	2.66		

Mpw(at Tx) =[[density of water @ Tx/density of water @ Ti] [Mpw@Ti – Mp]] + Mp

$$G_s = K[\frac{M_s}{M_s + M_{pw}(at T_x) - M_{pws}}]$$

Therefore the specific gravity of the soil is 2.66.

The sample of Site two (Babu):

S. No.	Observation Number	1	2	3
1	Weight of density bottle (W_1 g)	28.210	27.500	28.900
2	Weight of density bottle + dry soil (W_2 g)	38.440	37.570	39.240
3	Weight of bottle + dry soil + water at temperature T_x ^{0}C (W_3 g)	86.090	84.580	86.410
4	Tx ^{o} C	22.5	22.5	22.5
5	Weight of bottle + water (W_4 g) at temperature T_i^0 C	79.750	78.400	80.080
6	T	24.5	24.5	24.5
7	Mass of soil Ms (W_2-W_1 in g)	10.230	10.070	10.340
8	Density of water @ Ti(ρw@ti)	0.9972	0.9972	0.9972
9	Density of water @ Tx(ρw@tx)	0.9978	0.9978	0.9978
10	k	0.9995	0.9995	0.9995
11	ρw@tx/ρw@ti	1.001	1.001	1.001
12	Mpw@tx	79.781	78.431	80.111
13	Specific gravity G at T_x^0 C	2.608	2.567	2.558
14	Average specific gravity at T_x^0 C	2.58		

Mpw(at Tx) =[[density of water @ Tx/density of water @ Ti] [Mpw@Ti – Mp]] + Mp

$$G_s = K[\frac{M_s}{M_s + M_{pw}(at T_x) - M_{pws}}]$$

Therefore the specific gravity of the soil is 2.58.

The sample of Site three (Ambuye):

S. No.	Observation Number	1	2	3
1	Weight of density bottle (W_1 g)	28.210	27.500	28.900
2	Weight of density bottle + dry soil (W_2 g)	38.330	37.800	39.060
3	Weight of bottle + dry soil + water at temperature T_x^0 C (W_3 g)	86.180	84.920	86.390
4	T x0 C	20.500	20.500	20.500
5	Weight of bottle + water (W_4 g) at temperature T_i^0 C	79.810	78.400	80.030
6	Ti	23.000	23.000	23.000
7	Mass of soil (W2-W1 in g)	10.120	10.300	10.160
8	Density of water @ Ti(ρw@ti)	0.99757	0.99757	0.99757
9	Density of water @ Tx(ρw@tx)	0.99812	0.99812	0.99812
10	k	0.9991	0.9991	0.9991
11	ρw@tx/ρw@ti	1.0006	1.0006	1.0006
12	Mpw@tx	79.838	78.428	80.058
13	Specific gravity G at T_x^0 C	2.676	2.702	2.652
14	Average specific gravity at T_x^0 C	2.68		

Mpw(at Tx) =[[density of water @ Tx/density of water @ Ti] [Mpw@Ti – Mp]] + Mp

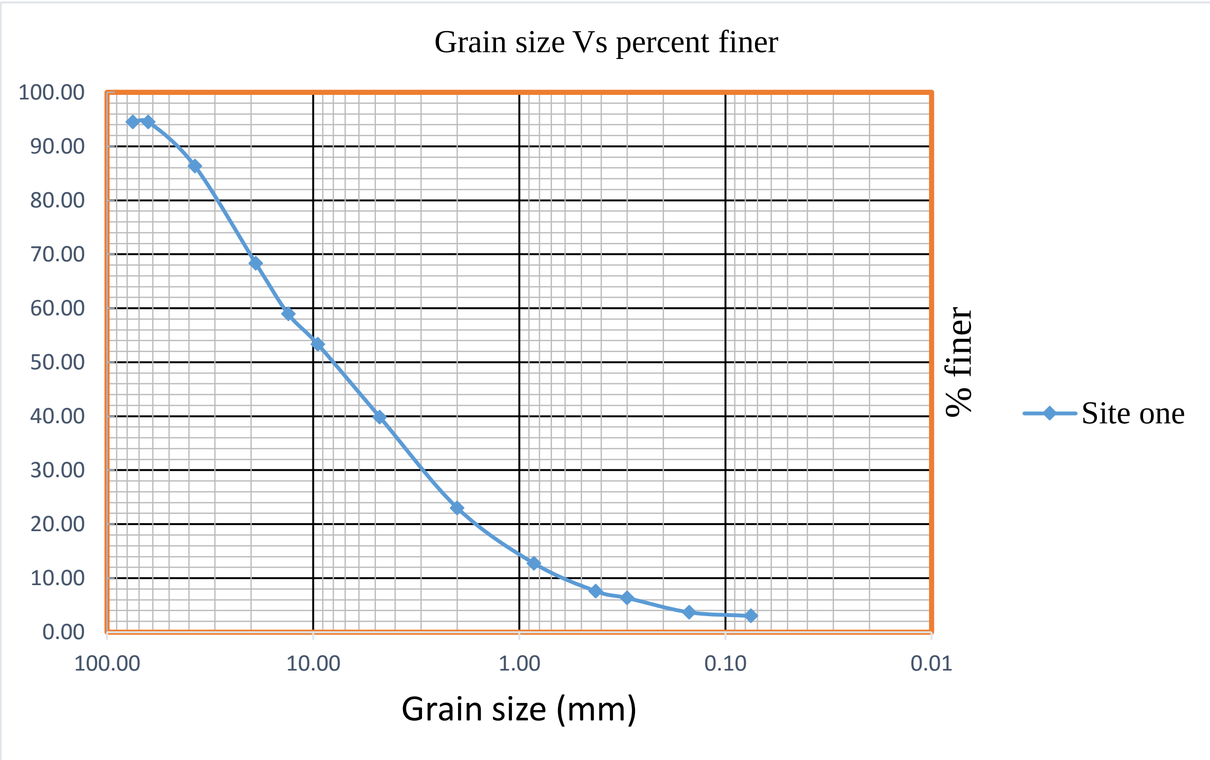
$$G_s = K[\frac{M_s}{M_s + M_{pw}(at T_x) - M_{pws}}]$$

Therefore the specific gravity of the soil is 2.68.

Grain size distribution

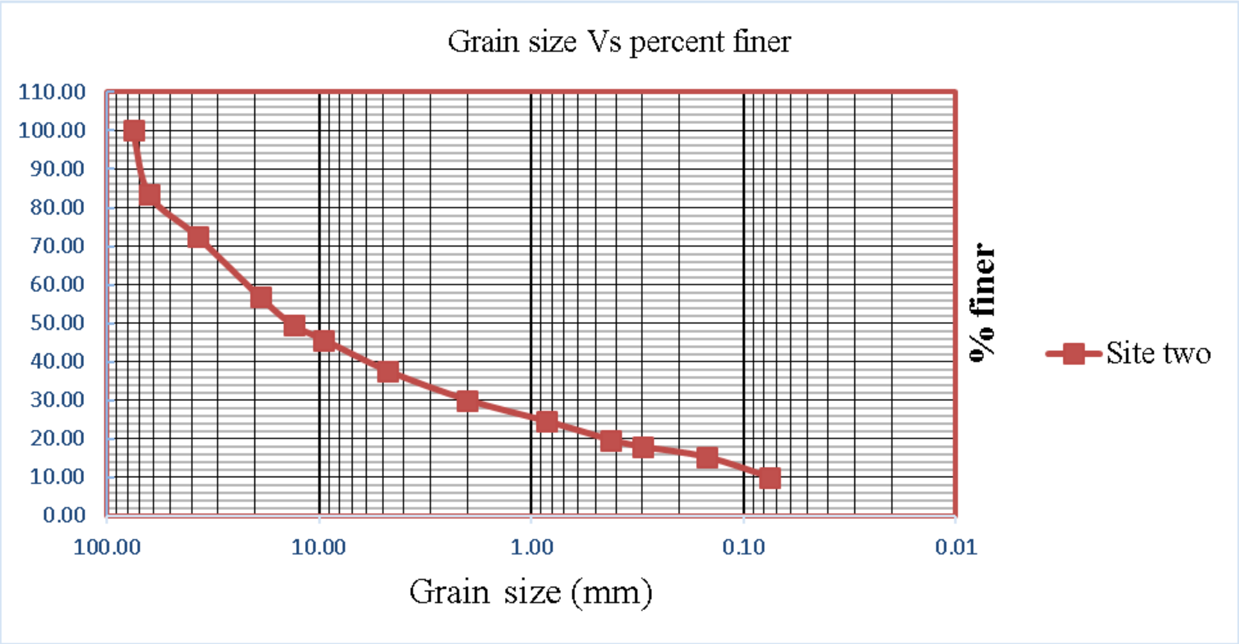
The sample of Site one (Limmu Genet):

Sieve no	Sieve opening (mm)	Mass of soil retained on each sieve, Wn	% retained, Rn	Cumulative percent retained, ∑Rn	% passing = 100-∑Rn
3-in	75	0.33	5.50	5.50	94.50
	63	0.00	0.00	5.50	94.50
	37.5	0.49	8.17	13.67	86.33
¾-in	19	1.08	18.00	31.67	68.33
	13.2	0.57	9.42	41.08	58.92
3/8-in	9.5	0.34	5.58	46.67	53.33
4	4.75	0.81	13.50	60.17	39.83
10	2	1.01	16.83	77.00	23.00
20	0.85	0.62	10.25	87.25	12.75
40	0.425	0.31	5.17	92.42	7.58
50	0.3	0.08	1.25	93.67	6.33
100	0.15	0.16	2.67	96.33	3.67
200	0.075	0.04	0.67	97.00	3.00
pan	-	0.14	2.25	99.25	0.75



