Jimma University

Jimma Institute of Technology

School of Graduate Studies

Faculty of Civil and Environmental Engineering

Geotechnical Engineering Chair

Improvement of Weak Subgrade Materials Performance by Blending with Selected Materials from Quarry Sites: A Case Study in Agaro Area.

A Thesis Submitted to the School of Graduate Studies of Jimma University In Partial Fulfillment of the Requirements for the Master's Degree of Civil Engineering in Geotechnical Engineering

By:

Ahmed Simeneh Tamene

January 2020

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Co-Advisor Signature Date

DECLARATION

I, the undersigned, declare that this thesis entitled: "**Improvement of Weak Subgrade Materials Performance by Blending with Selected Materials from Quarry Sites: A Case Study in Agaro Area.**" 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 this thesis have to be duly acknowledged.

Candidate:

Name: Ahmed Simeneh

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January 2020

As Master's Research Advisors, I hereby certify that I have read and evaluated this MSc Thesis prepared under my guidance by **Ahmed Simeneh** entitled: "**Improvement of Weak Subgrade Materials Performance by Blending with Selected Materials from Quarry Sites: A Case Study in Agaro Area.**"

Prof. Emer T. Quezon, P.Eng	______________	12/01/2020
Main Advisor	Signature	Date
Engr. Habte Tamirat	______________	____/_____/____
Co-Advisor	Signature	Date

ABSTRACT

Road pavements commonly consist of several layers of various materials and thickness. The performance of a pavement depends on the quality of its subgrade and subbase layers. There are many soil stabilization methods to gain the required engineering properties. These methods range from mechanical to chemical stabilization. Chemical treatments are relatively expensive to be implemented in most developing countries and the best way is to use locally available materials with relatively cheap costs and affordable. The native soil behavior of roads and their mode of failure were studied to establish the proper method of improving native soils using locally available materials. A wide variety of soil types occur across Agaro town, in-depth understanding of these subgrade soils where any pavement project is to be constructed is essential to sustain its design life. A laboratory experiment carried out in this study which aimed to highlight the physical mechanisms of improvement of weak subgrade materials performance by blending with selected materials. On the collected weak subgrade soils and selected materials from Agaro town quarry sites, a laboratory tests such as Atterberg's limit test, Specific Gravity, Grain Size Analysis, Compaction test, CBR and CBR Swell test were done. The results of consistency test shows that the subgrade soils have a PI values more than 30% and according to AASHTO soil classification system the soils were Clayey Soils laying under A-7-5 soil type with a GI greater than 20. This is an indication of the soil section with high PI and GI values and very poor to support the traffic load. The Selected materials classified under A-2-6(1) are found to be [Silty or Clayey Gravel and Sand] material having a PI value of 17.05%. The maximum dry density for subgrade soils ranges from 1.57g/cc to 1.65g/cc, and 2.17g/cc for selected materials. The subgrade soil were not give CBR more than 8%, then to reduce the thickness of pavement, improvement of the subgrade material is done, when blended with the selected material ranging from 50% to 85% to attain a CBR value ranging between 10% and 20% when compacted to 95% MDD modified compaction. The provision of improved layer avoids the necessity of an extraordinary thick sub-base, and provides an adequate working platform for sub-base compaction as well as reduces the risk of damage to the subgrade during construction. This shows 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.

Keywords: *Weak Subgrade Soils, Selected Materials, Engineering Characteristics, Blending.*

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List of Abbreviations and Acronyms

AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
CBR	California Bearing Ratio (as described in AASHTO T 193)
ERA	Ethiopian Roads Authority
FHWA	Federal Highway Administration
JiT	Jimma University Institute of Technology
LL	Liquid limit
MDD	Maximum dry density
OMC	Optimum moisture content
PI	Plasticity index
PL	Plastic Limit
QM	Quarry Materials (Selected Granular Materials)

CHAPTER ONE

1.0 INTRODUCTION

1.1 Background of the Study

Road pavements commonly consist of several layers of various materials and thickness. Each layer assists systematically in supporting traffic load and distributing it safely to the foundation soil, which is known as a subgrade. The subgrade may be either native soil or imported material. When the native soil is deemed to be unsuitable as a subgrade, it is normally treated appropriately and used to avoid the high cost that may be incurred for imported material [20].

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. 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[33].

To determine the type of subgrade material that can be used as well as the the appropriate type of treatment, a series of soil investigation has to be undertaken. Stability of the subgrade is normally expressed in terms of bearing capacity, which is related to certain geotechnical properties of the soil.

Subgrade plays an important role in safe and cost-effective pavement construction, given that the materials are suitable. Usually, there is a requirement for the improvement of both the plasticity and the bearing capacity of local soils. The stabilization methods are a common suggestion for such goals to be achieved. There are many techniques for soil stabilization and the choice between them depends on several economic, practical and environmental parameters. Discrete techniques are chemical stabilization, thermal stabilization, stabilization by additives such as lime and cement [16].

Proper treatment of problem soil conditions and the preparation of the foundation are extremely important to ensure a long-lasting pavement structure that does not require excessive maintenance. Mixing/ blending granular materials with subgrade soils and compaction can provide a stable working platform and foundation layer under pavements[18].

Sand is a naturally occurring granular material. Because of its high load-bearing capacity in confined conditions, sand could be used as a filler material. So, sand could be used in varying proportions as an admixture to cohesive soils altering the properties of plasticity, compaction and strength of the mixtures [26].

This research was conducted on Agaro town, one of the reform urban centers of the Oromia region is the second-largest urban center in Jima zone next to Jima city. The town is the administrative seat of the Gomma district and situated to the southwest of Addis Abeba at about 390kms and west of Jimma city at about 44kms. It is also located to the east of Bedele town (the closest large town) at about 93kms. Absolutely, Agaro town is found at 07^0 50’ 20” to 07^0 52’40” N latitude and 36^0 33’ 40” to 36^0 37’00” E longitudes.

The topography of Agaro is dominated by broken land features. Small hills followed by valleys, river valleys and sometimes flat terrain prone to flooding are common land features throughout the town. This type of land feature induces problems of flooding and erosion. Undulating topographic features pose constraints for activities that require a considerable land area with a gentle slope. Most of the existing activities are undertaken on a few pocket areas found on top of hills with flat top and hillsides with a possible slope for respective purposes or activities. The elevation of hilltops gradually decreases from south to north. Valley bottoms in the southern parts of the town are narrow, whereas they are wide and flat in the northern part. The elevation of the town ranges between 1,546 meters above sea level (m.a.s.l.) and 1,796 m.a.s.l. with an altitudinal range of 1m. Consequently, the average elevation of the town is 1671 m.a.s.l., which would make its climate sub-tropic. The mean temperature sensation for Agaro is found to be neutral (i.e., 20.6^0c). Agaro and its environments have eight continuous rainy months from March to October with a mean annual rainfall of 1,663.1mm. The wind direction of the area is east to west with a max speed of about 8km/hr.

Agaro town is situated on Addis Ababa/Jima-Metu asphalt road that serves the town as a principal arterial street and a bypass (expressway) for those vehicles that are crossing the town. The gravel road of Agaro–Gera is the major sub-arterial street segment of the town. The remaining streets are un-surfaced roads connecting the town with rural villages such as Bulado, Koye, Dalecho, Bulbulo and the like. Such road segments are categorized under major and minor collector streets. The existing inner-city road network of the town is characterized by highly damaged asphalt roads, non-standard gravel, and earthen roads.

A wide variety of soil types occur across Agaro town, in-depth understanding of these subgrade soils in any pavement project area is essential to appropriately engineer the construction, rehabilitation, or widening of a highway facility. Depending on the existing soils and project design, the properties of the subgrade may need to be improved, either mechanically, chemically, or both, to provide a platform for the construction of subsequent layers and to provide adequate support for the pavement over its design life.

This study covers the improvement of weak subgrade materials performance by blending with selected materials to achieve a new subgrade class or selected design subgrade/foundation class. The natural soil at an existing location have weak in nature. Suitable soil selected and this is to be blended with the available soils to improve the soil properties at a lesser cost and materials to achieve the best results.

1.2 Statement of the Problem

Construction of roads on fine-grained soils without any form of stabilization is a major problem all over the world. If the subgrade is poorly prepared (improper compaction, excessive moisture, etc.) or has a very low strength (such as with highly plastic clays), the subgrade cannot resist these high stresses and ruts will form, which could lead to significant damage to the pavement [15].

Subgrade plays an important role in safe and cost-effective pavement construction, given that the materials are suitable. Usually, there is a requirement for the improvement of both the plasticity and the bearing capacity of local soils. The stabilization methods are a common suggestion for such goals to be achieved. There are many techniques for soil stabilization and the choice between them depends on several economic, practical and environmental parameters [16].

A wide variety of soil types throughout the town, Stabilization needs therefore also vary considerably throughout the town, and local knowledge of the soil types is important in selecting an appropriate subgrade stabilization approach. According to stabilization techniques followed in order to improve the soil, the finer soil particles are replaced with coarser particles of selected granular admixture. In such a way, a uniform gradation of particles in the soil is created and the composite mix formed possesses both cohesion and friction. Furthermore, when properly mixed, placed and compacted at the site, the soil exhibits improved load-carrying capacity.

Impact of unsuitable and non-uniform soils on pavement performance, particularly stiffness and stress contributions. For weak subgrade, strengthening measures are required in order to provide a strong and uniform support for the pavement and to allow road construction vehicles to pass over the subgrade without damaging the layer. This can be achieved by providing a thick layer of sub-base on the subgrade but it may be more economical to provide blend layer of selected materials. Where the local soil will not give CBR more than 8%, then to reduce the thickness of pavement, improved sub-grade material is suggested.

1.3 Research Questions

The research questions needed to be answered here:-

1. What are the engineering properties of the subgrade soils and selected materials?
2. How can the selected materials affect the properties of weak subgrade soils?
3. Suggest the optimum blending amount and impacts on pavement performance meeting the requirements of the specifications.

1.4 Objectives of the Study

1.4.1 General Objective

The main objective of this research effort was to improve weak subgrade materials performance by blending with selected materials from Agaro town quarry sites.

1.4.2 Specific Objectives

1. To determine the engineering properties of the subgrade soil and selected materials.

2. To investigate the effect of selected materials on the properties of weak subgrade soils.
3. To suggest the optimum blending amount and impacts on pavement performance meet the requirements of the specifications.

1.5 Significance of the Study

In Ethiopia, there are numerous road projects being constructed and to be constructed in the future. So, subgrade materials have a significant role in road construction. For this reason, the researcher had arrived with the following signs of the study:-

- ✓ In the future, other researchers use the findings as a reference for further research on the improvement of weak subgrade materials performance through blending with selected materials.
- ✓ The mixing of existing soils eliminates pockets of high moisture contents.
- ✓ Blending is used to improve the native subgrade to achieve a new subgrade class or selected design subgrade class and reduce the thickness of pavement.
- ✓ The provision of improved layer avoids the necessity of an extraordinary thick sub-base, and provides an adequate working platform for sub-base compaction as well as reduces the risk of damage to the subgrade during construction.
- ✓ The construction time required for excavating problematic soils and/or hauling in additional materials will be reduced. It is the most economical pavement strategy.

1.6 Scope of the Study

Based on the existing theories and principles this research study should be addressed the general objective to improve weak subgrade materials performance by blending with selected materials: A case study on Agaro town comprised the following main tasks:-

- ✓ The finding of this study was limited for a representative sample of weak subgrade soil, the required laboratory test conducted. The study also investigates the effect of selected material on the engineering properties of weak subgrade soil.

- ✓ Optimal blends can be determined, Atterberg Limit and CBR tests are used to check if the properties of the blended material meet the study requirements and specifications.

- ✓ To develop the conclusion and recommendation based on laboratory results after conducting different laboratory tests such as grain size analysis, specific gravity, Atterberg limit, maximum dry density, optimum moisture content, CBR and CBR swell for each respective blends. The results were analyzed according to AASHTO, and ERA specifications.

1.7. Structure of the Thesis

This research study comprised of five chapter and their contents is outlined below.

1) In the first chapter an overview of the background of the research, statement of the problem, research questions, and objectives of the research, significant of the study, and Scope of the thesis work were discussed.
2) The second chapter deals with the literature review about pavement, subgrade materials, and type of laboratory test conducted for subgrade materials (sieve analysis, compaction, Atterberg's limit, and CBR tests) and types of improvment option.
3) The third chapter deals with the materials and methodology.
4) The fourth chapter results and discussion deals with assessments of test results and that are gathered from field and laboratory tests compared with a standard specification of AASHTO, and ERA.
5) Finally fifth chapter a conclusion and recommended remedial measures are derived based on the results of chapter four.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 Introduction

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

2.1.1 Definition

Highway subgrade or basement soil may be defined as the supporting structure on which pavement and its courses rest.

2.1.2 Types of Highway Subgrade

1) In cut sections, the subgrade (defined as cut or excavation) is the original soil lying below the special layers designated as base and sub-base materials.

2) Infill sections, the subgrade (defined as embankment or embankment fill) is constructed over the native ground and consists of imported material from nearby roadway cuts or from the borrow pit.

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. 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. Uniformity is important, especially for pavements, but the high level of subgrade support will allow the pavement to reach the design life. In most instances, once heavy earthwork and fine grading are completed, the uppermost zone of subgrade soil (roadbed) is improved. The typical improvement technique is achieved by means of mechanical stabilization (i.e., compaction)[28].

2.2 Design guide for Flexible Pavement AASHTO (1993)

Design guide for flexible pavement as per AASHTO (American Associations of State Highways and Transportation Officials**)** (1993) suggests the determination of Homogenous sections using the CBR at 95% of the MDD (Maximum dry density) and analysis of Unit delineation by cumulative differences. In this method, Group index value and quality of subgrade materials are correlated.

In AASHTO, (2000) standard, the following points are discussed in detail which refers to highway material characterizations and materials intended to be used as a sub-grade layer.

- ✓ Sample spacing for geotechnical site investigations be in the range from 150m to 450m interval during the construction phase
- ✓ CBR values and swell potential of cohesive soils.
- ✓ Density /Moisture content clay soils.

The AASHTO (2004) soils classification includes seven basic groups (A-1 to A-7) and twelve subgroups. Of particular interest is the Group Index, which is used as a general guide to the load-bearing ability of soil. The group index is a function of the liquid limit, the plasticity index and the amount of material passing the 0.075mm sieve. Under average conditions of good drainage and thorough compaction, the supporting value of a material may be assumed as an inverse ratio to its group index, i.e. a group index of ‘0’ indicates a “good” sub-grade material and a group index of ‘20’ or more indicates a poor sub-grade material.

Using AASHTO classification and test methods M145, the Group index is calculated by equation 2.1.

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

Where:

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

LL = liquid Limit,   PI = Plasticity index of the soil

2.3 Ethiopian Roads Authority Pavement Design manual (ERA, 2013)

The type of subgrade soil is largely determined by the location of the road. However, where the soils within the possible corridor for the road vary significantly in strength from place to place, it is desirable to locate the pavement on the stronger soils if this does not conflict with other constraints.

The strength of the road subgrade for flexible pavements is commonly assessed in terms of the California Bearing Ratio (CBR) and this is dependent on the type of soil, its density, and its moisture content. Direct assessment of the likely strength or CBR of the subgrade soil under the completed road pavement is often difficult to make. Its value, however, can be inferred from an estimate of the density and equilibrium (or ultimate) moisture content of the subgrade together with knowledge of the relationship between strength, density and moisture content for the soil in question. This relationship must be determined in the laboratory. The density of the subgrade soil can be controlled within limits by compaction at a suitable moisture content at the time of construction.

The eventual moisture content of the subgrade soil is governed by the local climate, the depth of the water table below the road surface, and the provisions that are made for both internal and external drainage. It is useful to recall some basic relationships relating soil strength, density and moisture content and how they affect the final subgrade strength (Section 2.3.1). In Section 2.3.2, the various steps leading to the selection of a design CBR are described.