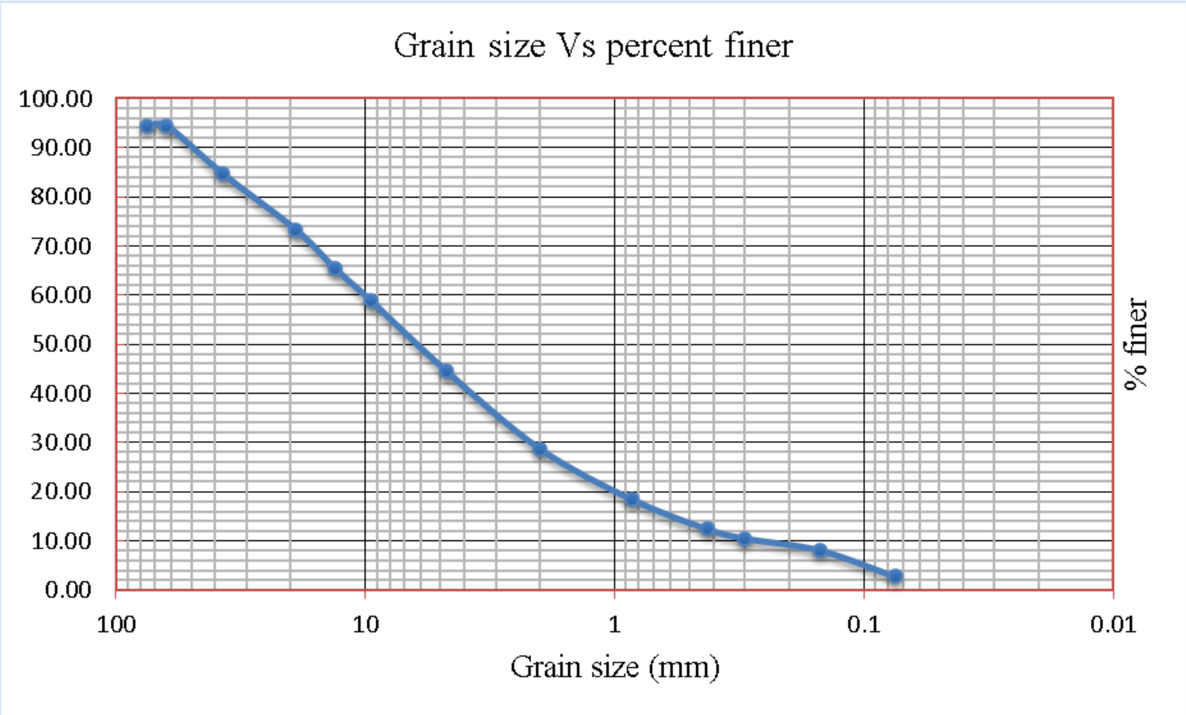
The sample of Site two (Babu):

Sieve no	Sieve opening (mm)	Mass of soil retained on each sieve, Wn	% retained, Rn	Cumulative percent retained, ∑Rn	% passing = 100-∑Rn
3-in	75	0.000	0.00	0.00	100.00
	63	0.995	16.58	16.58	83.42
	37.5	0.650	10.83	27.42	72.58
¾-in	19	0.945	15.75	43.17	56.83
	13.2	0.445	7.42	50.58	49.42
3/8-in	9.5	0.225	3.75	54.33	45.67
4	4.75	0.485	8.08	62.42	37.58
10	2	0.455	7.58	70.00	30.00
20	0.85	0.330	5.50	75.50	24.50
40	0.425	0.290	4.83	80.33	19.67
50	0.3	0.111	1.85	82.18	17.83
100	0.15	0.159	2.65	84.83	15.17
200	0.075	0.315	5.25	90.08	9.92
pan	-	0.593	9.88	99.97	0.03



The sample of Site three (Ambuye):

Sieve no	Sieve opening (mm)	Mass of soil retained on each sieve, Wn	% retained, Rn	Cumulative percent retained, ∑Rn	% passing = 100-∑Rn
3-in	75	0.38	6.25	5.50	94.50
	63	0.00	0.00	5.50	94.50
	37.5	0.59	9.75	15.25	84.75
¾-in	19	0.69	11.50	26.75	73.25
	13.2	0.48	7.92	34.67	65.33
3/8-in	9.5	0.39	6.50	41.17	58.83
4	4.75	0.86	14.25	55.42	44.58
10	2	0.95	15.83	71.25	28.75
20	0.85	0.62	10.33	81.58	18.42
40	0.425	0.36	6.00	87.58	12.42
50	0.3	0.12	1.92	89.50	10.50
100	0.15	0.15	2.42	91.92	8.08
200	0.075	0.32	5.33	97.25	2.75
pan	-	0.03	0.57	97.82	2.18



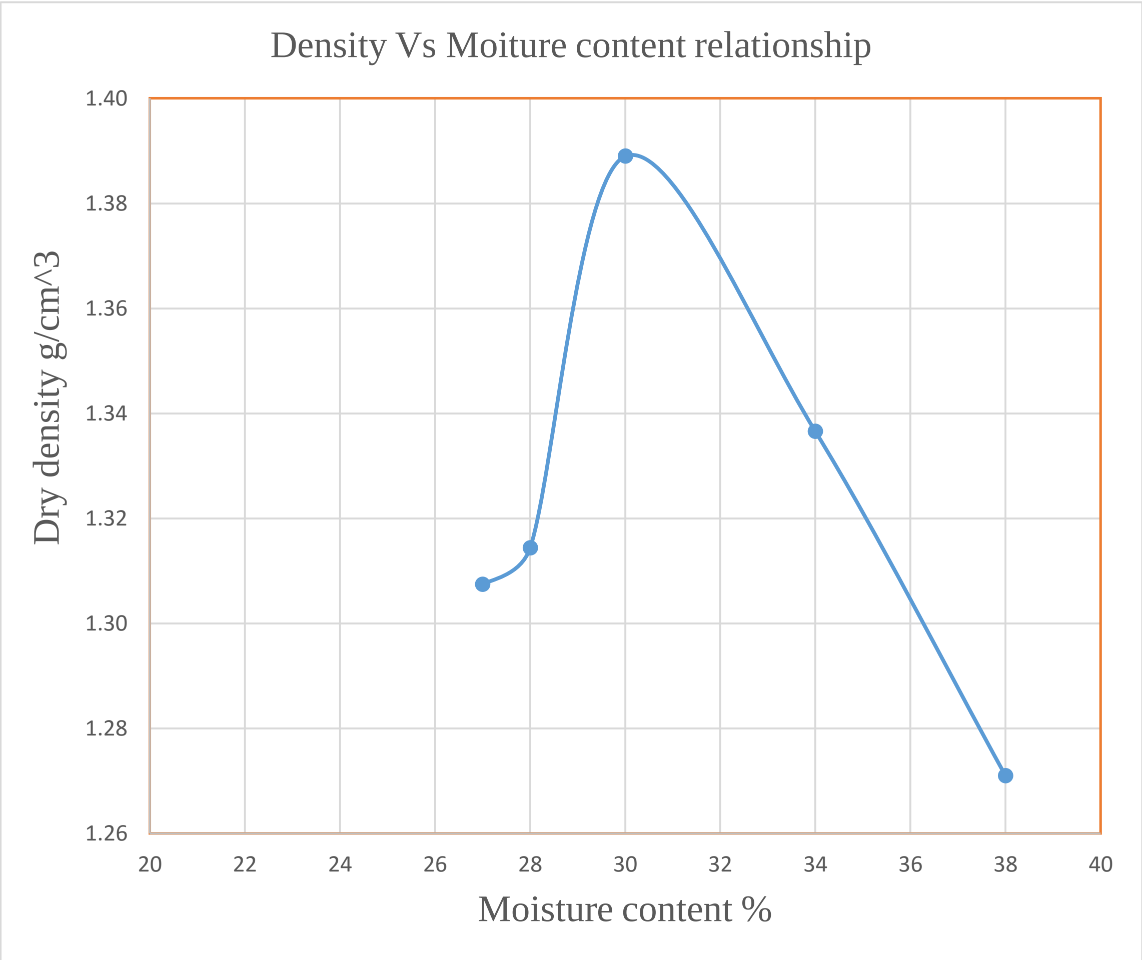
Appendix - C

Compaction test results

The sample of Site one (Limmu Genet):

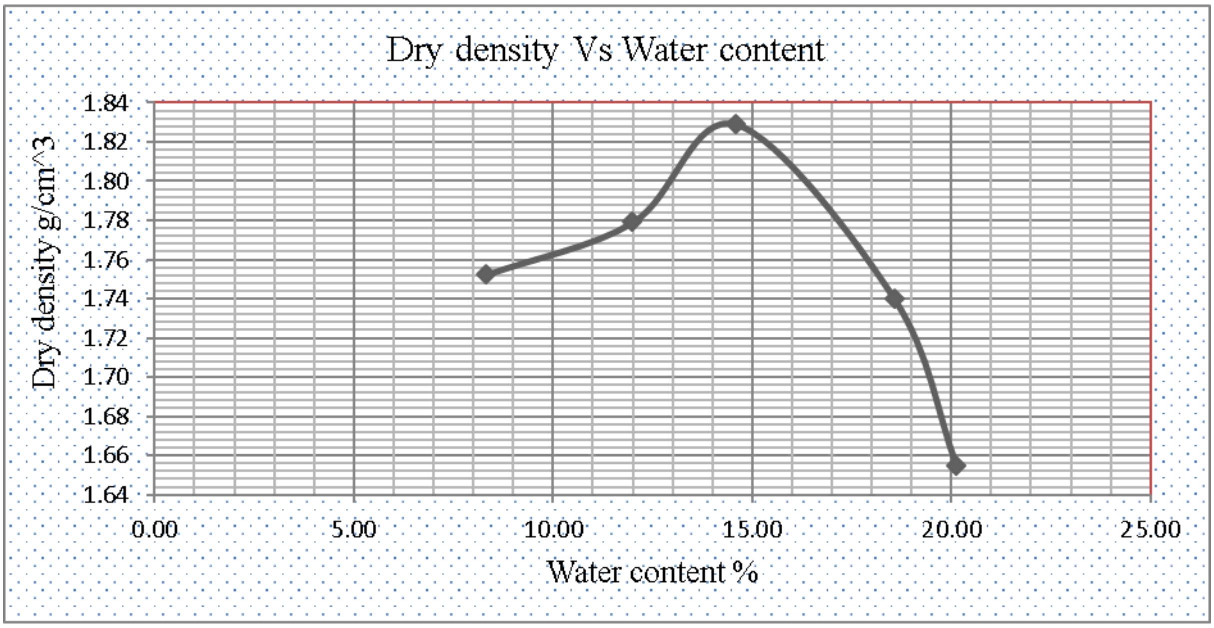
Trial No.	Mass of compacted soil + mold, g	Mass of compacted soil, g = (Msm - Mm)	Moisture content Determination							Bulk density (g/cm³)	Dry unit weight (KN/m³)
			Can No.	Mass of wet soil + can, g	Mass of dry soil + can, g	Mass of water, g	Mass of can, g	Mass of dry soil, g	Moisture content, %		
1	7815	3515	A	95.69	79.17	16.52	17.49	61.68	26.78	1.656	1.31
			B	72.3	60.86	11.44	17.68	43.18	26.49		
2	7870	3570	C	77.45	64.11	13.34	18.18	45.93	29.04	1.682	1.31
			D	98.41	81.42	16.99	18.09	63.33	26.83		
3	8145	3845	E	92.55	75.45	17.10	17.59	57.86	29.55	1.811	1.39
			F	90.9	73.36	17.54	17.20	56.16	31.23		
4	8110	3810	G	110.12	86.8	23.32	17.68	69.12	33.74	1.795	1.34
			H	106.78	84.01	22.77	18.59	65.42	34.81		
5	8015	3715	I	92.13	71.81	20.32	17.64	54.17	37.51	1.750	1.27
			J	115.68	88.72	26.96	17.48	71.24	37.84		

Average Moisture Content %	Dry density g/cm^3
27	1.31
28	1.32
30	1.39
34	1.34
38	1.27



The sample of Site two (Babu):

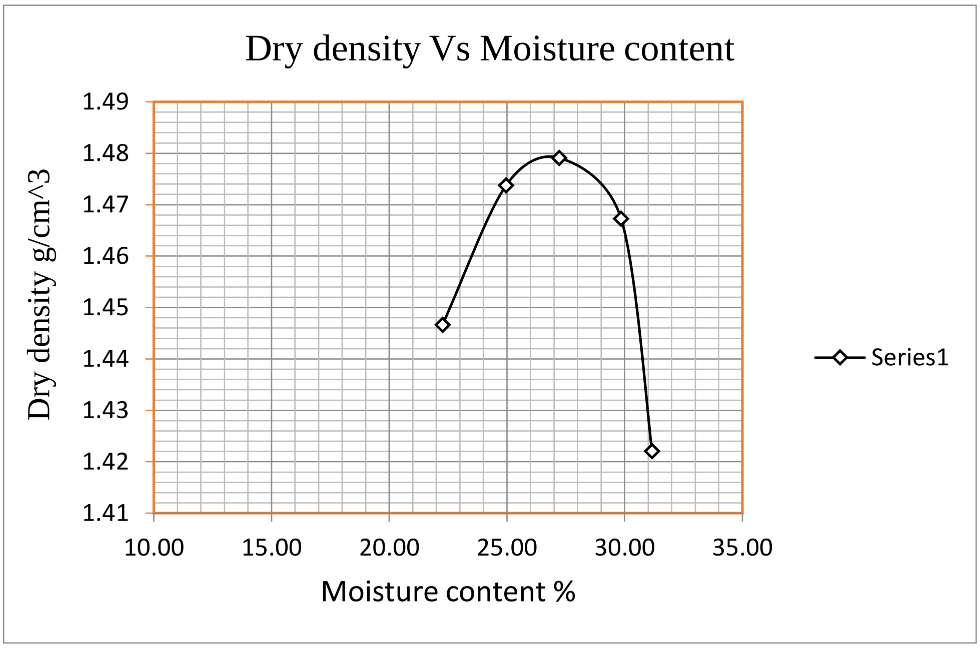
Trial No.	Mass of compacted soil + mold, g	Mass of compacted soil, g = (Msm - Mm)	Moisture content determination							Bulk density (g/cm^3)	Dry unit weight (Kn/m^3)
			Can No.	Mass of wet soil + can, g	Mass of dry soil + can, g	Mass of water, g	Mass of can, g	Mass of dry soil, g	Moisture content, %		
1	8330	4030	12	96.56	90.45	6.11	17.20	73.25	8.34	1.90	1.75
			13	65.82	62.22	3.6	18.90	43.32	8.31		
2	8530	4230	21	96.13	87.6	8.53	17.61	69.99	12.19	1.99	1.78
			22	93.96	85.86	8.10	17.25	68.61	11.81		
3	8750	4450	31	73.55	66.66	6.89	16.96	49.7	13.86	2.10	1.83
			32	73.76	66.28	7.48	17.46	48.82	15.32		
4	8680	4380	41	94.13	82.05	12.08	17.08	64.97	18.59	2.06	1.74
			42	116.72	102.43	14.29	25.35	77.08	18.54		
5	8520	4220	51	112.91	97.05	15.86	17.20	79.85	19.86	1.99	1.65
			52	94.76	81.70	13.06	17.68	64.02	20.40		



The sample of Site three (Ambuye):

Trial No.	Mass of compacted soil + mold, g	Mass of compacted soil, g = (Msm - Mm)	Moisture content determination							Bulk density (g/cm^3)	Dry unit weight (Kn/m^3)
			Can No.	Mass of wet soil + can, g	Mass of dry soil + can, g	Mass of water, g	Mass of can, g	Mass of dry soil, g	Moisture content, %		
1	8055	3755	11	75.52	64.43	11.09	17.30	47.13	23.53	1.77	1.45
			12	85.33	73.69	11.64	18.25	55.44	21.00		
2	8210	3910	21	118.09	98.71	19.38	18.54	80.17	24.17	1.84	1.47
			22	103.22	85.66	17.56	17.50	68.16	25.76		
3	8295	3995	31	75.38	62.72	12.66	17.39	45.33	27.93	1.88	1.48
			32	83.52	69.83	13.69	18.21	51.62	26.52		

4	8345	4045	41	91.33	74.69	16.64	17.72	56.97	29.21	1.91	1.47
			42	112.61	92.22	20.39	25.35	66.87	30.49		
5	8260	3960	51	113.22	90.34	22.88	17.63	72.71	31.47	1.87	1.42
			52	110.77	88.68	22.09	17.11	71.57	30.86		



Appendix - D

California bearing ratio (CBR) test results

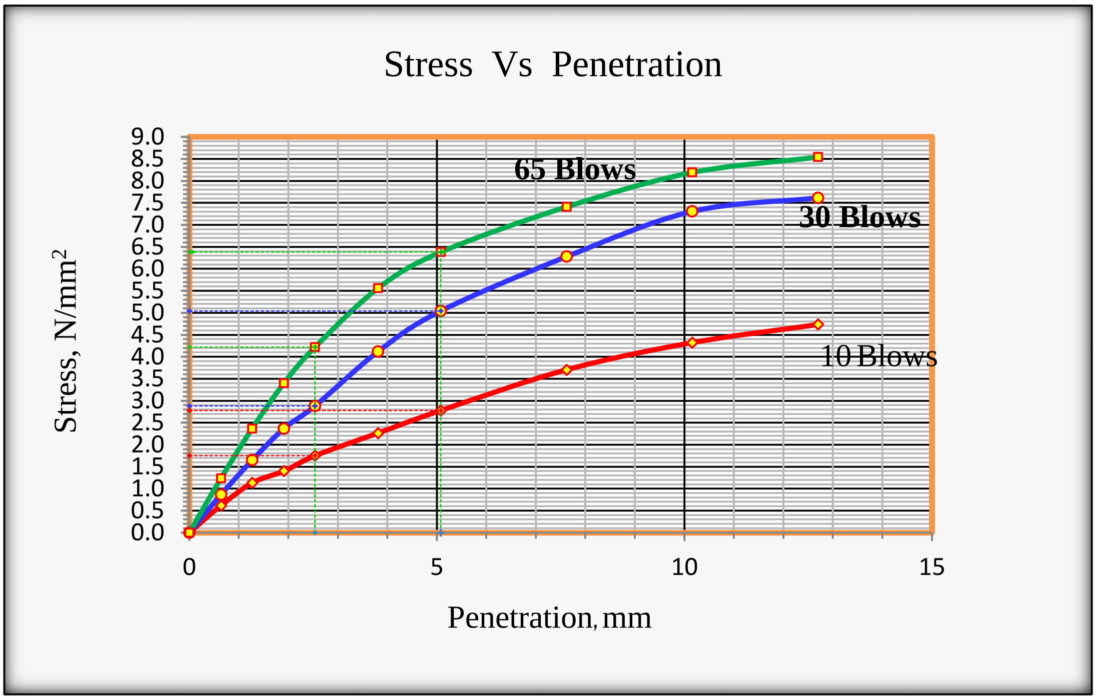
The sample of Site one (Limmu Genet):

Depth,m 10-15	Ring Calibration Factor, N/Div 398.24
Test pit (sample) No. 1	Plunger Area mm2 1935
Material Description subbase	Rate of strain, mm/min 1.27
Site Limmu-genet	Rammer wt. (kg) 4.54

CBR DATA

Penetration (mm)	Stand. Stress (N/mm^2)	10 Blows(Site 1)			30 Blows(Site 1)			65 Blows(Site 1)		
		Gauge read.	Stress N/mm^2	CBR %	Gauge read.	Stress N/mm^2	CBR %	Gauge read.	Stress N/mm^2	CBR %
0		0	0.00		0	0.00		0	0.00	
0.64		3	0.62		4.2	0.86		6	1.23	
1.27		5.5	1.13		8	1.65		11.5	2.37	
1.91		6.8	1.40		11.5	2.37		16.5	3.40	
2.54	6.9	8.5	1.75	25.35	14	2.88	41.76	20.5	4.22	61.15
3.81		11	2.26		20	4.12		27	5.56	
5.08	10.3	13.5	2.78	26.97	24.5	5.04	48.95	31	6.38	61.94
7.62		18	3.70		30.5	6.28		36	7.41	
10.16		21	4.32		35.5	7.31		39.8	8.19	
12.7		23	4.73		37	7.61		41.5	8.54	