2.3.1 Density-Moisture Content-Strength Relationships of the Subgrade

During road construction the (dry) density of the subgrade soil (and its moisture content) is modified from its original state by compaction at subgrade level (in cuts) and by compaction of the excavated materials used in embankments. The moisture content is adjusted in order to make it easier to achieve a high level of compaction.

Upon completion of the construction operations, the density of the compacted subgrade soil will remain approximately the same except for some residual compaction under traffic and possible volume variations of certain moisture sensitive soils. However the moisture content of the subgrade will change, depending on climate, soil properties, depth of water table, rainfall and drainage. It is knowledge of this condition of the subgrade that is required in the design process.

To illustrate the above discussion, Figures 2.1 and 2.2 illustrate examples of relationships between density, moisture content and CBR. Two figures are shown to emphasize that the relationships are specific to the nature of the subgrade soil. The figures indicate a 'likely level of compaction achieved during construction'. It can be seen from the figures that, as the moisture content increases at constant density (moving to the right) the CBR decreases quite quickly. If the soil becomes saturated, i.e. the air voids become filled with water and decrease to zero, the soil becomes very weak indeed.

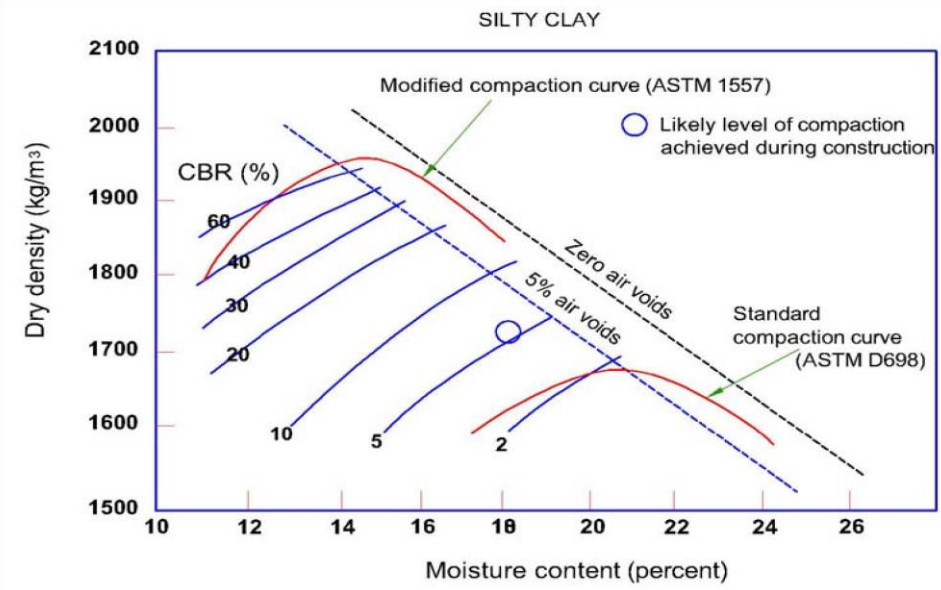

Figure 2.1 Dry Density, Moisture Content, Soil Strength Relationship for a Silty Clay(ERA, 2013)

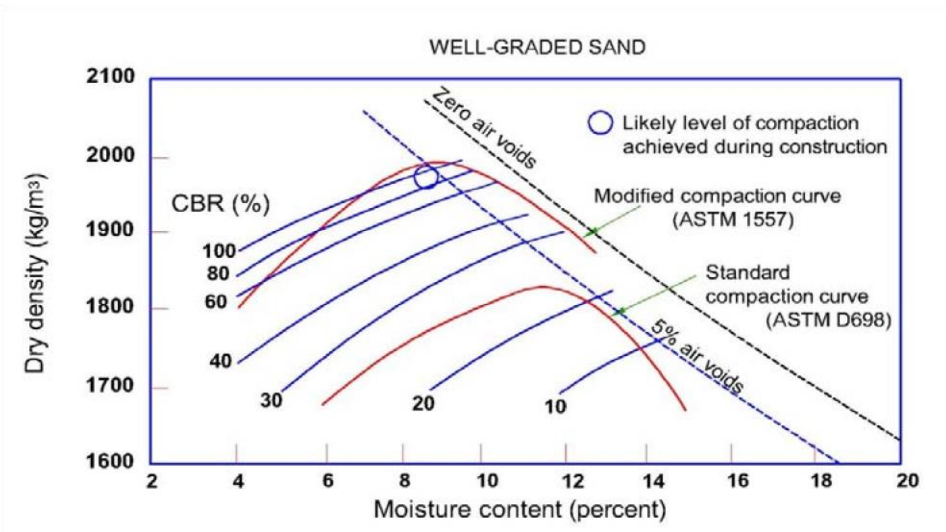

Figure 2.2 Dry Density, Moisture Content, Soil Strength Relationship for a Well Graded Sand (ERA, 2013)

2.3.2 Design Subgrade Strength

To determine the subgrade strength for the design of the road pavement, it is necessary to first determine the density-moisture content-strength relationship(s) specific to the subgrade soil(s) encountered along the road under study. It is then necessary to select the density which will be representative of the subgrade once compacted and to estimate the subgrade moisture content that will ultimately govern the design, i.e. the moisture content after construction.

2.3.2.1 Design CBR and Design Subgrade Strength Class

Figure 2.3 shows a detailed dry density/moisture content/CBR relationship for a sandy-clay soil that was obtained by compacting samples at several moisture contents to three levels of compaction. By interpolation, a design subgrade CBR of about 15 per cent is obtained if a relative density of 100 per cent of the maximum dry density obtained in the ASTM Test Method D 698 Test is specified and the subgrade moisture content was estimated to be 20 percent.

The structural catalogue given in this manual requires that the subgrade strength for design be assigned to one of six strength classes reflecting the sensitivity of thickness design to subgrade strength. The classes are defined in Table 2.1.

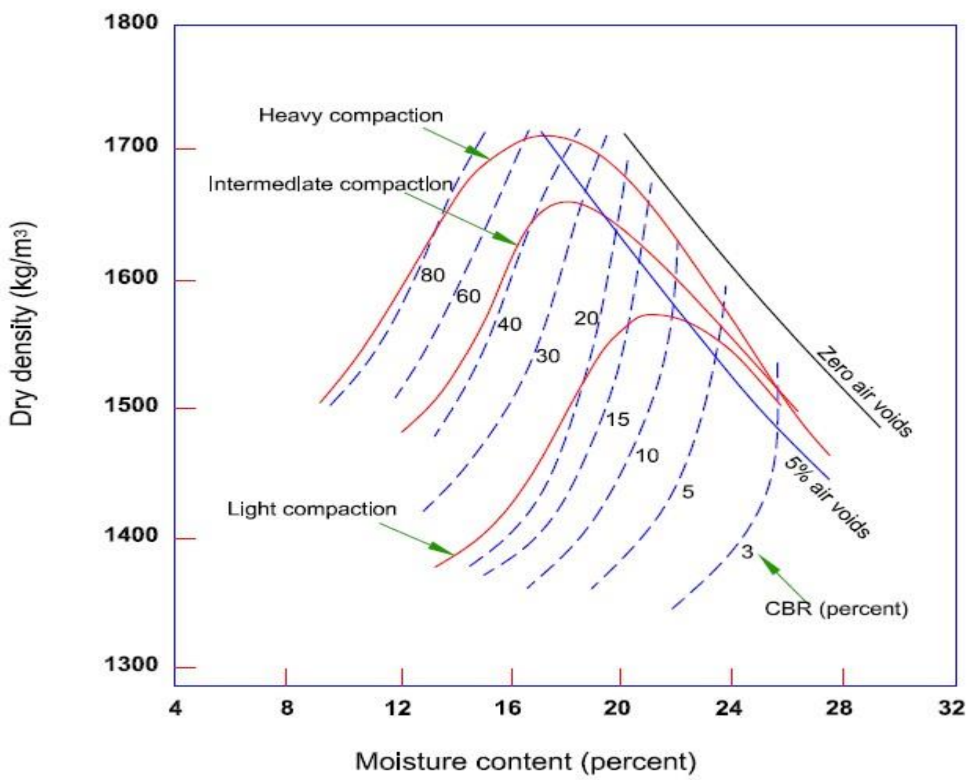

Figure 2.3 Dry Density, Moisture Content, CBR for Sandy-Clay Soil (ERA, 2013)

Table 2.1 Subgrade Strength Classes (ERA, 2013)

Class	CBR Range (%)
S1	<3
S2	3,4
S3	5,6,7
S4	8 - 14
S5	15 - 30
S6	>30

A less precise estimate of the minimum subgrade strength class can be obtained from Table 2.2. This table shows the estimated minimum strength class for five types of subgrade soil for various depths of water table, assuming that the subgrade is compacted to not less than 95 per cent of the maximum dry density attainable in the ASTM Test Method D 698 (Light Compaction). The table is appropriate for subgrade moisture Categories 1 and 2 but can be used for Category 3 if conservative strength estimates are acceptable.

Table 2.2 Estimated Design Subgrade Strength Class under Sealed Roads in the Presence of a Water Table (ERA, 2013)

Depth of water table from formation level (m)	Subgrade strength class				
	Non plastic sand	Sandy clay PI = 10	Sandy clay PI =20	Silty clay PI = 30	Heavy clay PI = 40
0.5	S4	S4	S2	S2	S1
1	S5	S4	S3	S2	S1
2	S5	S5	S4	S3	S2
3	S6	S5	S4	S3	S2

2.3.2.2 Delineation of Subgrade Areas

A road section for which a pavement design is undertaken should be subdivided into subgrade areas where the subgrade CBR can be reasonably expected to be uniform, without significant variations. Significant variations in this respect means variations that would yield different subgrade classes as defined above. However, it is not practical to create too many separate sections. The soils investigations should delineate subgrade design units on the basis of geology, pedology, drainage conditions and topography, and consider soil categories which have fairly consistent geotechnical characteristics (e.g. grading, plasticity, CBR). Usually, the number of soil categories and the number of uniform subgrade areas will not exceed 4 or 5 for a given road project unless the road is particularly long.

2.3.3 Selected Subgrade Materials and Capping Layers (GC)

These materials are often required to provide sufficient cover on weak subgrades. They are used in the lower pavement layers as a substitute for a thick sub-base to reduce costs, and a cost comparison should be conducted to assess their cost effectiveness.

In some of the design charts, substitution of part of the sub-base with GC quality material is allowed as mentioned in the footnotes to the charts. The substitution ratio is 1.3:1 so that 50mm of sub-base can be replaced with 65mm of GC, for example, provided that the rules in the footnotes are followed. Similarly, a layer of GC material on top of a weak subgrade effectively increases the subgrade class as illustrated in the design charts.

The requirements are less strict than for sub-bases. A minimum CBR of 15 per cent is specified at the highest anticipated moisture content measured on samples compacted in the laboratory at the specified field density. This density is usually specified as a minimum of 95 per cent of the maximum dry density in the ASTM Test Method D 1557 (Heavy Compaction). In estimating the likely soil moisture conditions, the designer should take into account the functions of the overlying sub-base layer and its expected moisture condition and the moisture conditions in the subgrade. If either of these layers is likely to be saturated during the life of the road, then the selected layer should also be assessed in this state. Recommended gradings or plasticity criteria are not given for these materials. However, it is desirable to select reasonably homogeneous materials since overall pavement behaviour is often enhanced by this. The selection of materials which show the least change in bearing capacity from dry to wet is also beneficial.

2.4 Subgrade Treatment

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.

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.

For subgrade of elastic modulus below 50MPa, strengthening measures are required in order to provide a strong and uniform support for the pavement and to allow road construction vehicles to pass over the subgrade without damaging the layer. This can be achieved by providing a thick layer of sub-base on the subgrade but it may be more economical to provide improved layer of selected materials. The provision of improved layer over a weak subgrade avoids the necessity of an extraordinarily thick sub-base, and provides an adequate working platform for sub-base compaction as well as reduces the risk of damage to the subgrade during construction. The CBR value of the improved layer shall be of at least 15%.

The recommended thicknesses of the improved layer for various CBR values of subgrade for pavements are shown in Table 2.3.

The sub-base forms the upper layer of the pavement foundation and provides a regulated working platform on which to transport, place and compact the bound layers of the pavement. Within a flexible pavement structure, the sub-base is also treated as a structural layer to spread the loading from the surface down to the subgrade.

The purpose of sub-base on pavements is primarily for controlling pumping, which can be achieved by using granular materials. The thickness of the sub-base layer is determined primarily from the strength of the subgrade, i.e. the CBR value.

The thickness of pavement will vary due to CBR value of sub-grade. CBR value also vary due to change of compaction. In rural roads, 100% MDD Standard Proctor is specified for sub-grade material for CBR value of 8%. In many of the Projects, equipment are available for compaction of 95% MDD modified. This advantage should be taken for obtaining better compaction and bearing capacity for the same material.

According to Mr. Hodge Kinson, Materials Expert of O " Sullivan & Graham who prepared the" Guide to the Design & Construction of Bitumen Surfaced Roads in Bangladesh - BRRL, Dhaka, Table 2.3 may be used.

Table 2.3 Depending on the CBR value of subgrade the thickness of Improved Sub-grade layer (Sullivan & Graham, 2012)

Subgrade CBR (Soaked)		Thickness of Improved Subgrade
Greater than	but Less than	
5%	8%	200 mm
4%	5%	250 mm
3%	4%	300 mm
2%	3%	450 mm

Minimum thickness of subbase shall be 150 mm for for traffic volume more than 0.3 million ESA for the design life.

Minimum thickness of Aggregate Base Course or WBM shall be 200 mm for having traffic volume more than 0.7 million ESA for the design life.

Typically it is thought that local soil will not give 8% CBR and imported or selected material is suggested. If 30% to 50% local soil is blended with selected material, CBR value may be attained from 15% to 40% when compacted to 100% MDD standard. From numerous experiments & field use, it is established that when poor soil is added to selected material, density as well as CBR increases. While blending the soil, it should be tested in soil laboratory; PI value of the combined blended material reduced.

Where the local soil will not give CBR more than 8%, then to reduce the thickness of pavement, selected or improved sub-grade material is suggested. This improved sub-grade material for 8% CBR value may be sand, silty or clayey sand or combination of sand-soil.

It is recommended that as a general practice the design for new construction should be based on the strength of samples prepared at optimum moisture content and dry density corresponding to Proctor compaction (standard or modified) as specified in field and soaked in water for a period of four days prior to testing. For field control, range of moisture should be specified i.e. field moisture should be + 1% to - 2% when Field density is tested. In some project areas, the CBR value of the Sub-grade or improved sub-grade material may be more than 20% to 30%; in that case no Sub-base is required.

A summary of the tests results of soils, sand, sand-soil and aggregate-sand-soil mixtures, For Road projects under Daulatdia - Jhenaidah - Kushtia, Daudkandi - Feni & Dohazari - Cox's Bazar Road sections, mostly locally available soils and only in Faridpur - Kushtia section blended fine local sand & soil gave more than 20% CBR (value achieved 20 to 35%) at a compaction of 95% MDD modified while the minimum CBR requirement specified was 20%. It is possible to get higher CBR if compaction requirement is increased.

2.5 Design of Improved Subgrade

If the material in cuttings or embankments does not meet the requirements of the selected foundation class, blended layer shall be constructed to improve the subgrade to achieve the design foundation class.

The improvement is designed to bring the existing native subgrade plus the blended layers up to an overall bearing strength level equivalent to that of the selected foundation.

The minimum thickness of each type of blended material required to improve the subgrade to a higher class. The minimum thicknesses have been calculated taking into account the respective elastic modulus of each class of soil, layers not exceeding 200mm to a dry density of at least 95% MDD (AASHTO T180)

2.6 Sources of Selected Materials

The selection of materials for a road pavement design is based on a combination of availability of suitable materials, environmental considerations, method of construction, economics and previous experience.