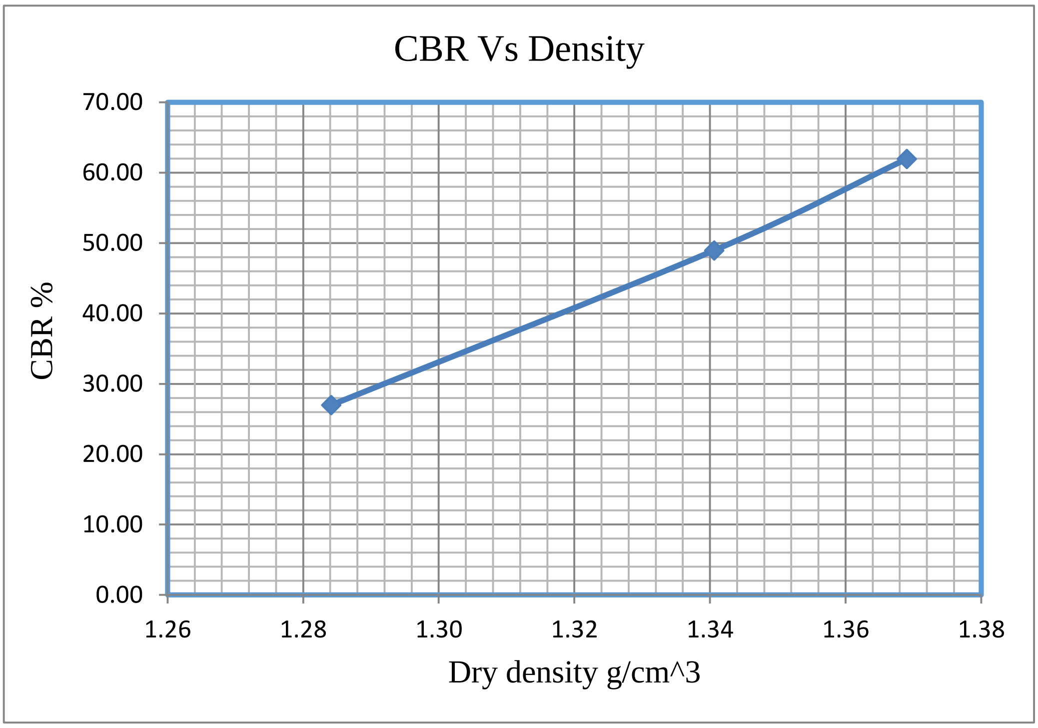
Day of month	Elapse time (Day)	Mould 1			Mould 2			Mould 3		
		Guage reading	Swell mm	Swell %	Guage reading	Swell mm	Swell %	Guage reading	Swell mm	Swell %
22/09/2017	0	0	0.23	0.20	0	0.07	0.06	0	0	0.00
26/09/2017	4	23			7			0		



MOISTURE CONTENT AND UNIT WEIGHT OF TEST SAMPLES						
Mould No.	6		7		17	
No. of layers	5		5		5	
No. of blows per layer	10		30		65	
CONDITION OF SAMPLE	Before soaking	After soaking	Before soaking	After soaking	Before soaking	After soaking
Wt. of wet sample+mould, g	11712	11876	11900	11999	12009	12066
Wt. of mold, g	8134	8134	8152	8152	8161	8161
Wt. of wet sample, g	3578	3742	3748	3847	3848	3905
Volume of mould, cm^3	2124	2124	2124	2124	2124	2124
Wet density, g/cm^3	1.68	1.76	1.76	1.81	1.81	1.84

MOISTURE CONTENT DETERMINATION						
Can No.	A2	C4	39	E4	D1	B5
Wt. of wet sample+can, g	373.5	302	392	837	377	710
Wt. of dry sample+can, g	330	229	341.5	680.5	350	583.5
Wt. of water, g	43.5	73	52.5	156.5	27	126.5
Wt. of can, g	190.5	5	173.5	189	266.5	194
Wt. of dry sample, g	139.5	224	166	491.5	83.5	389.5
% Moisture content	31.18	32.59	31.63	31.84	32.34	32.48
Dry density, g/cm^3	1.28	1.33	1.34	1.37	1.37	1.39

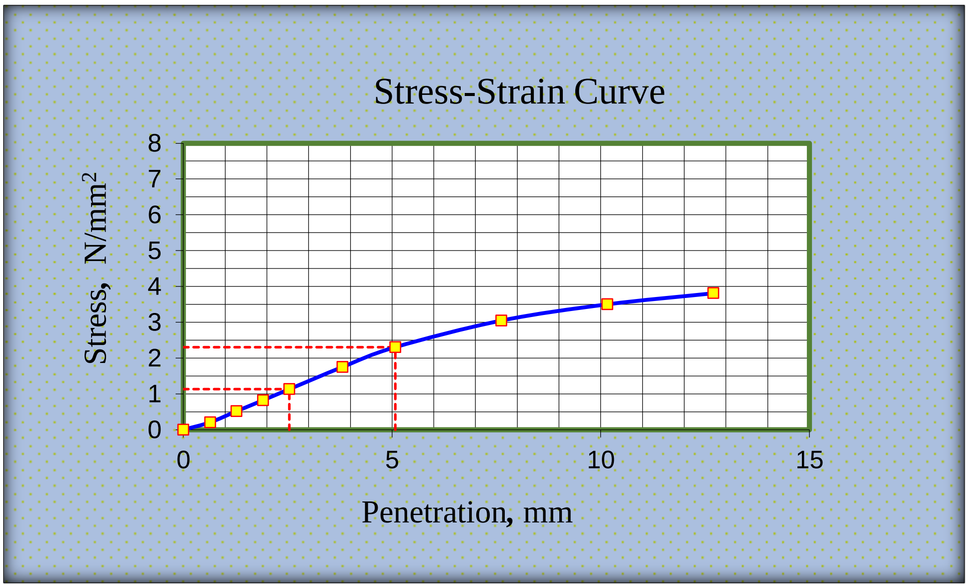
Blows/Layer	10/5	30/5	65/5	MDD, g/cm^3	1.39	OMC, %	30.4
Soaked CBR, %	26.97	48.95	61.94	Density required, %	95	Target density, g/cm^3	1.32
Density gm/cm^3	1.28	1.34	1.37	CBR, %	41.00		



The sample of Site two (Babu):

Depth,	8m-12m	Ring Calibration Factor, N/Div	398.5
Test Pit (sample) No.	2	Plunger Area, mm2	1935
Material Desc.	Subbase	Rate of strain, mm/min	1.27
Location	Babu	Rammer wt(kg)	4.54
Blow/ Layer	10/5		
Swell, %	0.50	Optimum Most. Content	17.25
CBR Value, %	**16.42**	Max. Dry Density	1.38

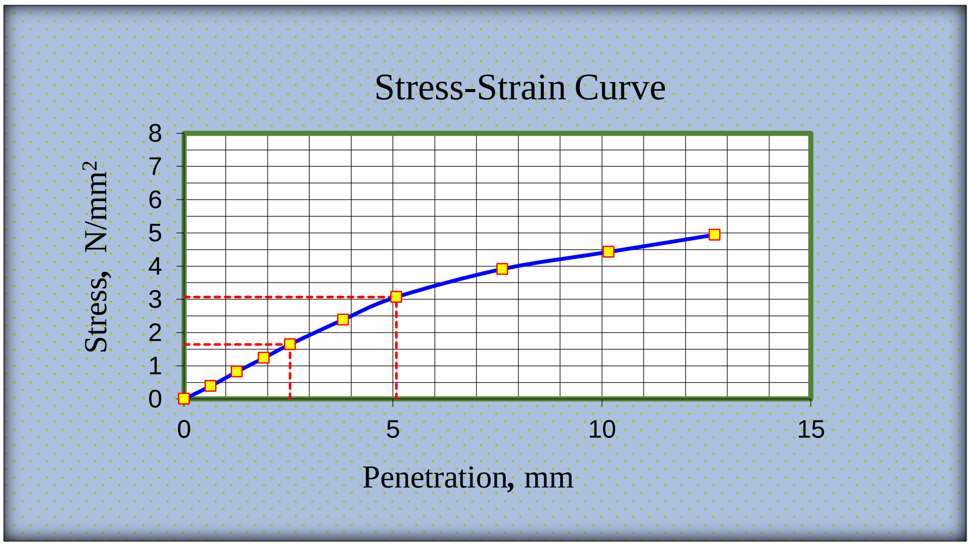
Penet. (mm)	Ring Reading (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0	0	0.00		
0.64	1	399	0.21		
1.27	2.5	996	0.51		
1.91	4	1594	0.82		
2.54	5.5	2192	1.13	6.9	16.42
3.81	8.5	3387	1.75		
5.08	11.2	4463	2.31	10.3	22.39
7.62	14.8	5898	3.05		
10.16	17	6775	3.50		
12.70	18.5	7372	3.81		



Depth, m	8m-12m	Ring Calibration Factor, N/Div	398.5
Test Pit (sample) No.	2	Plunger Area, mm2	1935
Material Desc.	Subbase	Rate of strain, mm/min	1.27
Location	Babu	Rammer wt.(kg)	4.54

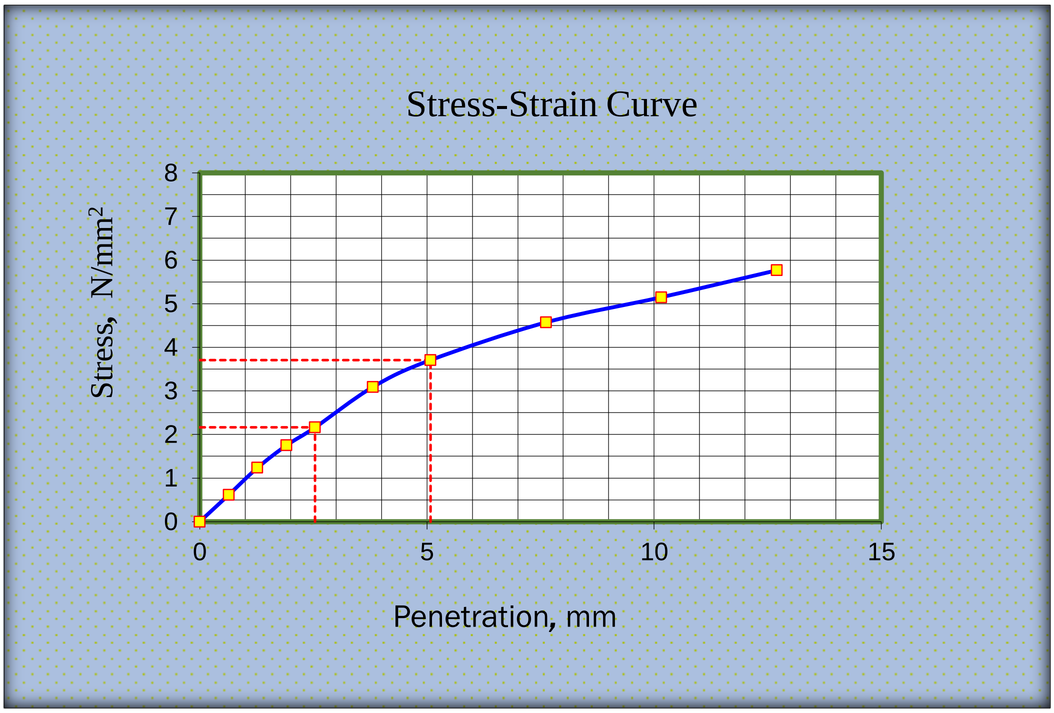
Blow/ Layer	30/5		
Swell, %	0.45	Optimum Most. Content	17.25
CBR Value, %	**23.88**	Max. Dry Density	1.38