2.6.1 Choice of the source of Selected Materials

Whether or not a naturally-occurring material from a borrow-pit or a waste material is used as the source of imported bulk fill will depend on the particular circumstances. If a waste material is not available within an economic haulage distance then wastes are clearly not going to be used.

A borrow-pit can be sited close to the road project so that haulage distances and the use by construction of public roads are minimized.

The disadvantages of using wastes as a source of bulk fill are: increased haulage costs, disturbance caused by haulage, and the greater variability of waste materials.

The use of wastes rather than borrow materials usually involves additional haulage costs because a borrow-pit will, almost invariably, be closer to the road site than a waste tip (if it is not, the advantage of using waste is obvious). In some case it may be necessary to strengthen the roads used for haulage and the costs of doing so should be added to any extra costs of haulage.

The disturbance caused by haulage is an environmental disadvantage if the haulage loams use public highways causing added congestion, noise, dust and deposition of the material on the road. Because it is not easily quantified it tends to be ignored, but is nonetheless real to people who live close to the haulage route. The removal of material from a spoil heap may also mean the exchange of a permanent but still disbenefit, to which local inhabitants have to some extent become adapted, for a temporary, but widespread, mobile nuisance.

The third possible disadvantage of using wastes is the fact that, in general, the material from a waste tip will be more variable than material from a borrow-pit.

In this study a borrow-pit (selected materials) from Agaro town quarry site can be sited close to the study area at about 2.5km so that haulage distances and the use by construction of public roads are minimized.

CHAPTER THREE

3.0 MATERIALS AND RESEARCH METHODOLOGY

3.1 Introduction

This chapter will present the detailed procedures of all the laboratory tests performed to achieve the objectives of this study. The soils were collected from different locations in Agaro and all the samples used in this study were tested in the laboratory according to the available standard procedures by the American Association of State Highway and Transportation Officials and Ethiopian Road Authority or based on the literature review. For the laboratory testing program, selected materials were considered as a candidate blend material to improve weak subgrade soil types.

Agaro town, one of the reform urban centers of the Oromia region, is the second-largest urban center in Jima zone next to Jima city. The town is the administrative seat of the Gomma district and situated to the southwest of Addis Ababa at about 390kms and west of Jimma city at about 44kms. It is also located to the east of Bedele town (the closest large town) at about 93kms. Absolutely, Agaro town is found at 07^0 50’ 20” to 07^0 52’40” N latitude and 36^0 33’ 40” to 36^0 37'00" E longitudes. The topography of Agaro is dominated by broken land features. Small hills followed by valleys, river valleys and sometimes flat terrain prone to flooding are common land features throughout the town. This type of land feature induces problems of flooding and erosion. Undulating topographic features pose constraints for activities that require a considerable land area with a gentle slope. Most of the existing activities are undertaken on a few pocket areas found on top of hills with flat top and hillsides with a possible slope for respective purposes or activities. The elevation of hilltops gradually decreases from south to north. Valley bottoms in the southern parts of the town are narrow, whereas they are wide and flat in the northern part. The elevation of the town ranges between 1,546 meters above sea level (m.a.s.l.) and 1,796 m.a.s.l. with an altitudinal range of 1m. Consequently, the average elevation of the town is 1671 m.a.s.l., which would make its climate sub-tropic.

Agaro town is situated on Addis Ababa/Jima-Metu asphalt road that serves the town as a principal arterial street and a bypass (expressway) for those vehicles that are crossing the town. The gravel road of Agaro–Gera is the major sub-arterial street segment of the town. The remaining streets are un-surfaced roads connecting the town with rural villages such as Bulado, Koye, Dalecho, Bulbulo and the like. Such road segments are categorized under major and minor collector streets. The existing inner-city road network of the town is characterized by highly damaged asphalt roads, non-standard gravel, and earthen roads.

3.2 Materials Used

Different subgrade soil types of different plasticity (low PI to high PI) and selected materials from quarry site as shown in Figure 3.1 were selected for inclusion in this study.

The possible disadvantage of using wastes is the fact that increased haulage costs, disturbance caused by haulage, in general, the material from a waste tip will be more variable than material from a borrow-pit.

In this study A borrow-pit (selected materials) from Agaro town quarry site can be sited close to the study area at about 2.5km so that haulage distances and the use by construction of public roads are minimized.

Figure 3.1 Soil used in the study

3.3. Study Area

3.3.1 Location and Accessibility of the Study Area

Agaro is one of the Oromia towns that has sustained a long historical period. Its emergence and growth were closely related to the history of the socio-economic development of the Gibe states and imperial expansion towards the region in the ninetieth century. The town was established around 1811 E.C and currently is about 200 years old.

Agaro town had been recognized as a district seat in 1927/8. However, the town got a municipal status in 1942. Since this period it began to embark on various development endeavors including social and infrastructures (official municipal document). The development of Agaro town was further enhanced by the development of various social services. Commerce, mainly coffee trade, is the main activity in the town.

Coffee is indigenous to the region and grows both wild and cultivated since the early time a place close to Agaro known as Katta Muudaa Ga’a, in Chooche Lammi village is believed to the place of coffee origin. It is also believed that the estates of the Gibe kings were dominated by the coffee farms. Coffee remained one of the major trade items of the Gibe states. However, the rise in coffee price in the 1920s encouraged increased coffee plantation, since the mid-nineteenth century agents of the imperial regime and the local landlords became coffee cultivators.

Agaro town, one of the reform urban centers of the Oromia region, is the second-largest urban center in Jima zone next to Jima city. The town is the administrative seat of the Gomma district and situated to the southwest of Addis Ababa at about 390kms and west of Jimma city at about 44kms. It is also located to the east of Bedele town (the closest large town) at about 93kms. Absolutely, Agaro town is found at 07^0 50’ 20” to 07^0 52’40” N latitude and 36^0 33’ 40” to 36^0 37˙00¨ E longitudes (See Figure 3.2 below).

Physically the town is bounded by rural gandas (Kabales) in all directions. Moreover, the town has connected with seven towns (Gera, Sigmo, Sentema, Toba, Limu-Shay, Bashasha and Gembe) with road transport. Currently, the town provides the high order services to these towns. In addition, Agaro town has a suitable climate, accessible location, and relatively better urban services and infrastructure. Thus the town has created wide range interaction with the nearby towns. The people of the hinterland and people from different parts of the country love to go and work in Agaro town and expect high services.

Agaro town has a common boundary with rural settlements of Bulado in the north, Dalecho, and Kujo in the south, Chidero Susie in the east as well as KoyeSeja in the west. Currently, the town is composed of five kebeles/Ganda. According to the current base map, the town has a total area of 2610.45 hectares. The Agaro area is attractive in many ways. The beautiful scenery, the streams including Tamsa, Dogajja, Tijje, Qoree, Chessache, Bulbula and other; the beautiful peaks like Qinddilla are really impressive, nevertheless, the imperial agent, RasDesta might have equally attracted by the economic activities of the area, above all the trade center or the market. A pioneer church in the area, st George church was built in Agaro in 1923, as a first practical step for a move to Agaro by the imperial agents. In 1927/8 the seat of administration was shifted to Agaro and the Ras urged the boundary of the town to be demarcated.

The existing urbanized land which is situated within the boundary of Agaro town as per defined by the existing development plan has about 2.04 km length from High school in the north to Silase Church in the south side and 2.32 km from District administration in the east to Abattoir in the west when measured along the bypass road. In general terms, these two widths of the urbanized land indicate the town shaped compact to the east-west direction along the Asphalt road. But as the town is urbanized, the land stretched to the south its extent or the width from west to north declined and narrowed.

3.3.2. Project Location and Topography

The topography of Agaro is dominated by broken land features. Small hills followed by valleys, river valleys and sometimes flat terrain prone to flooding are common land features throughout the town. This type of land feature induces problems of flooding and erosion. Undulating topographic features pose constraints for activities that require a considerable land area with a gentle slope. Most of the existing activities are undertaken on a few pocket areas found on top of hills with flat top and hillsides with a possible slope for respective purposes or activities. The elevation of hilltops gradually decreases from south to north. Valley bottoms in the southern parts of the town are narrow, whereas they are wide and flat in the northern part. The elevation of the town ranges between 1,546 meters above sea level (m.a.s.l.) and 1,796 m.a.s.l. with an altitudinal range of 1m. Consequently, the average elevation of the town is 1671 m.a.s.l., which would make its climate sub-tropic.

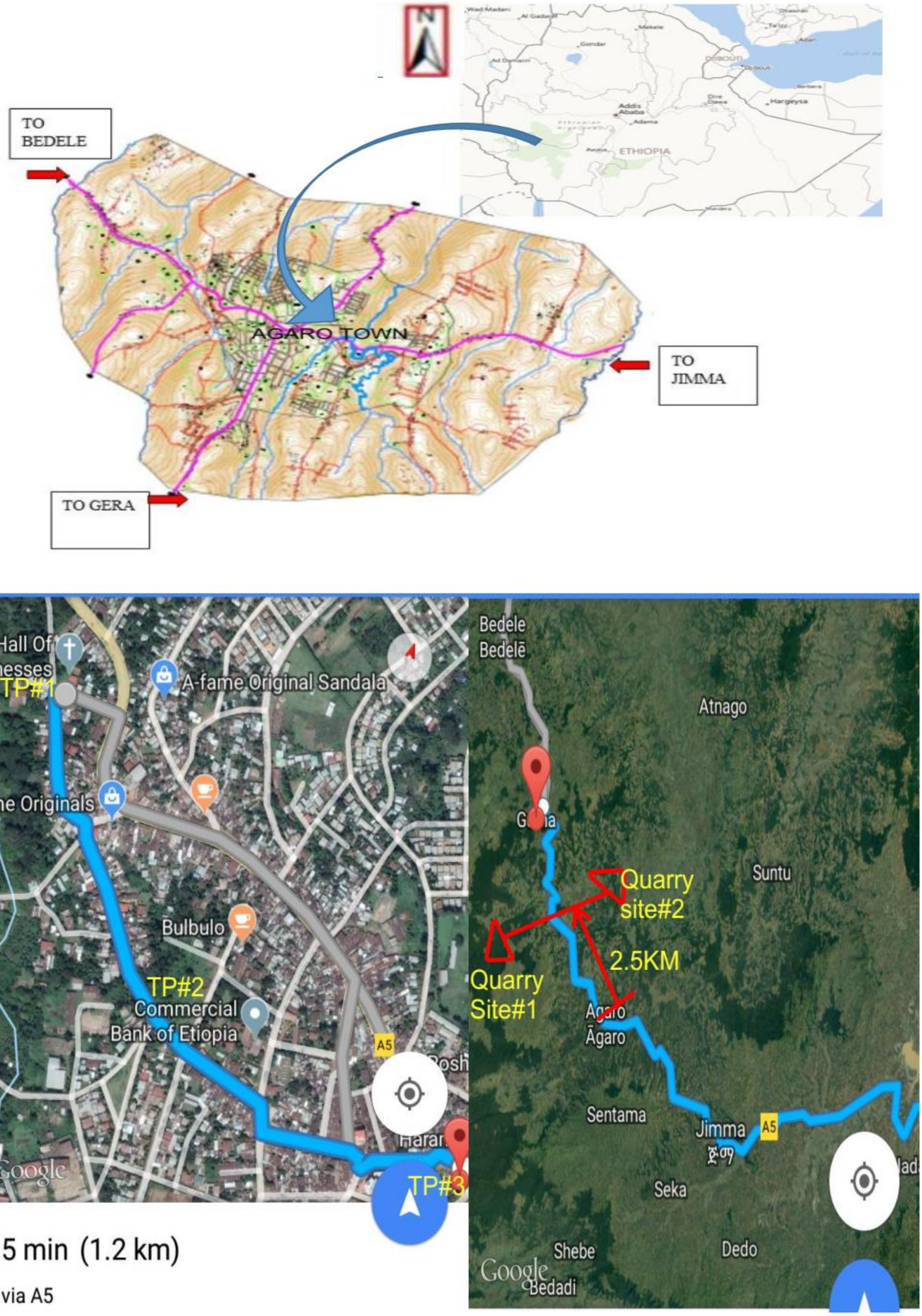

Figure 3.2 Location map of the study area (source: Google Maps)

3.3.3. The Climate of the Study Area

The mean temperature sensation for Agaro is found to be neutral (i.e., 20.6^{0}c). Agaro and its environments have eight continuous rainy months from March to October with a mean annual rainfall of 1,663.1mm. The wind direction of the area is east to west with a max speed of about 8km/hr.

3.4 Study Design

This research methodology followed the experimental type which designed to answer the research questions and achieve its objectives based on experimental findings through quantitative analysis approach. The overall activity and research process in the study include Reviewed related literatures, Problem identification of the study area, Material collection and preparation of the sample for laboratory test, Conduct laboratory test for subgrade and selected materials. Specify the optimum percent of selected materials required for improving Subgrade soil, and finally, conclusions and recommendations have been made based on the findings. The research project is expected to be completed within five months, including all the necessary activities that should be implemented. It is planned to conduct from May to October 2019.

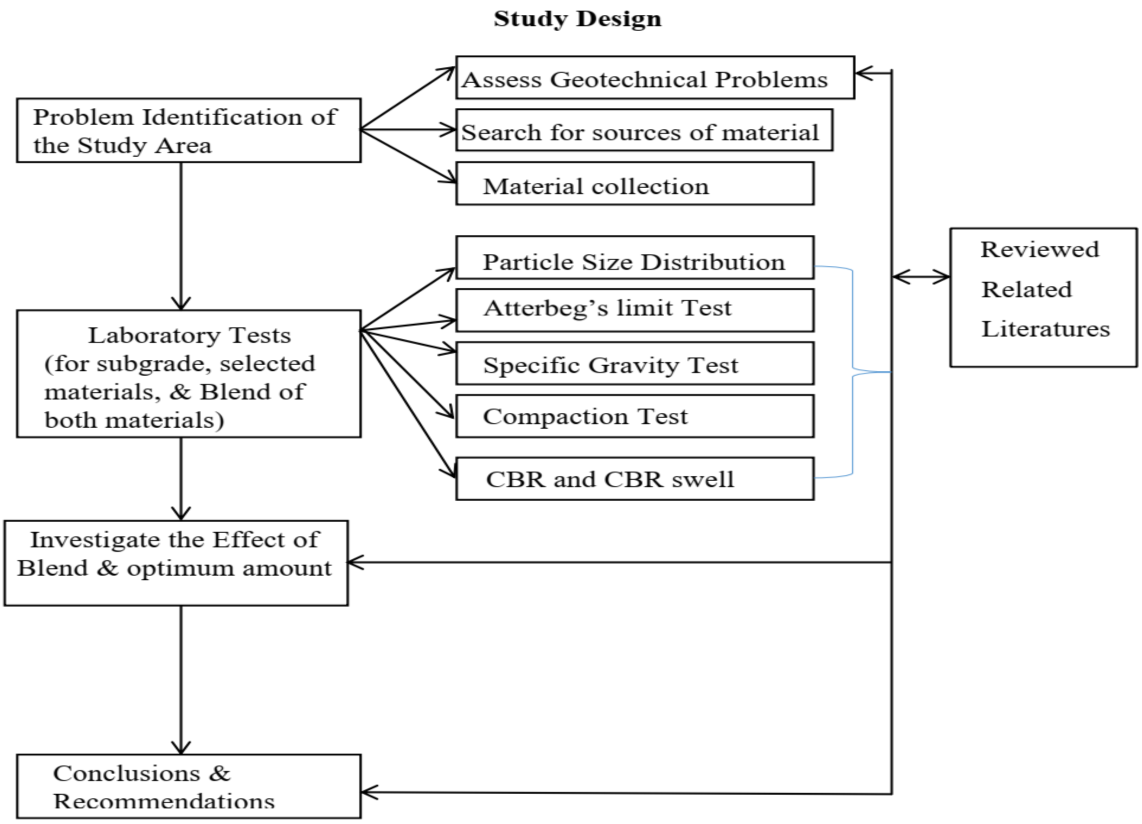

Figure 3.3 Study Design

3.5. Study Procedure

- ✓ Reviewed related literatures
- ✓ The researcher was identified the study stretch;
- ✓ Samples extracted by the statistical method of the different stations;
- ✓ Laboratory tests conducted (Sieve analysis, specific gravity, Atterberg limits, Compaction test, California Bering ratio (CBR) and CBR swell);
- ✓ The results are discussed compared with a standard specification of AASHTO, and ERA;
- ✓ Conclusions would be developed and
- ✓ Recommended appropriate remedial measure.

3.6. Study Variables

The research variables are both the independent and dependent variables

3.6.1. Independent Variables

- ✓ Grain size analysis of particle size distribution;
- ✓ Atterbeg's limit (LL, PL, and PI);
- ✓ Specific Gravity
- ✓ Compaction (relation between OMC and MDD);
- ✓ California bearing ratio (CBR), and CBR swell.

3.6.2. Dependent Variables

- ✓ Effect of selected materials blend on the performance of weak subgrade soil

3.7. Sample Size and Data Collection Process

3.7.1. Sample Size

In order to generate data for the general and specific objective, field survey and a laboratory test was carried out on diffrent subgrade soils were statistically collected from Agaro Town. Excavation will be made by hand using a shovel. Diffrent representatives selected materials samples were collected from Agaro town quarry sites with a 400m difference. The samples collected for this study can be disturbed samples.

3.7.2. Data Collection Process

Quantitative data utilized based on the necessary input parameters for analysis and compare with AASHTO, and ERA specification manuals.

3.8. Field Observation

Field observation was necessary to begin by site visit was taken on the whole portion of the study area. The data collection process includes field visual inspection, Field investigation, sampling representative samples in preparation for laboratory tests, and measurements. Diffrent subgrade soils were statistically collected from Agaro Town. Excavation will be made by hand using a shovel. Representative selected materials were collected from Agaro town quarry sites with a 400m difference. For this study disturbed samples of 300kg collected for each subgrade and selected materials were tested in the laboratory.

3.9. Sampling Technique

For Road along Kingdom Hall - Bulbulo (commercial Bank of Ethiopia, Main Branch) - Total Agaro Service area Road section, mostly locally available weak soils were statistically collected and selected materials from the Agaro Town quarry which sited close to the study area at about 2.5km collected to make sure as the engineering parameters had certain characteristics as applied for this study.

Figure 3.4 Samples extracted from subgrade and Quarry site

3.10. Instrument Used

The following instruments and software were used for the research project study: Meter tape, plastic bags, manual hand auger equipment, laboratory equipment, GPS, Digital Camera for documentation, MS Word, and Excel into analysis laboratory data and display research data were used.

3.11. Laboratory Test Materials

Soils are used as construction material and also as a foundation for engineering structures. However, the wide range of properties under different conditions affects their performance and use. For this reason, soils have to be properly sampled and subjected to various tests so as to understand their properties towards these engineering applications.

Sample extracted and laboratory performed are Atterbegs limit (for comparison and determination of liquid limit and plastic limit), Specific Gravity, Grain Size Analysis (distribution of particle size analysis), Compaction test (for determination of maximum dry density and optimum moisture contents), California Bearing ratio (CBR) test (for Determine of shear strength of materials. The tests are performed according to AASHTO, and ERA specifications.

3.11.1 Grain Size Distribution

Gradation, or the distribution of particle size within a soil, is an essential descriptive feature of soils. Soil textural (e.g., gravel, sand, silty clay, etc.) and engineering classifications are based in large part on gradation, and many engineering properties like relative compaction, strength, swelling potential are closely correlated with gradation parameters. Gradation is measured in the laboratory using two tests: a mechanical sieve analysis for the sand and coarser fraction, and a hydrometer test for the silt and finer clay material. Grain size distribution is done by a mechanical sieve to this study in order to determine the percent pass of soils through No 200 sieve for AASHTO soil classification purposes and group index determination.

Apparatus: Series of standard sieves (for gravel fraction 4.75-75mm aperture size, and for sand fraction 0.075-2mm aperture size), Lid (cover), Pan (receiver), sieve shaker, Balance sensitive to 0.1g, Soft wire brush, Sample splitter, Mortar, and rubber-covered pestle for breaking up aggregates of soil particles, & Oven.

Figure 3.5 Apparatuses for Grain Size Analysis Test

3.11.2 Atterberg Limits

These clays are checked for their liquid limits and plasticity Index in accordance with AASHTO T 89 and 90 to determine the nature and response of subgrade soils upon change to moisture content. Only the materials passing through sieve size 0.425mm (No 40) are considered for Atterberg limit tests. The liquid limit and plastic limit tests collectively are called the tests for Atterberg limits. These Atterberg limit test results are reported to the nearest whole number with the number of blows in the logarithmic scale in the horizontal direction against the percent of moisture content in the vertical direction. The moisture content at 25 blows in the graph represents the value of the Liquid limit for that particular subgrade soil sample (Arora, 1997).

In general, soils that exhibit plastic behavior over wide ranges of moisture content and that have a high liquid limit, have greater potential for swelling and shrinking. Besides, the amounts of swell will increase with the amount of clay present in the soils. The plasticity index of soils is the difference between the liquid limits and the plasticity indices of those soils (Chen 1988).

Apparatus: Grooving tools, Casagrande, tools, and spatula.

Figure 3.6: Apparatuses for atterberg's test

Once the grain size distribution and Atterberg limits are determined, then the classification for the subgrade soils has to be carried out. Based on the results from laboratory tests the soils have been classified as per AASHTO M 145 as follows in Table 3.1.

Table 3.1 Soils classification of the study area by AASHTO Method

S. No	Soil classes
TP1	A-7-5(56)
TP2	A-7-5(38)
Selected granular material (QM)	A-2-6(1)

It can be concluded, the subgrade soil samples are under soils group A–7–5, the study area show GI greater than 20. This is an indication that the sections of the alignment with high PI and GI values will be very poor to support the traffic load and are considered hereafter to be weak for foundation material.

Tests results obtained for the study area showed that selected materials are found to be Silty or Clayey Gravel and Sand material having a PI value less than 20%. From the GI values, it can be concluded that selected granular materials are considered suitable soils to be used as improved layer material under pavement.

3.11.3 The Group Index (GI)

The nature of the sub-grade material can also be characterized by their Group Index Values. The Group Index characterizes the clayey nature of the soil and is calculated by equation.3.1 as per AASHTO M145-91 (1995);

$$GI = (F - 35)\,[0.2 + 0.005\,(LL - 40)] + 0.01\,(F - 15)\,(PI - 10) \qquad \text{eq. 3.1}$$

Where; 'F' is the percentage passing 0.075 mm (No. 200) sieve expressed as a whole number. This percentage is based only on the material passing 0.075 mm. 'LL' is the liquid limit and 'PI' is the plasticity index. Eq. 3.1 is applicable for soil classes that do not belong to A–2 – 6 and A–2–7. For these soil classes, only partial group index is applied, for this equation.3.2 is used.

$$GI = 0.01(F - 15)\,(PI - 10) \qquad \text{eq. 3.2}$$

One of the assumptions in this formula is that, when the value is negative, the group index shall be reported as zero (0). The other assumptions are discussed below;

Under average conditions of good drainage and thorough compaction, the supporting value of a material as subgrade may be assumed as an inverse ratio to its Group index; that is, a Group index of zero indicates a "good" subgrade material and a Group index of 20 or greater indicates a "very poor" sub-grade material" (AASHTO, 1993).

The Group Index values are shown in parenthesis for each of the samples collected during field investigation in the study area. Classification of subgrade soils using Group index values (GI) taken from test results is also shown in Table 3.2.

Table 3.2 Soils classification of the study area with its Group Index Values

		% pass	LL	PI	Group classification	Usual types of significant constituent materials	SOIL	General Rating as Subgrade
TP1	#10	99.0	79.25	49.05	A-7-5	Clayey Soils	A-7-5(56)	GI>20
	#40	97.8						very Poor
	#200	96.7			PI ≤ LL-30			
TP2	#10	98.6	67.5	32.42	A-7-5	Clayey Soils	A-7-5(38)	GI>20
	#40	96.9						very Poor
	#200	94.3			PI ≤ LL-30			
QM	#10	35.3	37.87	17.05	A-2-6	Silty or Clayey Gravel and Sand	A-2-6(1)	Good,
	#40	26.7						Suitable for sub grade
	#200	22.3						

3.11.4 Compaction Test (Density - Moisture Relationship)

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

The subgrade soil samples were subjected to the determination of maximum dry density (MDD) and optimum moisture content (OMC) in the laboratory. The moisture-density curve is different for each soil type.

Granular, well-graded soils generally have fairly high maximum densities at low optimum moisture contents, while clay soils have lower densities. The edge-to-side bonds between clay particles resist compaction efforts to force them into a denser structure. Whereas, well-graded granular soils have spaces between large particles that are filled with smaller particles when compacted that lead to a higher density than with uniform soils (FHWA, 2006).

Figure 3.7 Apparatuses for a compaction test

Table 3.3 Range of MDD & OMC of soils in the study area.

	Soils Descripitons	AASHTO classes	MDD, g/cc	OMC, %
TP1	Materials with moderate plasticity index (clay soils)	A - 7 - 5	1.6475	27.90
TP2	Materials with moderate plasticity index (clay soils)	A - 7 - 5	1.57	28.8
Selected material (QM)	Silty or Clayey Gravel and Sand	A - 2 - 6	2.1675	14

3.11.5 California Bearing Ratio (CBR) Test

The CBR is a comparative measure of the shear resistance of the soil. The test consists of measuring the load required to cause a piston of standard area, 19.35cm^2 (3 inch^2) to penetrate a soil specimen at a specified rate, 1.25mm/min. This load is divided by the load required to force the piston to the same depth in a standard sample of crushed stone (a high quality crushed stone material with a CBR value of 100%) (AASHTO, 1993).

CBR is the most widely used method for designing pavement structures. The method is primarily intended but not limited to evaluate the strength of cohesive materials. The test procedure is based on, American society for testing and materials, AASHTO T 193. The CBR value for a soil depends upon its density, molding moisture content and moisture content after soaking. For the present study three points, CBR tests were carried out with 4 days soaking which helped to anticipate the subgrade soils at the worst moisture conditions.

Figure 3.8: Apparatuses for CBR test

To determine the strength and swelling potential of the subgrade soil samples, the test has been carried out by 4-days soaking-3-point CBR and loaded Swell testing procedure. The subgrade soil strength has been used for design purposes by interpolating the CBR values at different compaction levels, with 10, 30 and 65 blows.

CBR tests can be performed either through one point or with three points for pavement material. For one point system, the CBR value at 100% MDD is considered for design, but for three points, usually the CBR value at 95% of MDD is the design CBR. When CBR tests are conducted for cohesive soils by three-point methods at 4 days soaking conditions, the minimum density obtained at 65 blows of CBR value shall be the Maximum Dry Density, MDD, obtained at 56 blows and the value of penetration at 2.54mm is higher than at 5.08mm, which will be considered for design purpose. When the reverse happens, the penetration value at 5.08mm is considered after the second trial (Arora, 1997).

3.11.6 The CBR Swell Values

The swelling potential tests conducted during CBR soaking can also be used to estimate the weak nature of the subgrade soils. The swelling potential is defined as the percentage swell of a laterally confined sample which has been compacted to MDD at OMC and allowed to swell under a surcharge load of 4.54kg conducted on CBR specimen (AASHTO T 193).

Table 3.4 CBR swell ranges of soils (AASHTO T 193).

No	Swell range (percentage)	Description	Remarks
1	≤ 1.5	low	suitable
2	1.5 to 2.00	intermediate	suitable
3	> 2	high	unsuitable

CHAPTER FOUR

4. RESULTS AND DISCUSSIONS

4.0 Characterization of Materials

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

4.1 Properties of Subgrade Soils and Selected Materials

The laboratory test carried out has been focused to investigate the grading, Liquid Limits (LL), Plasticity Index (PI), Specific gravity, maximum dry density (MDD), optimum moisture content (OMC), Soil Strength (CBR), and the potential to swell of the subgrade soils and Selected Materials.

4.1.1 Grain Size Distributions (Gradations)

Gradation, or the distribution of particle size within a soil, is an essential descriptive feature of soils. Soil textural (e.g., gravel, sand, silty clay, etc.) and engineering classifications are based in large part on gradation, and many engineering properties like relative compaction, strength, swelling potential are closely correlated with gradation parameters. Gradation is measured in the laboratory using two tests: a mechanical sieve analysis for the sand and coarser fraction, and a hydrometer test for the silt and finer clay material. Grain size distribution is done by a mechanical sieve to this study in order to determine the percent pass of soils through No 200 sieve for AASHTO soil classification purposes and group index determination.

The nature of the subgrade material can also be characterized by their Group Index Values. The Group Index characterizes the clayey nature of the soil and is calculated by equation.4.1 as per AASHTO M145-91 (1995);

$$GI = (F - 35)\,[0.2 + 0.005\,(LL - 40)] + 0.01\,(F - 15)\,(PI - 10) \qquad \text{……….….eq. 4.1}$$

Where; ‘F’ is the percentage passing 0.075 mm (No. 200) sieve expressed as a whole number. This percentage is based only on the material passing 0.075 mm. ‘LL’ is the liquid limit and ‘PI’ is the plasticity index. Eq. 4.1 is applicable for soil classes that do not belong to A–2 – 6 and A–2–7. For these soil classes, only partial group index is applied, for this equation.4.2 is used.

$$GI = 0.01(F - 15)(PI - 10) \quad \text{eq. 4.2}$$

One of the assumptions in this formula is that, when the value is negative, the group index shall be reported as zero (0). The other assumptions are discussed below;

Under average conditions of good drainage and thorough compaction, the supporting value of a material as subgrade may be assumed as an inverse ratio to its Group index; that is, a Group index of zero indicates a "good" subgrade material and a Group index of 20 or greater indicates a "very poor" sub-grade material" (AASHTO, 1993).