Penet. (mm)	Ring Reading (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0	0	0.00		
0.64	1.9	757	0.39		
1.27	4	1594	0.82		
1.91	6	2391	1.24		
2.54	8	3188	1.65	6.9	23.88
3.81	11.6	4623	2.39		
5.08	14.9	5938	3.07	10.3	29.79
7.62	19	7572	3.91		
10.16	21.5	8568	4.43		
12.70	24	9564	4.94		



Depth, m	8m-12m	Ring Calibration Factor, N/Div		398.5
Test Pit (sample) No.	2	Plunger Area, mm2		1935
Material Desc.	Subbase	Rate of strain, mm/min		1.27
Location	Babu	Rammer wt. (kg)		4.54
Blow/ Layer	65/5			
Swell, %	0.43	Optimum Most. Content		17.25
CBR Value, %	**31.34**	Max. Dry Density		1.38

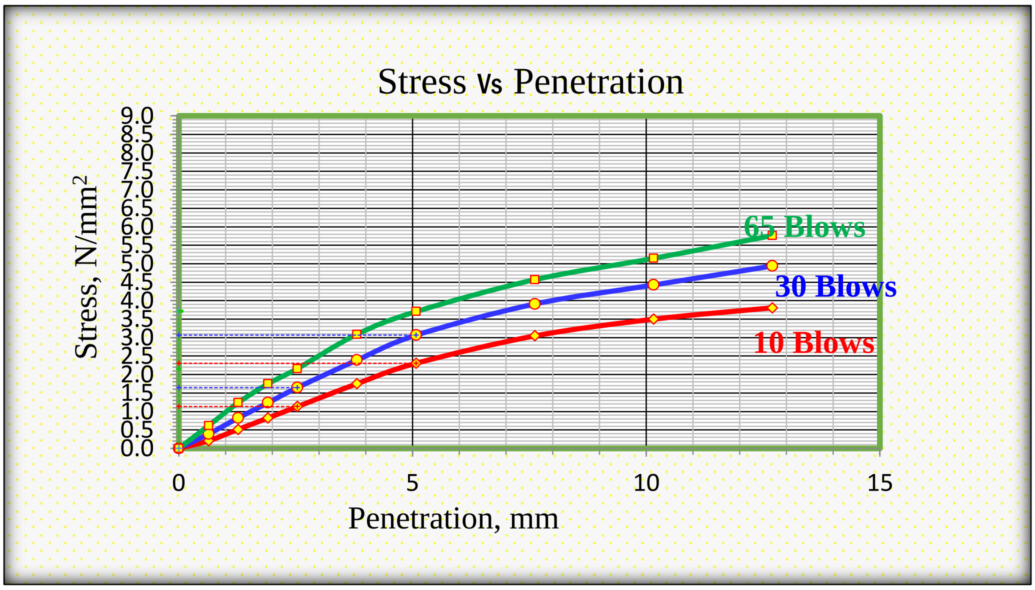
Penet. (mm)	Ring Reading (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0	0	0.00		
0.64	3	1196	0.62		
1.27	6	2391	1.24		
1.91	8.5	3387	1.75		
2.54	10.5	4184	2.16	6.9	31.34
3.81	15	5978	3.09		
5.08	18	7173	3.71	10.3	35.99
7.62	22.2	8847	4.57		
10.16	25	9963	5.15		
12.70	28	11158	5.77		



Depth, m	8-12	Ring Calibration Factor N/Div	398.24
Test pit (sample)	2	Plunge Area mm^2	1935
Material description	Subbase	Rate of Strain, mm/min	1.27
Location	Babu	Rammer.wt.(kg)	4.54

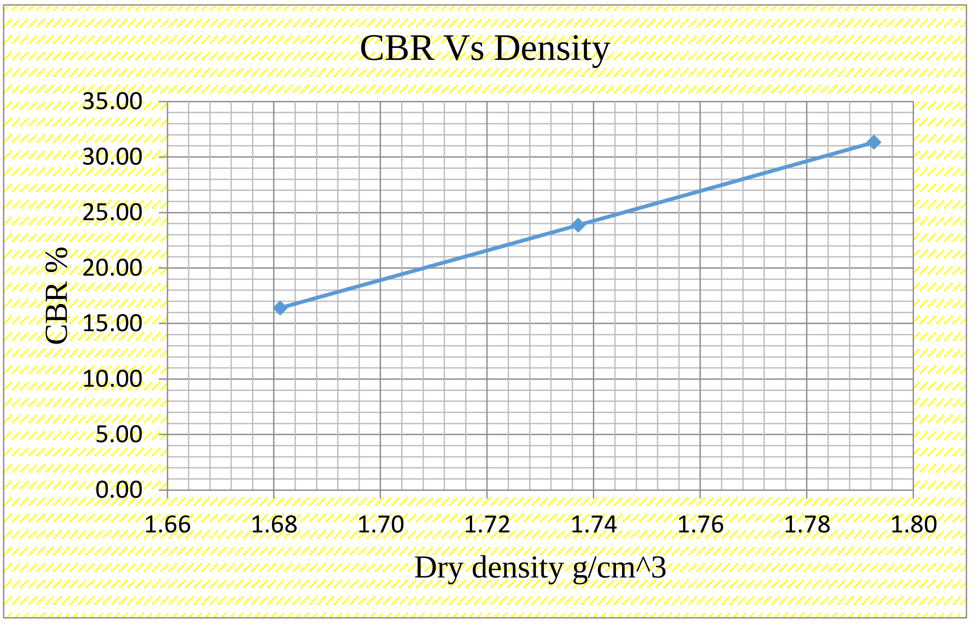
CBR TEST DATA

Penetration (mm)	Stand. Stress (N/mm²)	10 Blows(Site 2)			30 Blows(Site 2)			65 Blows(Site 2)		
		Gauge read.	Stress N/mm²	CBR %	Gauge read.	Stress N/mm²	CBR %	Gauge read.	Stress N/mm²	CBR %
0		0	0.00		0	0.00		0	0.00	
0.64		1	0.21		1.9	0.39		3	0.62	
1.27		2.5	0.51		4	0.82		6	1.23	
1.91		4	0.82		6	1.23		8.5	1.75	
2.54	6.9	5.5	1.13	16.41	8	1.65	23.86	10.5	2.16	31.32
3.81		8.5	1.75		11.6	2.39		15	3.09	
5.08	10.3	11.2	2.31	22.38	14.9	3.07	29.77	18	3.70	35.97
7.62		14.8	3.05		19	3.91		22.2	4.57	
10.16		17	3.50		21.5	4.42		25	5.15	
12.7		18.5	3.81		24	4.94		28	5.76	



MOISTURE CONTENT AND UNIT WEIGHT OF TEST SAMPLES						
Mould No.	6		7		17	
No. of layers	5		5		5	
No. of blows per layer	10		30		65	
Condition of Sample	Before soaking	After soaking	Before soaking	After soaking	Before soaking	After soaking
Wt. of wet sample+mould, g	12282	12383	12486	12509	12577	12597
Wt. of mould, g	8071	8071	8160	8160	8218	8218
Wt. of wet sample, g	4211	4312	4326	4349	4359	4379
Volume of mould, cm^3	2124	2124	2124	2124	2124	2124
Wet density, g/cm^3	1.98	2.03	2.04	2.05	2.05	2.06
MOISTURE CONTENT DETERMINATION						
Can No.	A2	C4	39	E4	D1	B5
Wt. of wet sample+can, g	346.5	312	469.5	893	309.5	635
Wt. of dry sample+can, g	324	286.3	430	785	294	576.5
Wt. of water, g	22.5	25.7	39.5	108	15.5	58.5
Wt. of can, g	198.5	165	201	189	187	194
Wt. of dry sample, g	125.5	121.3	229	596	107	382.5
% Moisture content	17.93	21.19	17.25	18.12	14.49	15.29
Dry density, g/cm^3	1.68	1.68	1.74	1.73	1.79	1.79

Blows/Layer	10/5	30/5	65/5	MDD, g/cm^3	1.828	OMC, %	30.4
Soaked CBR, %	16.41	23.86	31.32	Density required, %	95	Target density,g/cm^3	1.74
Density gm/cm^3	1.68	1.74	1.79	CBR, %	24.00		



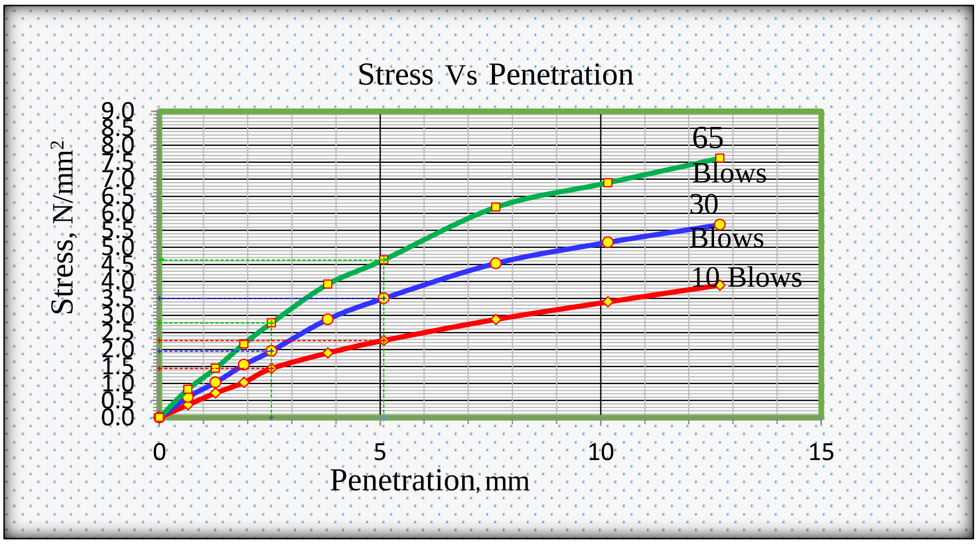
SWELL DATA									Initial height of sample, mm =		116.64
Day of month	Elapse time (Day)	Mould 1			Mould 2			Mould 3			
		Guage reading	Swell		Guage reading	Swell		Guage reading	Swell		
			mm	%		mm	%		mm	%	
22/09/2017	0	0	0.585	0.50	0	0.5252	0.45	0	0.5	0.43	
26/09/2017	4	58.5			52.52			50			

The sample of Site three (Ambuye):

Depth,m	10-12	Ring Calibration Factor N/Div	398.24
Test pit (sample)	3	Plung Area mm^2	1935
Material description	Subbase	Rate of Strain, mm/min	1.27
Location	Ambuye	Rammer. wt.(kg)	4.54

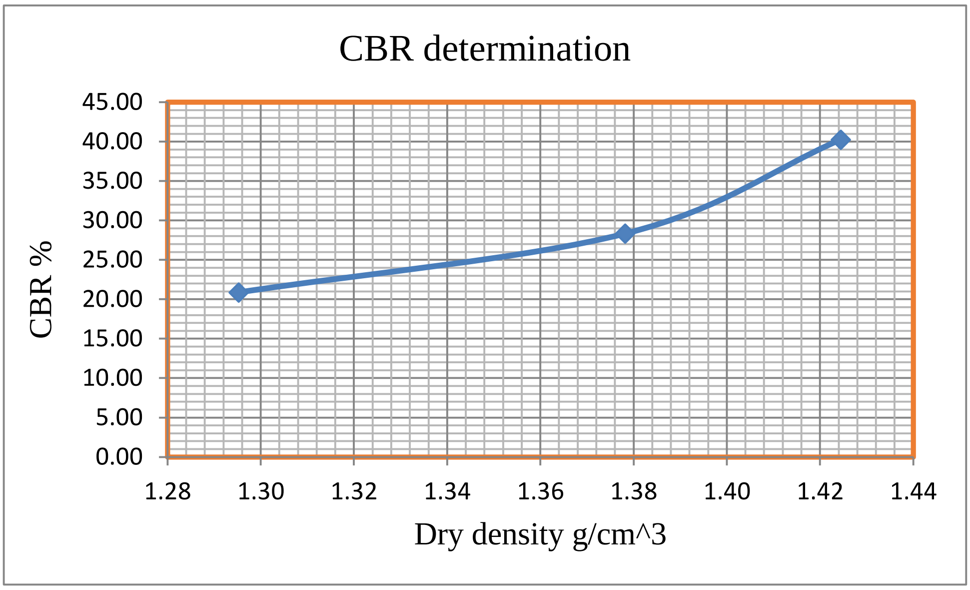
CBR DATA

Penetration (mm)	Stand. Stress (N/mm^2)	10 Blows(Site 3)			30 Blows(Site 3)			65 Blows(Site 3)		
		Gauge read.	Stress N/mm^2	CBR %	Gauge read.	Stress N/mm^2	CBR %	Gauge read.	Stress N/mm^2	CBR %
0		0	0.00		0	0.00		0	0.00	
0.64		1.8	0.37		2.9	0.60		4	0.82	
1.27		3.5	0.72		5	1.03		7	1.44	
1.91		5	1.03		7.5	1.54		10.5	2.16	
2.54	6.9	7	1.44	20.88	9.5	1.96	28.34	13.5	2.78	40.27
3.81		9.2	1.89		14	2.88		19	3.91	
5.08	10.3	11	2.26	21.98	17	3.50	33.97	22.5	4.63	44.96
7.62		14	2.88		22	4.53		30	6.17	
10.16		16.5	3.40		25	5.15		33.5	6.89	
12.7		18.9	3.89		27.5	5.66		37	7.61	



MOISTURE CONTENT AND UNIT WEIGHT OF TEST SAMPLES						
Mould No.	6		7		17	
No. of layers	5		5		5	
No. of blows per layer	10		30		65	
Condition of the sample	Before soaking	After soaking	Before soaking	After soaking	Before soaking	After soaking
Wt. of wet sample+mould, g	11597	11898	12178	12287	12019	12097
Wt. of mould, g	8095	8095	8435	8435	8100	8100
Wt. of wet sample, g	3502	3803	3743	3902	3919	3997
Volume of mould, cm^3	2124	2124	2124	2124	2124	2124
Wet density, g/cm^3	1.65	1.79	1.76	1.84	1.85	1.88
MOISTURE CONTENT DETERMINATION						
Can No.	A2	C4	39	E4	D1	B5
Wt. of wet sample+can, g	566	302	550	837	328	709
Wt. of dry sample+can, g	502	229	341.5	680.5	299.5	583
Wt. of water, g	64	73	62	156.5	28.5	126
Wt. of can, g	267.5	5	265.5	189	203	194
Wt. of dry sample, g	234.5	224	222.5	491.5	96.5	389
% Moisture content	27.29	32.59	27.87	31.84	29.53	32.39
Dry density, g/cm^3	1.30	1.35	1.41	1.39	1.42	1.42

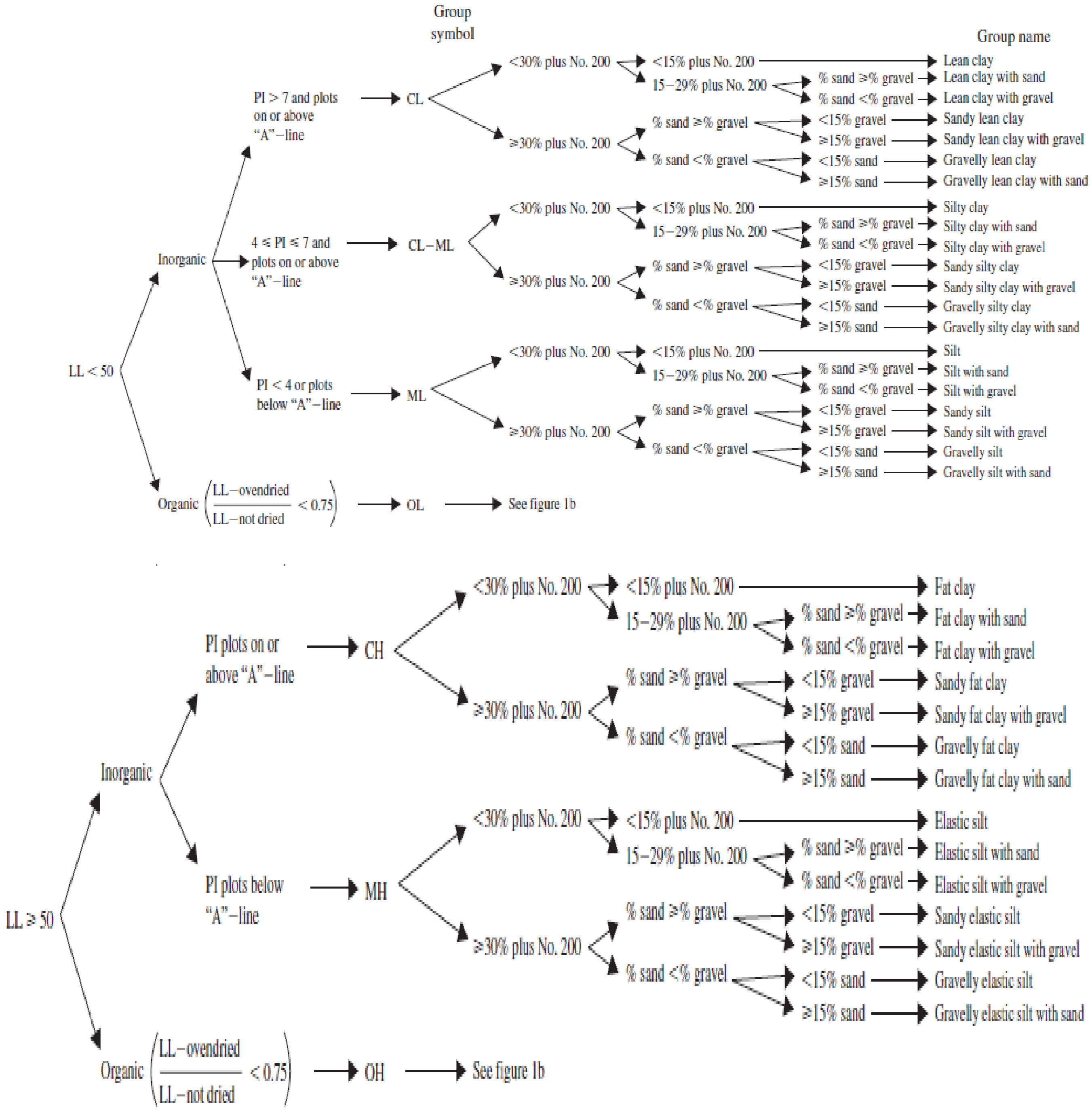
Blows/Layer	10/5	30/5	65/5	MDD, g/cm^3	1.48	OMC, %	27
Soaked CBR, %	22.37	40.27	47.72	Density required, %	95	Target density, g/cm^3	1.41
Density gm/cm^3	1.30	1.38	1.42	CBR, %	34.00		



Day of month	Elapse time (Day)	Mould 1			Mould 2			Mould 3		
		Guage reading	Swell		Guage reading	Swell		Guage reading	Swell	
			mm	%		mm	%		mm	%
22/09/2017	0	5	0.37	0.32	1	0.31	0.27	0	0.3	0.26
26/09/2017	4	37			30.99			30		

Appendix - F

Charts of soil classification for USCS method and ERA manual of DCP test



Flow Chart for Classifying Organic Fine-Grained Soil (50 % or More Passes No. 200 Sieve).