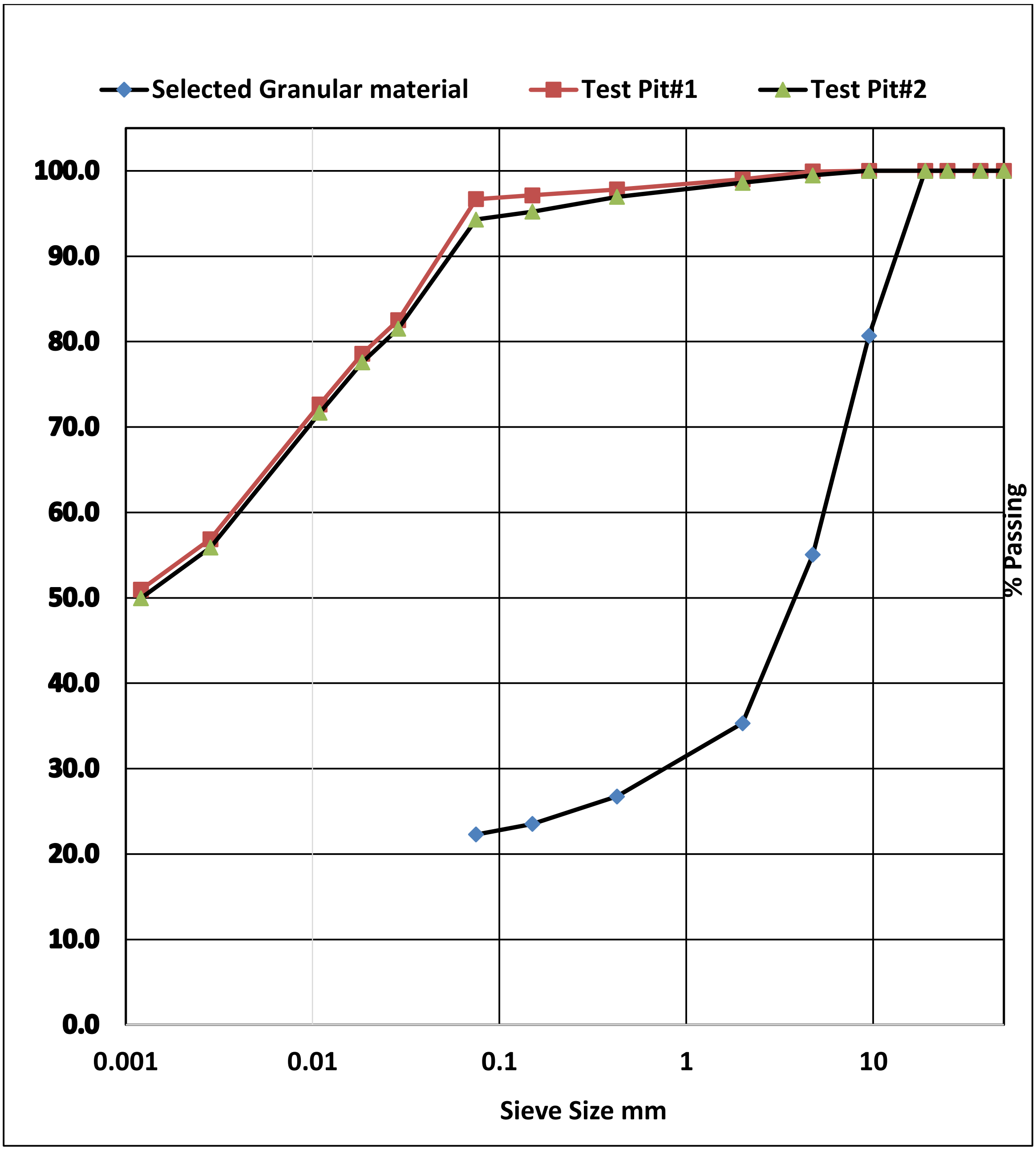

Figure 4.1: Material Grading Data for the Study Area

Table 4.1 Subgrade soils and Selected Materials classification by AASHTO Method

		% pass	LL	PI	Group classification	Usual types of significant constituent materials	SOIL	General Rating as Subgrade
TP1	#10	99.0	79.25	49.05	A-7-5	Clayey Soils	A-7-5(56)	GI>20
	#40	97.8						very Poor
	#200	96.7			PI ≤ LL-30			
TP2	#10	98.6	67.5	32.42	A-7-5	Clayey Soils	A-7-5(38)	GI>20
	#40	96.9						very Poor
	#200	94.3			PI ≤ LL-30			
QM	#10	35.3	37.87	17.05	A-2-6	Silty or Clayey Gravel and Sand	A-2-6(1)	Good, Suitable for sub grade
	#40	26.7						
	#200	22.3						

It can be concluded that, the subgrade soil samples are under soils group A–7–5, the study area show GI greater than 20. This is an indication that the sections of the alignment with high PI and GI values will be very poor to support the traffic load and are considered hereafter to be weak for foundation material.

Tests results obtained for the study area showed that selected materials are found to be Silty or Clayey Gravel and Sand material having a PI value less than 20%. From the GI values, it can be concluded that selected granular materials are considered suitable soils to be used as improved layer material under pavement.

4.1.2 Atterberg Limits

Correspond to values of moisture content where the consistency of the soils change as it is progressively dried from the slurry. Plasticity is the response of soil to changes in moisture content. When adding water to soil changes its consistency from hard and rigid to soft and workable, the soil is said to exhibit plasticity. Clays can be very plastic, silts are only slightly plastic, and sands and Gravels are non-plastic. For fine-grained soils, engineering behavior is often more closely correlated with plasticity than gradation. Plasticity is a key component of AASHTO soil classification. Soil plasticity is quantified in terms of Atterberg limits. The liquid limit (LL), defines the transition between the liquid and plastic states while the transition between the plastic and semi-solid states defines the plastic limit (PL) and the difference between the two values is termed as the plasticity index (PI). The shear strength of clay soils at its liquid limit is constant but variable at the plastic limits. Arora (1997) on the other hand, stated that the shear strength of all soils at the liquid limits is constant and equals to 2.7kN/m^2.

The clay material having liquid limits exceeding 60%, plasticity index exceeding 30% is considered unsuitable as subgrade material. Among the index tests, the most important consistency index is the plasticity index; PI. Clay soils have a higher value of plasticity than the granular soils.

Table 4.2 Soil Consistency Test Pit #1

Soil Consistency Test Result (Test Method：AASHTO T89, T90)					
Sampled and Tested by:- Ahmed Simeneh					
Material location	Agao Town				
Sampling date	15/08/2019		Pit Number	Pit #1	
Testing date	05/09/2019				
Material for	SUBGRADE				
	Liquid Limit			Plastic Limit	
No. of Blows	32	24	17		
Container Number	LL-1	LL-2	LL-3	PL-1	PL-2
Wt. of Container + Wet Soil (g) = (W_1)	49.507	28.856	24.51	18.317	16.68
Wt. of Container + Dry Soil (g) = (W_2)	43.001	22.75	16.284	15.58	14.04
Wt. of Container (g) = (W_3)	34.64	14.947	6.341	6.47	5.31
Weight of Moisture (g) = (W_1 - W_2) = A	6.51	6.103	8.228	2.74	2.65
Weight of Dry Soil (g) = (W_2 - W_3) = B	8.36	7.81	9.94	9.11	8.72
Moisture Content (%) = (A / B)x 100	77.85	78.18	82.8	30.08	30.32
		AV. Plastic Limit		30.2	

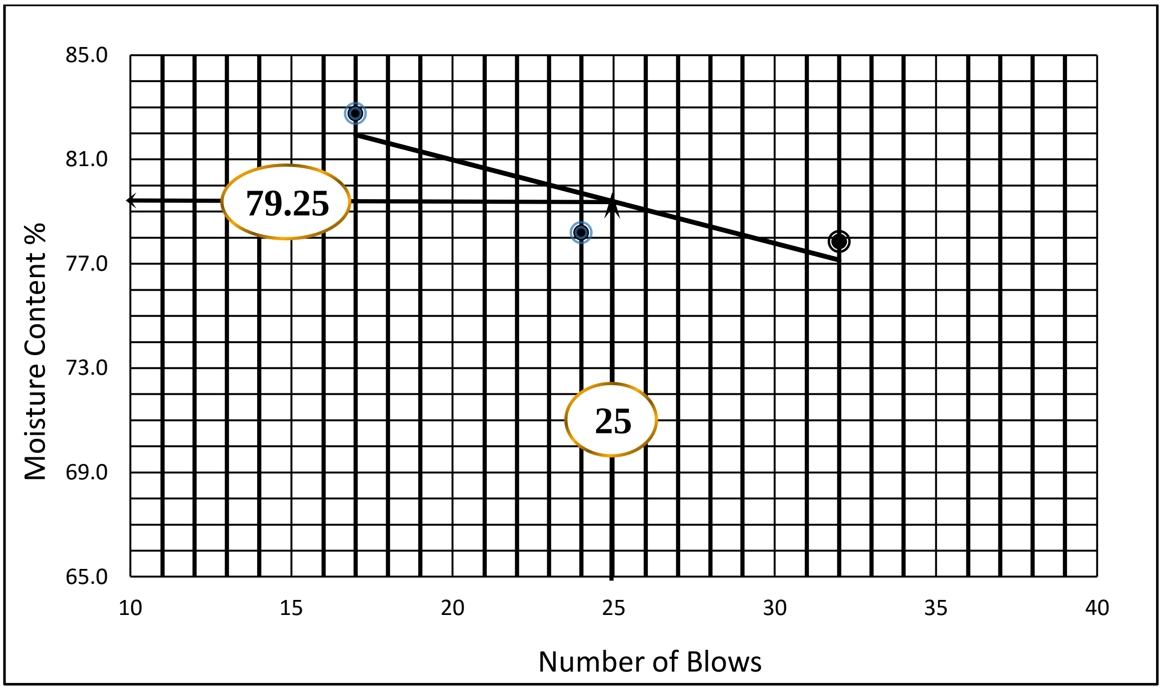

Figure 4.2 Soil Consistency Test Pit #1

Table 4.3 Soil Consistency Test Selected Materials (QM)

Soil Consistency Test Result (Test Method : AASHTO T89 , T90)					
Sampled and Tested by:- Ahmed Simeneh					
Material location	Agao Town				
Sampling date	15/08/2019		Selected Granular Materials (QM)		
Testing date	06/09/2019				
Material for	SUBGRADE				
	Liquid Limit			Plastic Limit	
No. of Blows	34	23	19		
Container Number	LL-1	LL-2	LL-3	PL-1	PL-2
Wt. of Container + Wet Soil (g) = (W_1)	32.67	46.855	31.95	35.968	36.44
Wt. of Container + Dry Soil (g) = (W_2)	28.515	43.15	27.963	32.85	33.18
Wt. of Container (g) = (W_3)	17.60	33.074	17.702	17.89	17.47
Weight of Moisture (g) = (W_1 - W_2) = A	4.16	3.705	3.989	3.12	3.26
Weight of Dry Soil (g) = (W_2 - W_3) = B	10.92	10.08	10.26	14.95	15.72
Moisture Content (%) = (A / B)x 100	38.07	36.77	38.9	20.89	20.75
		AV. Plastic Limit		20.8	

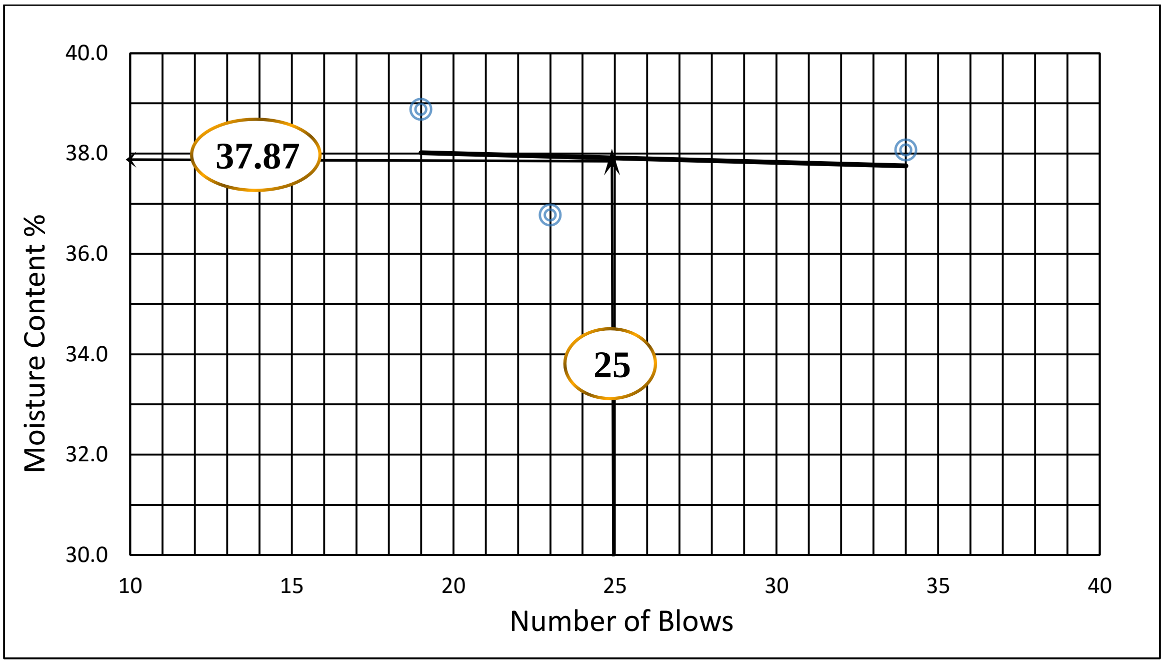

Figure 4.3 Soil Consistency Test Selected Materials (QM)

Chen (1988) demonstrated that the plasticity indices and the liquid limits are useful indices for determining the swelling characteristics of clays. In addition, the liquid limits and the swelling of clays both depend on the amount of water clay tries to absorb as shown in table 4.4.

In general, the larger the plasticity index implies more problems associated with the use of the soil as an engineering material, such as road subgrade. Many soil properties and engineering behaviors have been correlated with the plasticity index including swelling shrinkage potential (USGS 1999).

Table 4.4 Relationship between swelling potential and plasticity index, PI (Chen, 1988)

Plasticity index, PI (%)	Swelling potential
0 - 15	Low
10 - 35	Medium
20 - 55	High
≥ 35	Very high

Table 4.5 Swelling potential and plasticity index of soil in the study area.

No	Test Performed	Standard Test Method	Accepted Criteria	Test Results	Swelling potential	Remarks
TP1	Liquid Limit	AASHTO T 89	Max. 60%	79.25		unsuitable
	Plasticity Index	AASHTO T 90	Max. 30%	49.05	High	
TP2	Liquid Limit	AASHTO T 89	Max. 60%	67.5		unsuitable
	Plasticity Index	AASHTO T 90	Max. 30%	32.42	High	
TP3	Liquid Limit	AASHTO T 89	Max. 60%	60		Poor to Good
	Plasticity Index	AASHTO T 90	Max. 30%	28	Medium	
QM	Liquid Limit	AASHTO T 89	Max. 60%	37.87		suitable
	Plasticity Index	AASHTO T 90	Max. 30%	17.05	Low	

It was observed from the Atterberg limits test that the plasticity index increases with the amount of fine fraction in the soils.

For pavement design, taking the higher value of the swelling potential may be advantageous in order to make the safest estimation. However, other parameters such as; climate, drainage, and economy, should also be considered in conjunction with safety.

4.1.3 The Group Index (GI) Values

The nature of the subgrade material can also be characterized by their Group Index Values. The Group Index characterizes the clayey nature of the soil and is calculated by equation.3.1, already discussed in Chapter 3 of this thesis.

Eq. 3.1 (Chapter 3) is applicable for soil classes that do not belong to A–2–6 and A–2–7. For these soil classes, only partial group index is applied i.e $GI = 0.01(F - 15)(PI - 10)$.

One of the assumptions in this formula is that, when the value is negative, the group index shall be reported as zero (0). The other assumptions are already discussed in Chapter 3.

Table 4.6 Group Index value of each of the samples collected in the study area.

	Sieve	% pass	LL	PI	Usual types	Group Index (GI)	Soil
TP1	#10	99.0	79.25	49.05	Clayey Soils	56.3	A-7-5(56)
	#40	97.8					
	#200	96.7					
TP2	#10	98.6	67.5	32.42	Clayey Soils	37.8	A-7-5(38)
	#40	96.9					
	#200	94.3					
Selected Granular Materials	#10	35.3	37.87	17.05	Silty or Clayey Gravel and Sand	0.51	A-2-6(1)
	#40	26.7					
	#200	22.3					

A perusal of Table 4.6 It can be concluded that, the subgrade soil samples are under soils group A–7–5, the study area show GI greater than 20. This is an indication that the sections of the alignment with high PI and GI values will be very poor to support the traffic load and are considered hereafter to be weak for foundation material.

Tests results obtained for the study area showed that selected materials are found to be Silty or Clayey Gravel and Sand material having a PI value less than 20%. From the GI values, it can be concluded that selected granular materials are considered suitable soils to be used as improved layer material under pavement.

4.1.4 Density- Moisture Relationships of Soils

The dry density achieved depends upon the type of soils. Cohesive soils have high air voids. These soils attained a relatively lower MDD as compared with the cohesionless soils. Such soils require more water than cohesionless soils and therefore, OMC is higher.

The moisture content and plasticity index of the subgrade soils of the present study area are shown in Table 4.9. From this table, it can be concluded that highly plastic material has high moisture content.

In general, coarse-grained soils can be compacted to higher dry density than the fine-grained soils. According to FHWA (2006), soil compaction is one of the most important geotechnical concerns during the construction of highway pavements and related fills and embankments. Compaction improves the engineering properties of soils in many ways, including,

- ✓ Decreased compressibility, which reduces the potential for excessive long-term settlement.
- ✓ Increases strength which increases bearing capacity and decreases instability potential for slopes.
- ✓ Decreased hydraulic conductivity (permeability), which inhibits the flow of water through the soil.
- ✓ Decreases void ratio, which reduces the amount of water that can be held in the soil and, thus, helps maintain desired strength and stiffness properties and increases erosion resistance.