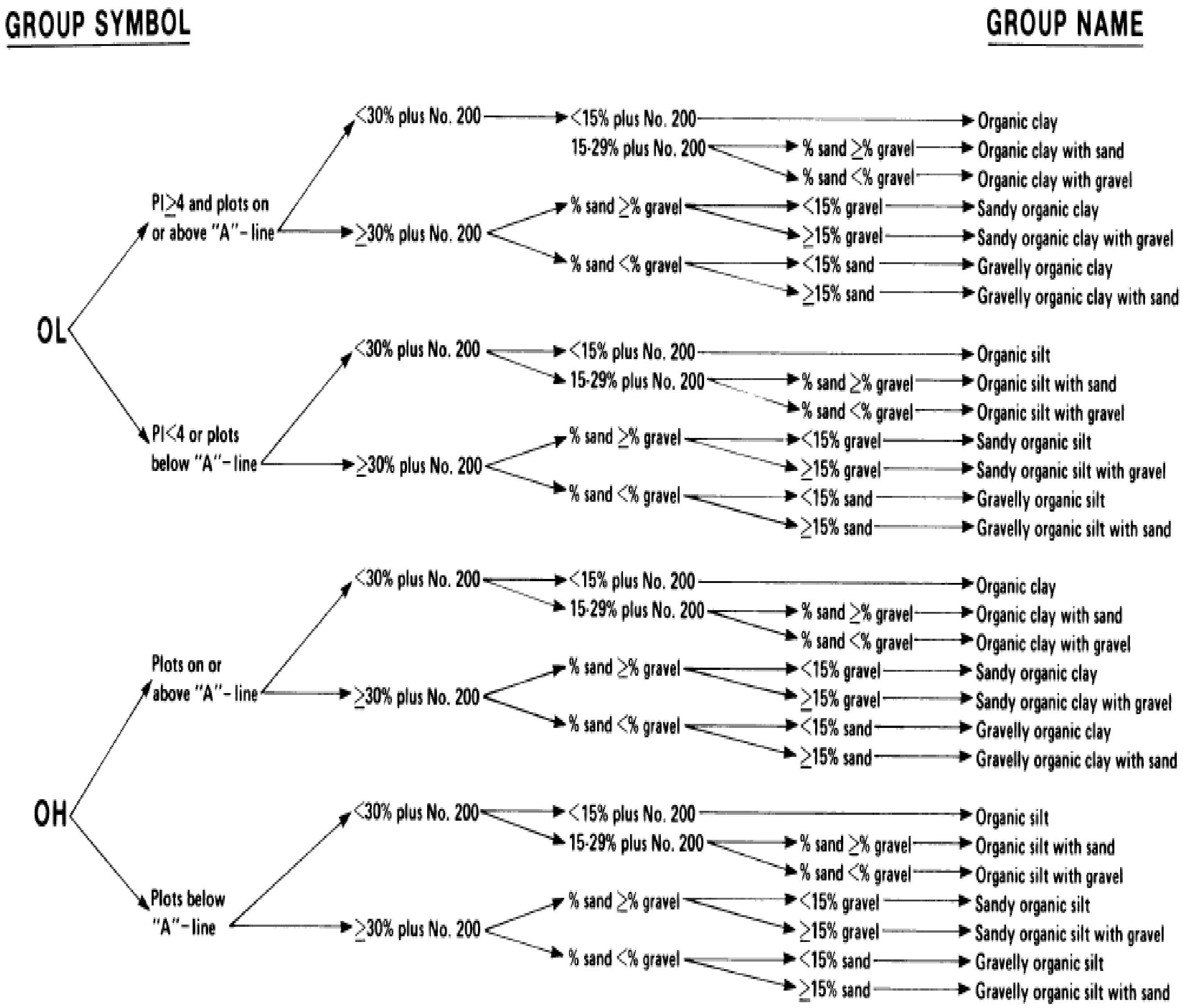
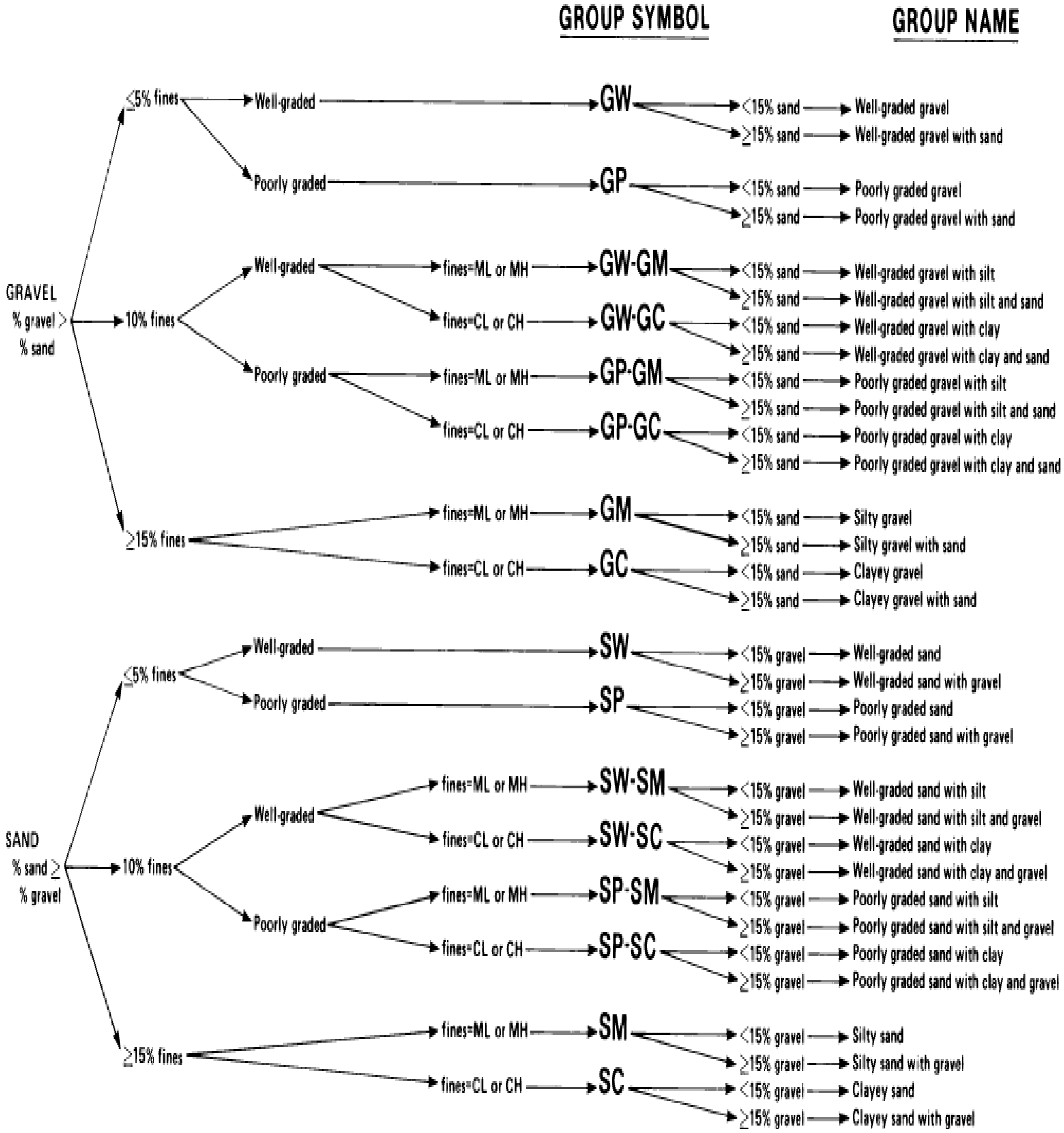


FIG.1 Flow Chart for Classifying Fine Grained Soil using the USCS

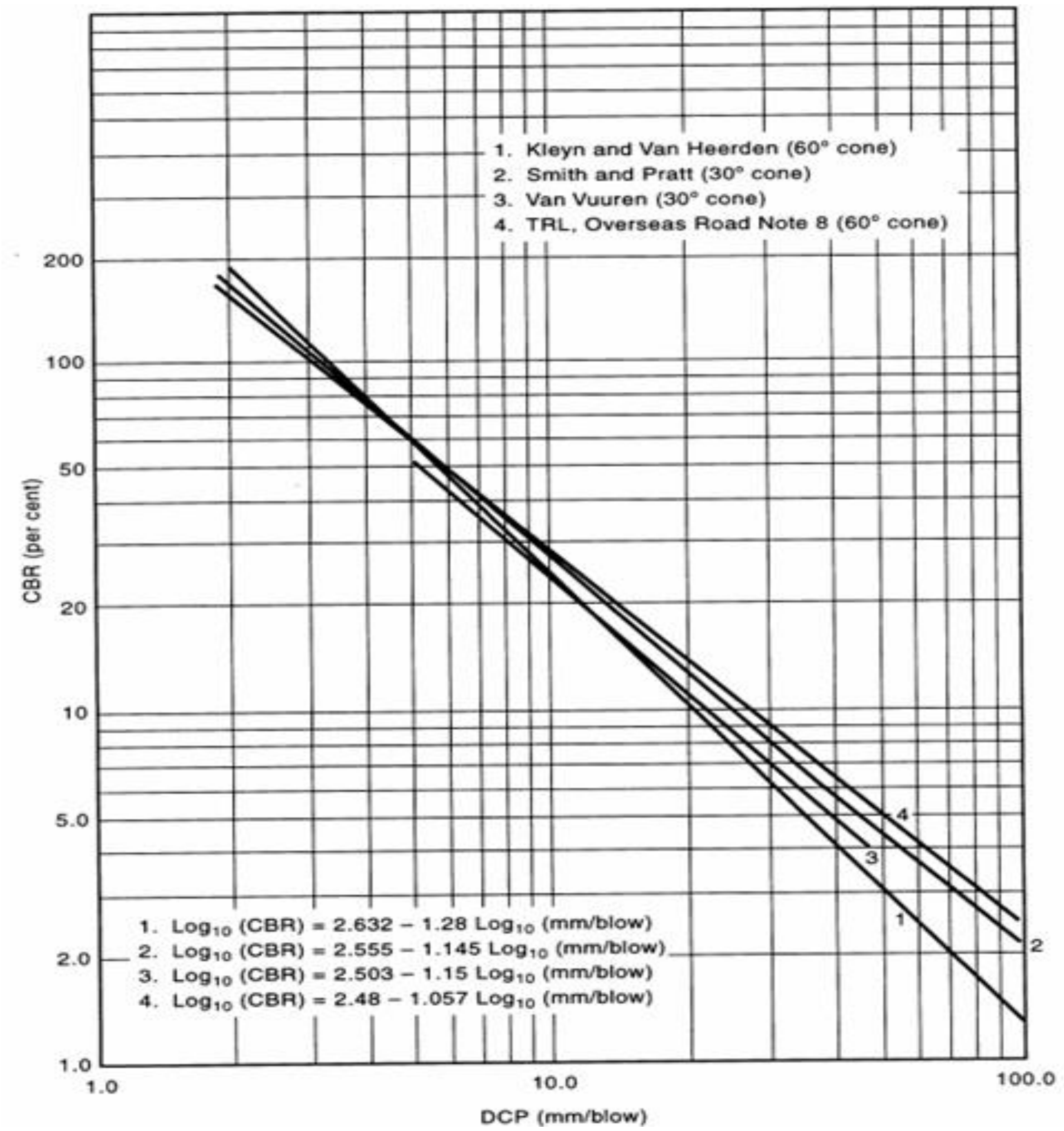
(50 % or More Passes No. 200 Sieve).