Table 4.7 Moisture Density Relationship of Pit #1

Moisture Density Relationship of Soil					
Test Method: AASHTO T-180 METHOD D					
Sampled and Tested by- Ahmed Simeneh					
Material location	Agao Town				
Sampling date	15/08/2019	Pit Number		Pit #1	
Testing date	10/09/2019				
Material for	SUBGRADE				
DENSITY	Trial Number	1	2	3	4
	Weight Of Soil + Mold g	10559.3	10988.9	10828.6	10702
	Weight Of Mold g	6,653	6,653	6,653	6,653
	Weight Of Soil g	3906.8	4336.4	4,176	4,050
	Volume Of Mold cc	2060.214	2060.214	2060.214	2060.214
	Wet Density Of Soil g/cc	1.90	2.10	2.03	1.97
MOISTURE	Container Number	ZE	SG3	NB	P1
	Wet Soil + Container g	108.0875	85.2115	63.628	86.681
	Dry Soil + Container g	93.87	71.364	53.057	68.991
	Weight Of Water g	14.2175	13.8475	10.571	17.69
	Weight Of Container g	33.9	21.5	17.66	17.77
	Weight Of Dry Soil g	60.0105	49.815	35.397	51.22
	Moisture Content %	23.69	27.80	29.86	34.54
DRY Density of Soil g/cc		1.53	1.65	1.56	1.46

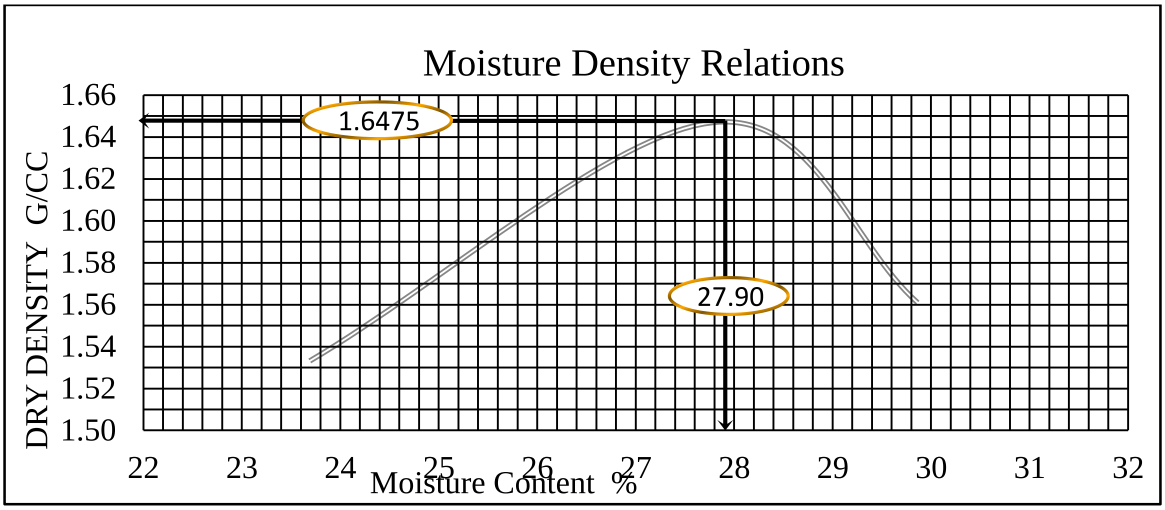

Figure 4.4 Moisture Density Relationship of Pit #1

Table 4.8 Moisture Density Relationship of Selected Materials (QM)

Moisture Density Relationship of Soil					
Test Method: AASHTO T-180 Method D					
Sampled and Tested by- Ahmed Simeneh					
Material location	Agao Town				
Sampling date	15/08/2019	Selected Granular Materials (QM)			
Testing date	10/09/2019				
Material for	SUBGRADE				
DENSITY	Trial Number	1	2	3	4
	Weight Of Soil + Mold g	11340.2	11608.8	11714.6	11342.6
	Weight Of Mold g	6,549	6,549	6,549	6,549
	Weight Of Soil g	4791.4	5060	5165.8	4793.8
	Volume Of Mold cc	2060.214	2060.214	2060.214	2060.214
	Wet Density Of Soil g/cc	2.33	2.46	2.51	2.33
MOISTURE	Container Number	G7	P3	N	M
	Wet Soil + Container g	110.15	178.269	242.186	245.357
	Dry Soil + Container g	102.623	161.48	211.211	215.221
	Weight Of Water g	7.527	16.789	30.975	30.136
	Weight Of Container g	17.389	36.0	18.2	18.031
	Weight Of Dry Soil g	85.234	125.485	193.018	197.19
	Moisture Content %	8.83	13.38	16.05	15.28
Dry Density Of Soil g/cc		2.14	2.17	2.16	2.02

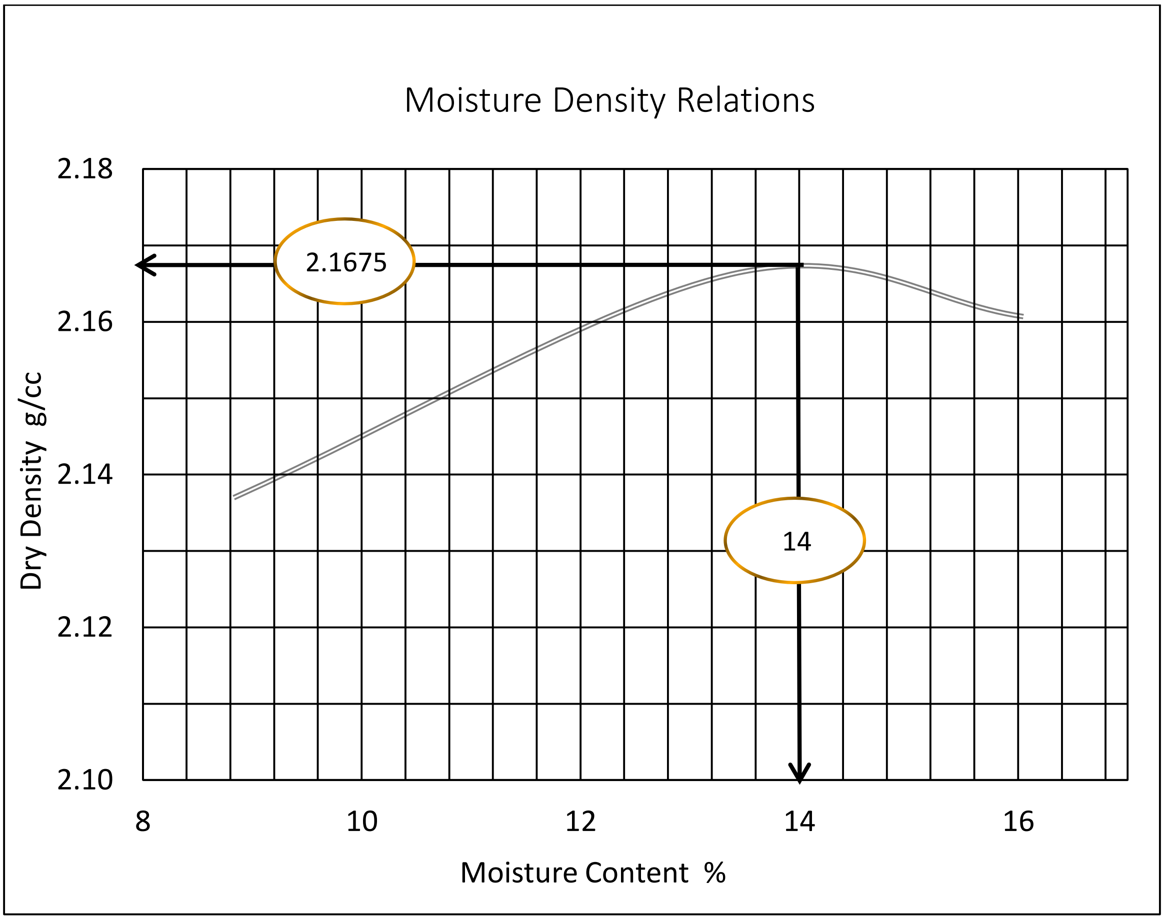

Figure 4.5 Moisture Density Relationship of Selected Granular Materials (QM)

Table 4.9. The Optimum Moisture Content, Maximum Dry Density, and Plasticity Index of the Soil Samples for the Study Area.

No.	PI	MDD,g/cc	OMC,%
TP1	49.05	1.6475	27.90
TP2	32.42	1.57	28.8
Selected granular material	17.05	2.1675	14

A perusal of Table 4.9 shows the optimum moisture content and maximum dry density of the soil samples for the study area. The optimum moisture content obtained from the tests ranges from 27.90% to 28.8%. Whereas the maximum dry density ranges from 1.6475g/cc to 1.57g/cc. A higher value of dry density is obtained in coarser materials than fine ones. Dry density is related to the voids ratio and plasticity nature of the material.

4.1.5 California Bearing Ratio (CBR) of the Subgrade Soils

The strength of the subgrade soil material of the study area was conducted on samples compacted to optimum moisture content. The CBR values at 95% of the Modified AASHTO (T180, Method D) Density have been interpolated from the CBR at densities obtained from different compaction levels and by interpolating single compaction level. The test results are shown in Appendix D. The natural subgrade soils in the study area, which has poor engineering properties as subgrade material. During excavation, the subgrade soils are again checked for vertical and lateral continuity of the soil texture in order to classify as suitable or unsuitable for the road foundation. The CBR values of coarser materials are higher than the fine material. Similarly, well-graded soils attain higher values of CBR than the poorly or gap graded ones (AASHTO, 1993)

California Bearing Ratio is a measure of shearing resistance of the material under controlled density and moisture conditions. The test consisted of causing a cylindrical plunger of 50mm diameter to penetrate a pavement component material at 1.25mm/minute. The loads for 2.54mm and 5.08mm were recorded. This load is expressed as a percentage of standard load value at a respective deformation level to obtain a CBR value.

The characterization results, which have been presented in the previous chapter and the empirical relations in this chapter indicate that localized poor subgrade soil needs stabilization before its intended use as subgrade material. This can be done by mechanical stabilization such as compaction control, cut and fill with a suitable material, and reconsidering the appropriate pavement designs that consider the unsuitability of the subgrade soil during the construction phase.

When the percent pass of the subgrade materials is higher, the swell value, as well as the plasticity index, will increase. CBR and MDD values also are becoming higher for coarse-grained materials**.** As explained by seed et al (1962)**,** Swell potential and Plasticity index are linearly increasing**.** Similarly, higher values of group index & moisture content indicate the unsuitability of the materials for bearing stratum as well as construction materials. Thus, high plasticity index, liquid limits, group index values & moisture content will proof the general poor behavior of the materials. On the other hand, high dry density, CBR values, and coarse-grained texture show that the suitability of the materials for engineering uses.

Table 4.10 Standard Method of Test for CBR TP#2

Standard Method of Test for CBR: AASHTO T-193			
Sample date:	15/08/2019		
Soak date:	13/9/2019		
Test Date:	17/9/2019	Type of Material:	TP#2

Compaction Determination						
Compaction Data	65 Blows		30 Blows		10 Blows	
	Before soak	After soak	Before soak	After soak	Before soak	After soak
Mould No.	F1	F1	F2	F2	F3	F3
Mass of soil + Mould g	10865.5	11039.5	10186.7	10557.3	10118.3	10489.2
Mass Mould g	6668.8	6668.8	6595.2	6595.2	6653.5	6653.5
Mass of Soil g	4196.7	4370.7	3591.5	3962.1	3464.8	3835.7
Volume of Mould g	2060.21	2060.214	2060.21	2060.21	2060.21	2060.21
Wet density of soil g/cc	2.037	2.121	1.743	1.923	1.682	1.862
Dry density of soil g/cc	1.595	1.603	1.365	1.389	1.316	1.327

Moisture Determination												
MOISTURE CONTENT DATA	65 Blows				30 Blows				10 Blows			
	Before soak		After soak		Before soak		After soak		Before soak		After soak	
Moisture content %	28.0	27.4	31.0	33.7	27.4	27.9	37.8	39.1	27.7	27.9	40.9	39.6
Average moisture content %	27.7		32.3		27.7		38.5		27.8		40.3	

CBR Penetration Determination								
Penetration after 96 hrs Soaking Period				Surcharge Weight:4.55 KG				
65 Blows			30 Blows			10 Blows		
Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %
0.00	0.049		0.00	0.001		0.00	0.004	
0.64	0.726		0.64	0.103		0.64	0.091	
1.27	1.081		1.27	0.161		1.27	0.102	
1.91	1.182		1.91	0.209		1.91	0.116	
2.54	0.902	6.76	2.54	0.248	1.86	2.54	0.13	0.97
3.18	0.704		3.18	0.283		3.18	0.147	
3.81	0.563		3.81	0.315		3.81	0.157	
4.45	0.529		4.45	0.343		4.45	0.17	
5.08	0.515	2.58	5.08	0.365	1.83	5.08	0.183	0.92
5.72	0.495		5.72	0.381		5.72	0.198	

Modified Max.Dry Density g/cc	1.570	OMC %	28.8

Swell Determination							
Date		65 Blows		30 Blows		10 Blows	
		Gauge reading	Swell in %	Gauge reading	Swell in %	Gauge reading	Swell in %
		mm		mm		mm	
13/9/2019	Initial	18.92	5.15	20.455	4.30	23.28	1.87
17/9/2019	Final	24.92		25.46		25.46	
Dry Density at 95% of MDD			1.492				

No.of blows	MCBS %	DDBS g/cm3	Correcrt CBR %	% of Compaction
10	27.8	1.316	1.0	84
30	27.7	1.365	1.9	87
65	27.7	1.595	6.8	102

CBR % at 95 % MDD	5.3	Swell %	4.30

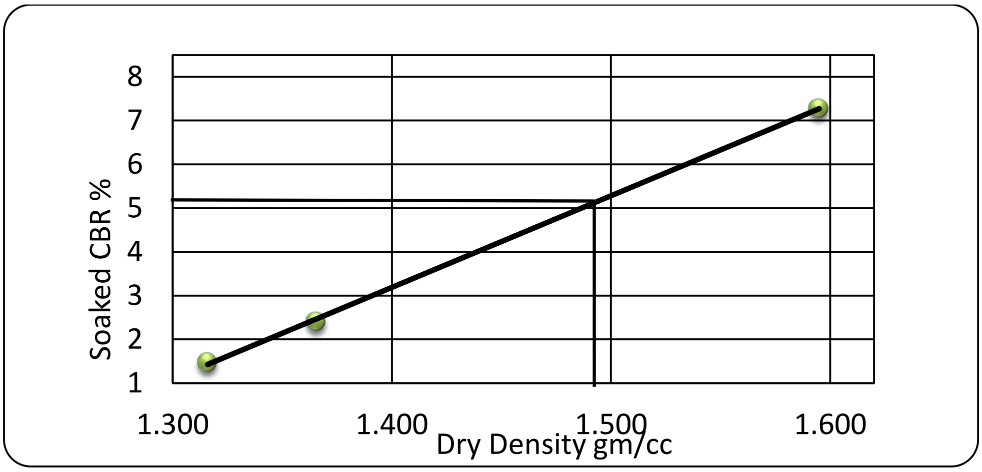

Figure 4.6 Soaked CBR Vs Dry Density for TP#2

Table 4.11 Standard Method of Test for CBR Selected Materials (QM)

Standard Method of Test for CBR: AASHTO T-193			
Sample date:	15/08/2019		
Soak date:	13/9/2019		
Test Date:	17/9/2019	Type of Material:	Selected Granular Materials (QM)

CBR Penetration Determination								
Penetration after 96 hrs Soaking Period				Surcharge Weight:4.55 KG				
65 Blows			30 Blows			10 Blows		
Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %
0.00	0.001		0.00	0.002		0.00	0.004	
0.64	0.168		0.64	0.268		0.64	0.201	
1.27	0.563		1.27	0.663		1.27	0.323	
1.91	1.331		1.91	1.431		1.91	0.516	
2.54	2.135	16.00	2.54	1.553	11.64	2.54	1.146	8.59
3.18	2.277		3.18	1.582		3.18	1.194	

3.81	2.612		3.81	1.608		3.81	1.228	
4.45	2.907		4.45	1.896		4.45	1.492	
5.08	3.091	15.46	5.08	2.083	10.42	5.08	1.575	7.88
5.72	3.407		5.72	2.453		5.72	2.067	

Modified Max.Dry Density g/cc	2.168	OMC %	14.0

Swell Determination							
Date		65 Blows		30 Blows		10 Blows	
		Gauge reading	Swell in %	Gauge reading	Swell in %	Gauge reading	Swell in %
		mm		mm		mm	
13/9/2019	Initial	24.46	0.15	19.455	0.20	24.46	0.20
17/9/2019	Final	24.64		19.69		24.69	

Dry Density at 95% of MDD:	2.059

No.of blows	MCBS %	DDBS g/cm3	Correcrt CBR %	% of Compaction
10	12.1	2.035	8.6	94
30	11.6	2.199	12.3	101
65	12.0	2.092	16.1	97

CBR % at 95 % MDD	11.6	Swell %	0.20

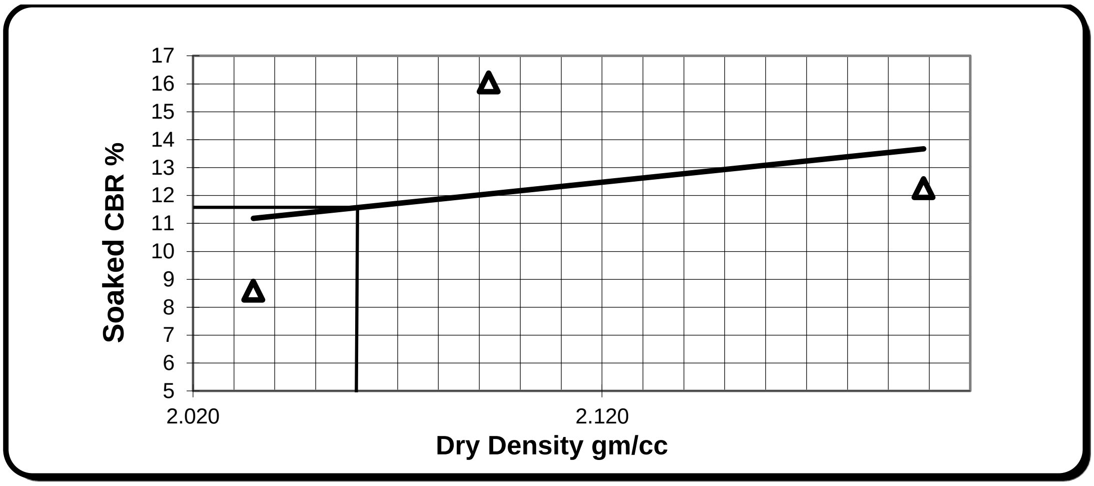

Figure 4.7 Soaked CBR Vs Dry Density for Selected Materials (QM)

Table 4.12 Density- Moisture Relationships and California Bearing Ratio for tested soil.

	TP1	TP2	QM
MDD	1.6475	1.57	2.1675
OMC	27.9	28.8	14
CBR	6.9	5.3	11.6

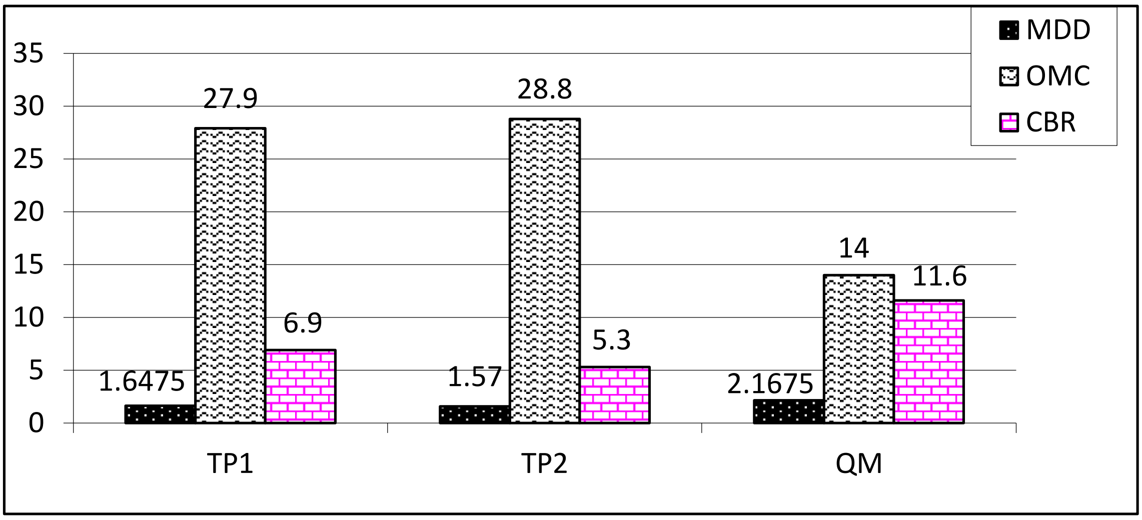

Figure 4.8 Density- Moisture Relationships and California Bearing Ratio of Tested soils

4.2 Effects of selected materials on the properties of weak subgrade soil.

The subgrade soil samples are under soils group A–7–5, the study area show GI greater than 20. This is an indication of the sections of the alignment with high PI and GI values were very poor to support the traffic load.

Tests results obtained for the study area showed that selected materials are found to be Silty or Clayey Gravel and Sand material having a PI value less than 20%. From the GI values, it can be concluded that selected materials are considered suitable soils to be used as improved layer material under pavement.

The subgrade soil were not give CBR more than 8%, then to reduce the thickness of pavement, improvement of the subgrade material is done, when blended with the selected material ranging from 50% to 85% to attain a CBR value ranging between 10% and 30% when compacted to 95% MDD modified compaction. From numerous experiments & field use, it is established that when poor soil is added to selected material, density as well as

CBR increases. While blending the soil, it should be tested in soil laboratory; PI value of the combined blended material reduced.

Therefore, Adjust the percentages of sample blend of subgrade soil from 15% to 85% of selected materials. The laboratory testing procedures checked for substantial improvement in strength with the addition of selected material percentages 55%,70%, and 85% by weight of soil.

Table 4.13 Swelling potential and plasticity index of soil in the study area.

No	Test Performed	Standard Test Method	Accepted Criteria	Test Results	Swelling potential	Remarks
TP1	Liquid Limit	AASHTO T 89	Max. 60%	79.25		unsuitable
	Plasticity Index	AASHTO T 90	Max. 30%	49.05	High	
TP2	Liquid Limit	AASHTO T 89	Max. 60%	67.5		unsuitable
	Plasticity Index	AASHTO T 90	Max. 30%	32.42	High	
TP3	Liquid Limit	AASHTO T 89	Max. 60%	60		Poor to Good
	Plasticity Index	AASHTO T 90	Max. 30%	28	Medium	
QM	Liquid Limit	AASHTO T 89	Max. 60%	37.87		suitable
	Plasticity Index	AASHTO T 90	Max. 30%	17.05	Low	suitable
S15+QM85	Liquid Limit	AASHTO T 89	Max. 60%	46.12		suitable
	Plasticity Index	AASHTO T 90	Max. 30%	23.78	Medium	
S30+QM70	Liquid Limit	AASHTO T 89	Max. 60%	50.75		suitable
	Plasticity Index	AASHTO T 90	Max. 30%	24.09	Medium	
S45+QM55	Liquid Limit	AASHTO T 89	Max. 60%	52		suitable
	Plasticity Index	AASHTO T 90	Max. 30%	25.80	Medium	

It was observed from the Atterberg limits test that the plasticity index increases with the amount of fine fraction in the soils.

The liquid limit of both soil and soil- selected granular material mixtures (passing the No. 40 sieve) was found using the Casagrande method. The liquid limit values decreased with the addition of selected material. The same trend has been shown by the plasticity limit values, though the rate was less intense than that of LL. The decrease in plasticity index values is mainly due to the decrease in LL values. The variation in Atterberg limits with the addition of selected granular material in various proportions shown in Table 4.14.

Table 4.14.The Optimum Moisture Content, Maximum Dry Density, and Plasticity Index of the Soil Samples for the Study Area.

No.	PI	MDD,g/cc	OMC,%
TP1	49.05	1.6475	27.90
TP2	32.42	1.57	28.8
Selected granular material	17.05	2.1675	14
15%Soil+85% Selected granular material	23.78	2.09	13.2
30%Soil+70% Selected granular material	24.09	1.925	16.8
45%Soil+55% Selected granular material	25.80	1.865	19.6

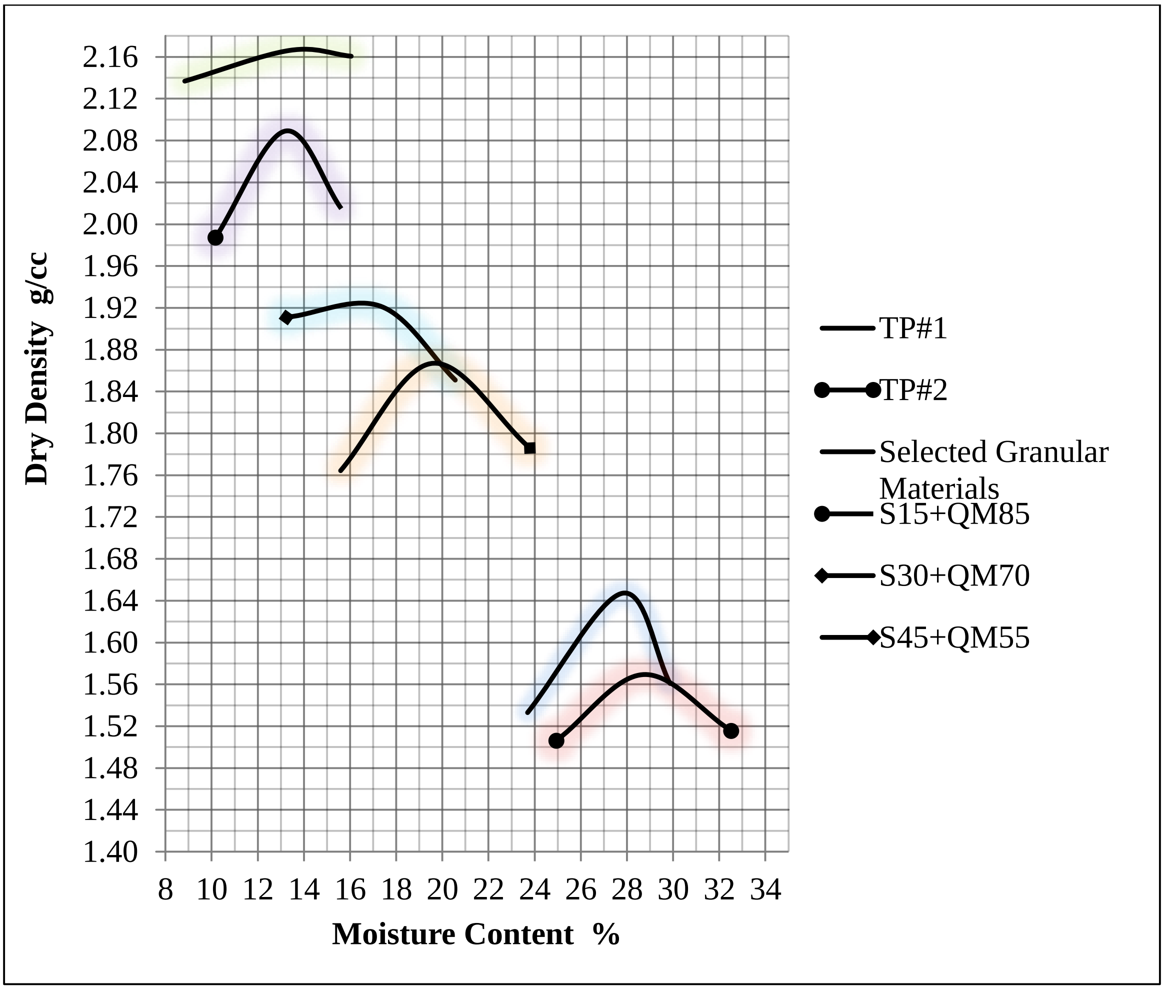

Figure 4.9 Moisture Density Relationship of the Soil Samples for the Study Area.

The values of maximum dry density for the different soil-selected granular material mixtures are presented in Table 4.14. A continuous increase of MDD has been noted mainly because of the higher specific weight of the admixture material.

In general, the compaction characteristics have been enhanced (increased maximum dry density and decreased optimum moisture content) with the addition of selected material.

Table 4.15 Density- Moisture Relationships and California Bearing Ratio for tested soil.

	TP1	TP2	QM	S15+QM85	S30+QM70	S45+QM55
MDD	1.6475	1.57	2.1675	2.09	1.925	1.865
OMC	27.9	28.8	14	13.2	16.8	19.6
CBR	6.9	5.3	11.6	19	11.3	10.3

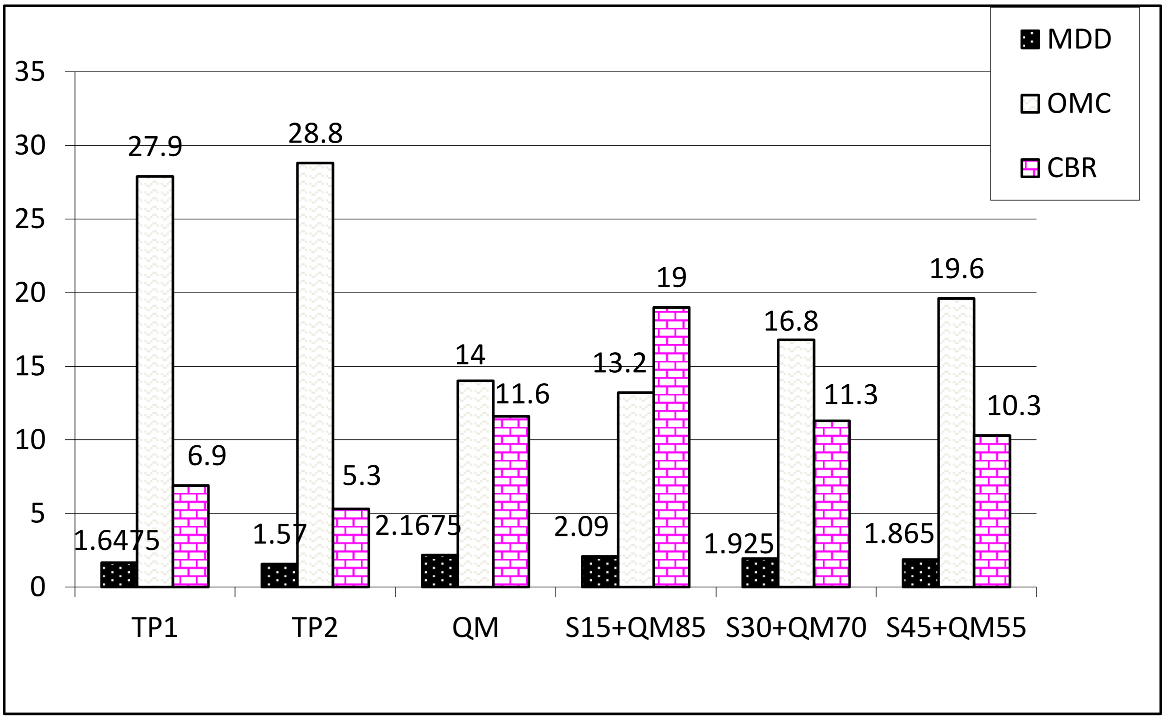

Figure 4.10 Density- Moisture Relationships and California Bearing Ratio of Tested soils

CBR value of Soil is increased as percentage of selected granular material increases. For pavement structure design, materials with a higher value of CBR, low plasticity & swell value will require less thickness for each layer & vice versa.