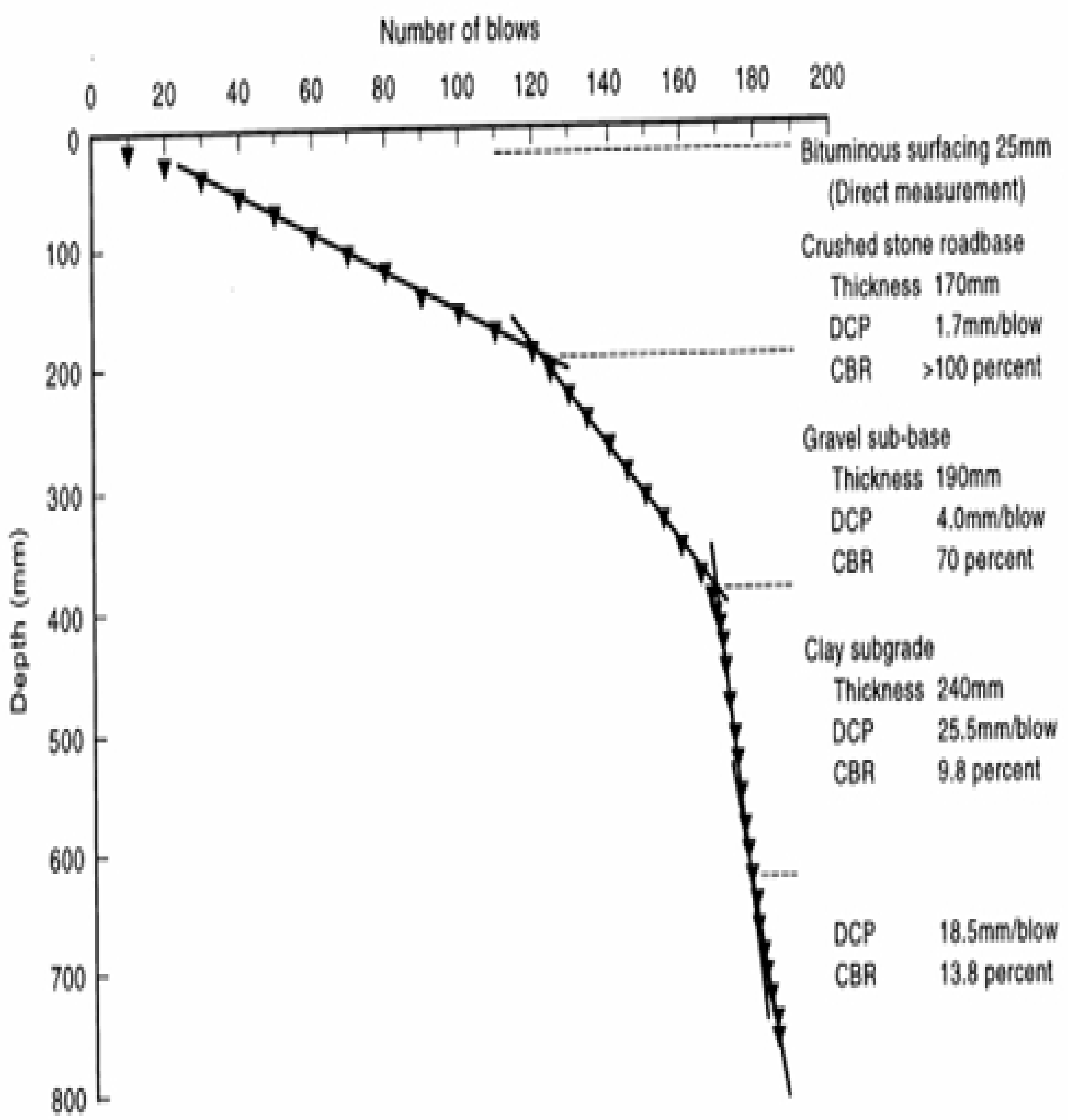
Flow Chart for Classifying Coarse-Grained Soils

(More Than 50 % Retained on No. 200 Sieve).

DCP-CBR Relationships



(Source: ERA manual Appendix C, 2002.)



DCP Test Result

(Source: ERA manual Appendix C, 2002.)

Appendix – G

Laboratory test picture and Calibration factor for CBR test

Atterberg limit test

Specific gravity test


Compaction test

Sample preparation

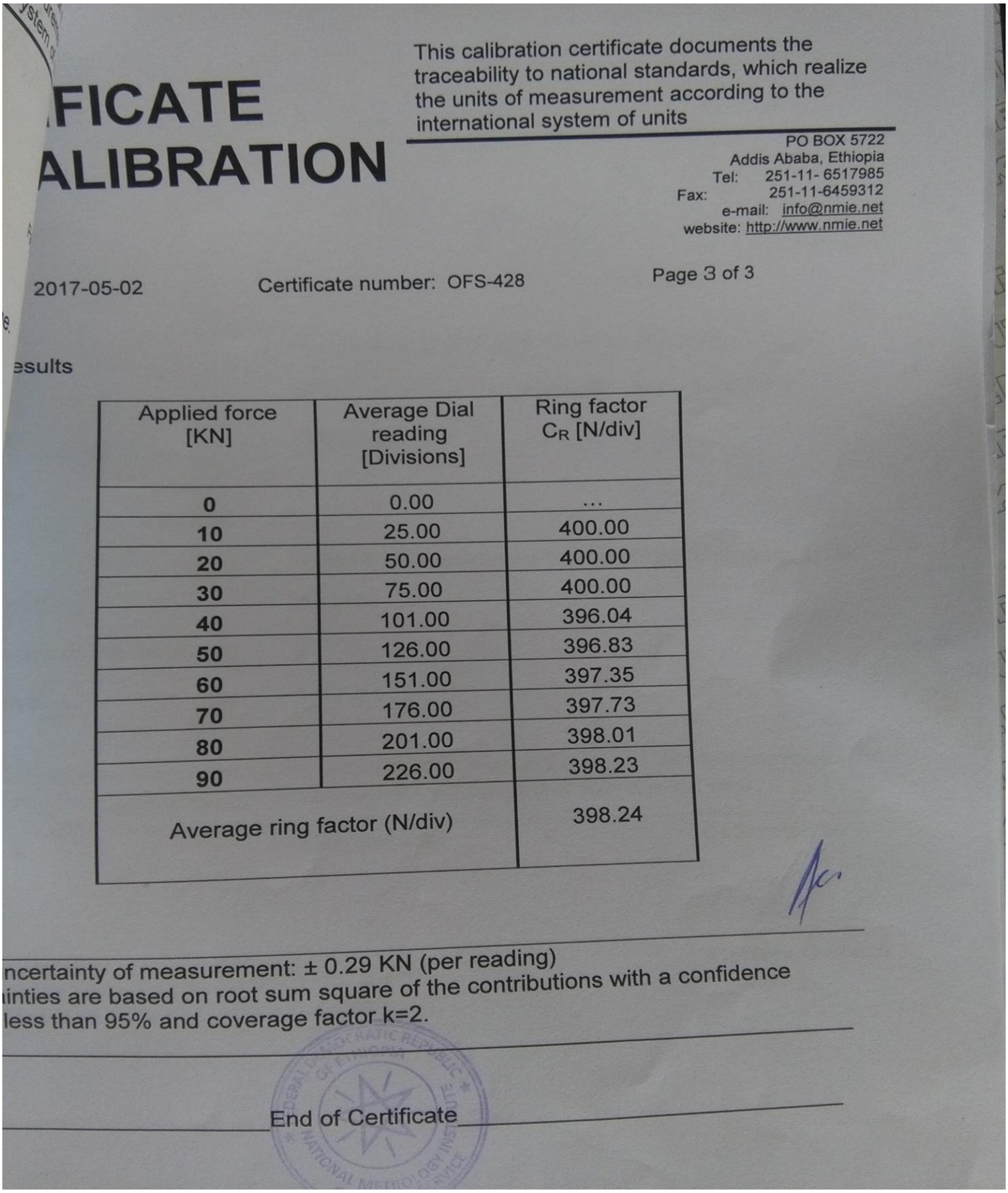
Compaction

Soaking for four days

CBR machine

CBR test

Calibration factor certificate for CBR test



FICATE
ALIBRATION

This calibration certificate documents the traceability to national standards, which realize the units of measurement according to the international system of units

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esults

Applied force [KN]	Average Dial reading [Divisions]	Ring factor C_R [N/div]
0	0.00	...
10	25.00	400.00
20	50.00	400.00
30	75.00	400.00
40	101.00	396.04
50	126.00	396.83
60	151.00	397.35
70	176.00	397.73
80	201.00	398.01
90	226.00	398.23
Average ring factor (N/div)		398.24

ncertainty of measurement: ± 0.29 KN (per reading)
inties are based on root sum square of the contributions with a confidence less than 95% and coverage factor k=2.

End of Certificate