4.3 Suggesting the optimum blending amount and impacts on pavement performance.

For weak subgrade soils, strengthening measures are required in order to provide a strong and uniform support for the pavement and to allow road construction vehicles to pass over the subgrade without damaging the layer. This can be achieved by providing a thick layer of sub-base on the subgrade but it may be more economical to provide improved layer of selected materials. The provision of improved layer over a weak subgrade avoids the necessity of an extraordinary thick sub-base, and provides an adequate working platform for sub-base compaction as well as reduces the risk of damage to the subgrade during construction.

In this study the subgrade soil were blended with the selected material ranging from 50% to 85% and attain a CBR value ranging between 10% and 20% when compacted to 95% MDD modified compaction. When poor soil is blended with selected material, density as well as CBR increases, and PI value of the blended material reduced.

The improvement is designed to bring the existing native subgrade plus the blended layers up to an overall bearing strength level equivalent to that of the selected foundation.
Placing blended layer to improve the subgrade not only increases the bearing strength of the direct support for the pavement, but also;

- ✓ Protects the upper layers of the earthworks against adverse weather conditions.
- ✓ Facilitates the movement of construction traffic.
- ✓ Assists with obtaining good compaction of the pavement layers.
- ✓ Reduces the variation in subgrade bearing strength.
- ✓ Prevents the contamination of open-textured sub-bases by plastic fines from the natural subgrade.

The minimum thickness of each type of blended material required to improve the subgrade to a higher class. The minimum thicknesses have been calculated taking into account the respective CBR values of each class of soil, layers not exceeding 200mm to a dry density of at least 95% MDD (AASHTO T180).

CHAPTER FIVE

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusions

Based on the analysis and interpretations of the test results, the subgrade soils in the study area possess poor engineering characteristics. Since problematic soils are a worldwide problem, there is an increasing demand for techniques to improve their behavior. Countries like Ethiopia with their economies in crisis (low per capita income) or having low cement and lime production, can use the blending with selected material technique in order to enhance soils to be used as pavement improved subgrades. Based on the experimental results on weak clayey soil blended with selected material, it could be concluded that:

- Blending improve the stability of cohesive soils of low strength by adding coarse material. Well-graded soils can be compacted to high densities at the optimum moisture content, this usually found to be the effective process for improving poorly graded materials.
- The Atterberg limits changed in a decreasing mode with the selected granular material mixtures in ascending percentages. The lower PI values could be mainly attributed to the decrease in LL values. LL<60 and PI<30 satisfied.
- The optimum moisture content of soil is decreased, with increase in Percentages of selected material.
- The maximum dry density of soil is increased by addition of selected material.
- The mixing of existing soils eliminates pockets of high moisture contents.
- Blending is used to improve the native subgrade to achieve a new subgrade class or selected design subgrade/foundation class and reduce the thickness of pavement.
- The provision of improved layer avoids the necessity of an extraordinary thick sub-base, and provides an adequate working platform for sub-base compaction as well as reduces the risk of damage to the subgrade during construction.
- The construction time required for excavating problematic soils and/or hauling in additional materials will be reduced.

5.2 Recommendations

Based on the geotechnical characterization of subgrade soils, selected material, and blend of both material in general and for the subject road in particular, the above-mentioned conclusions have been made. Based on the conclusion made above, the appropriate recommendations are given.

- For subgrade where the strength/bearing capacity is marginally below the design requirements, additional compaction can be considered to increase it to the required levels as an alternative to more costly and time-intensive chemical stabilization, which may not be cost-effective for the small increase in strength that is required. The use of additional compaction to meet strength requirements should not be considered on expansive soils. Care must also be taken not to over compact the soil, which can lead to crushing of aggregate particles.
- Moisture contents need to be carefully controlled, ensuring that in situ moisture is factored into the calculation of optimum moisture content. Silt and clay soils are usually compacted at or slightly above optimum moisture content (i.e., typically one or at most two percent above optimum to reduce permeability). Silt and clay soils are, however, unworkable at higher moisture contents. Cohesionless soils are typically compacted at moisture contents close to or slightly above optimum moisture contents.
- For Nonplastic materials, Embankment and subgrade material need some cohesion to bind and fill those voids between aggregates. These materials need blending with plastic soils in order to incorporate them into the pavement structures.
- For subgrade material having CBR values at 95% of MDD are more than 15%: its value for swell less than 1.5% and plasticity index of maximum 15%: can be taken as the level of improved subgrade (capping) layer and other earthwork parts intended to construct may be omitted and the subbase layer can directly be put on the native subgrade layer.
- The minimum thicknesses taking into account a CBR values of blend of selected material percentages 70% by weight of soil, layers not exceeding 200mm to a dry density of at least 95% MDD (AASHTO T180) recommended.
- For subgrade where the blend materials need to be imported, costs will need to be compared with other methods of stabilization.

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Appendix A. AASHTO Soil Classification System (M 145)

Properties	Group Classification										
	Granular Materials (35% or less passing the 0.075 mm sieve)							Silt-Clay Materials (>35% passing the 0.075 mm sieve)			
	A-1		A-2				A-3	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, % passing 2.00 mm (#10) 0.425 mm (#40) 0.075 mm (#200)	 ≤ 50 ≤ 30 ≤ 15	 --- ≤ 50 ≤ 25	 --- --- ≤ 35	 --- --- ≤ 35	 --- --- ≤ 35	 --- --- ≤ 35	 --- ≤ 51 ≤ 10	 --- --- ≥ 36	 --- --- ≥ 36	 --- --- ≥ 36	 --- --- ≥ 36
Atterberg Limits (on 0.425 mm [#40]) Liquid limit (LL) Plasticity index (PI)	 --- ≤ 6		 ≤ 40 ≤ 10	 ≥ 41 ≤ 10	 ≤ 40 ≥ 11	 ≥ 41 ≥ 11	 --- N.P.[1]	 ≤ 40 ≤ 10	 ≥ 41 ≤ 10	 ≤ 40 ≥ 11	 ≥ 41 ≥ 11[2]
Usual types of significant constituent materials	Stone fragments, gravel and sand		Silty or clayey gravel and sand				Fine sand	Silty soils		Clayey soils	
General rating as a subgrade	Excellent to good							Fair to poor			

[1] N.P.: Non-plastic
[2] A-7-5: PI ≤ LL – 30. A-7-6: PI > LL – 30

Appendex B. Particle Size Distribution

Sieve Size	weight	%	%
(mm)	Retained	Retained	pass
50	0	0.0	100.0
37.5	0	0.0	100.0
25	0	0.0	100.0
19	0	0.0	100.0
9.5	0	0.0	100.0
4.75	1.1	0.1	99.9
2	13.8	0.9	99.0
0.425	17.9	1.2	97.8
0.15	10.2	0.7	97.1
0.075	6.7	0.4	96.7
Pan	1450.3	96.7	
Total	1500	100.0	

	Diameter of soil Particle (mm)	Test Pit#1 percentage passing
Sieve Analysis	50.000	100.00
	37.500	100.00
	25.000	100.00
	19.000	100.00
	9.500	100.00
	4.750	99.93
	2.000	99.01
	0.425	97.81
	0.150	97.13
	0.075	96.69
Hydrometer Analysis	0.029	82.50
	0.018	78.56
	0.011	72.64
	0.003	56.87
	0.001	50.96

1.Particles larger than 2mm = 1%

2.Coarse Sand 2mm - 0.425mm = 1%

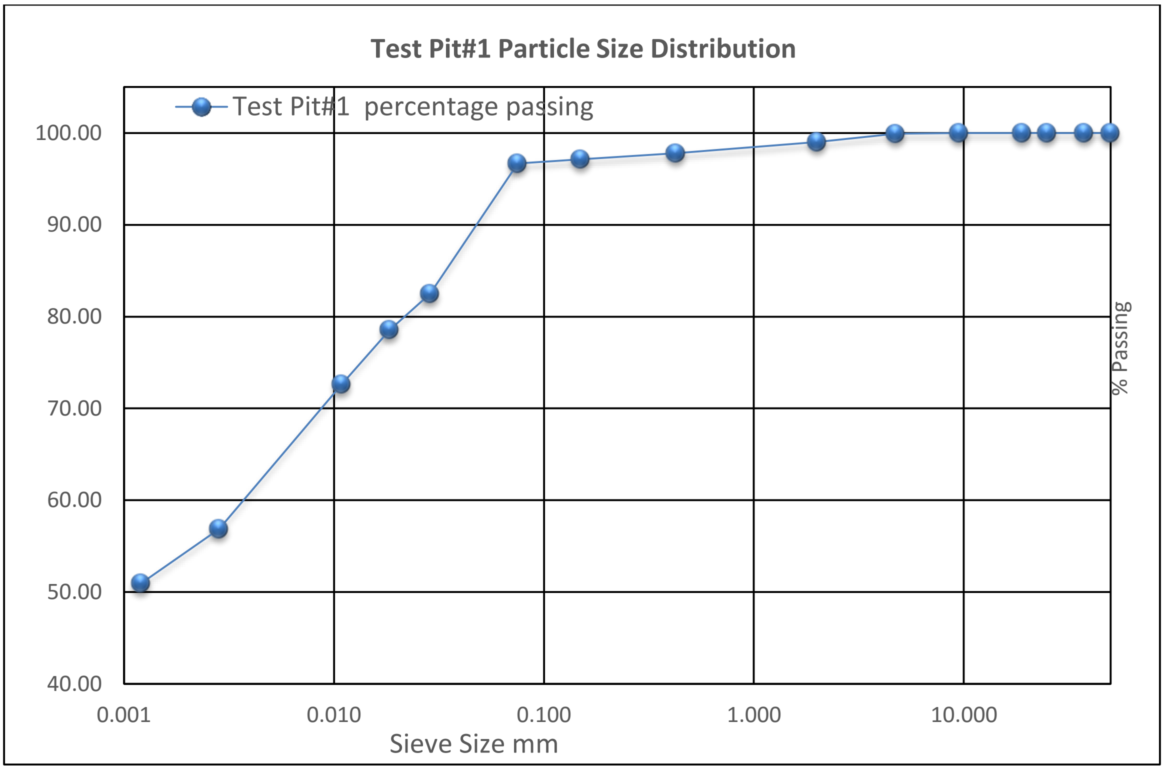
3.Fine Sand 0.425mm - 0.075mm = 1%

4.Silt 0.075-0.002mm = 44%

5.Clay smaller than 0.002mm =53%

Hydrometer analysis Data **Specific Gravity=2.72**

Time (minutes)	Hydrometer Reading	Temp.	Corrected H.Reading R'	Corrected H.Reading R"	Correction factor(a)	Effe. Depth of Hydrometer (L)	Values of K	Diameter of soil Particle(mm)	% finer, P
2	46	21	47.85	41.85	0.9846	9.40	0.01323	0.0287	82.50
5	44	21	45.85	39.85	0.9846	9.70	0.01323	0.0184	78.56
15	41	21	42.85	36.85	0.9846	10.20	0.01323	0.0109	72.64
30	40	21	41.85	35.85	0.9846	10.40	0.01323	0.0078	70.67
60	37.5	21	39.35	33.35	0.9846	10.90	0.01323	0.0056	65.74
250	33	21	34.85	28.85	0.9846	11.50	0.01323	0.0028	56.87
1440	30	21	31.85	25.85	0.9846	12.00	0.01323	0.0012	50.96



Sieve Size	weight	%	%
(mm)	Retained	Retained	pass
50	0	0.0	100.0
37.5	0	0.0	100.0
25	0	0.0	100.0
19	0	0.0	100.0
9.5	0	0.0	100.0
4.75	7.9	0.5	99.5
2	13.2	0.9	98.6
0.425	24.7	1.6	96.9
0.15	26	1.7	95.2
0.075	13.7	0.9	94.3
Pan	1414.5	94.3	
Total	1500	100.0	

Diameter of soil Particle (mm)	Test Pit#2 percentage passing
50.000	100.00
37.500	100.00
25.000	100.00
19.000	100.00
9.500	100.00
4.750	99.47
2.000	98.59
0.425	96.95
0.150	95.21
0.075	94.30
0.029	81.51
0.019	77.57
0.011	71.66
0.003	55.89
0.001	49.97

1.Particles larger than 2mm= 1%

2.Coarse Sand 2mm - 0.425mm = 2%

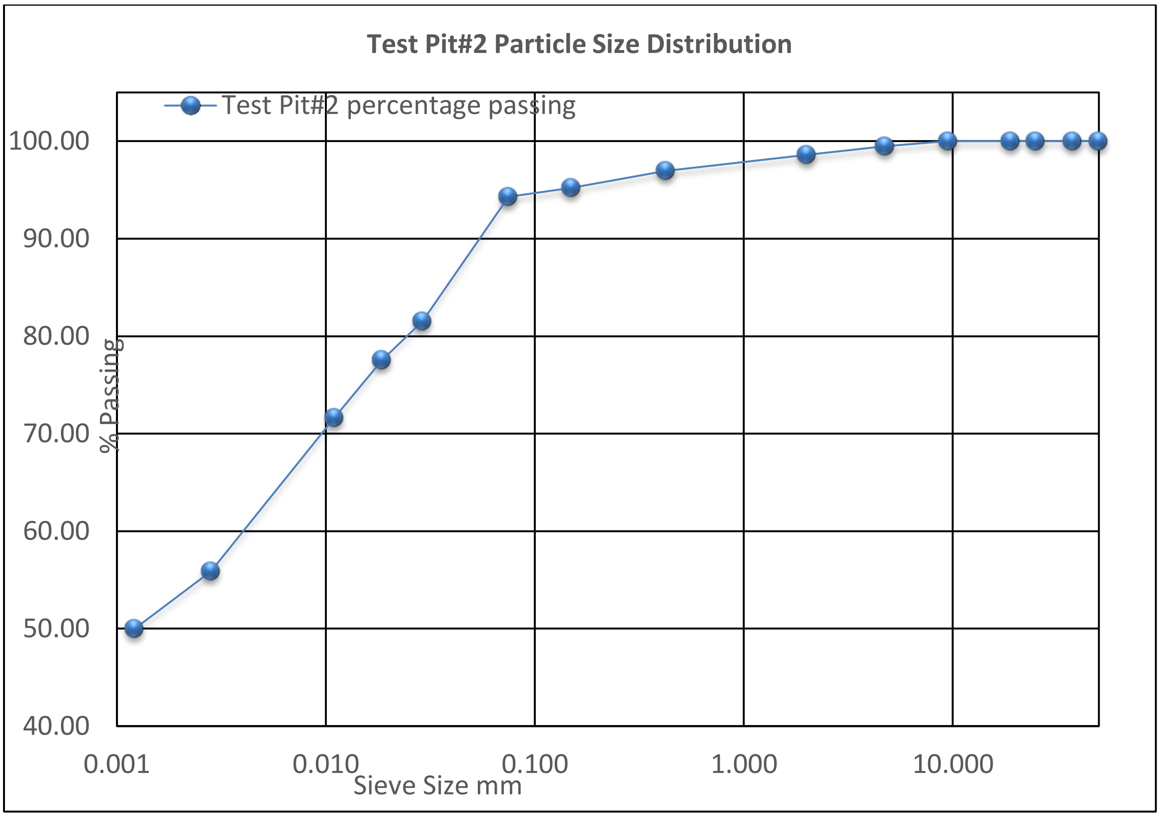
3.Fine Sand 0.425mm - 0.075mm = 3%

4.Silt 0.075-0.002mm = 41%

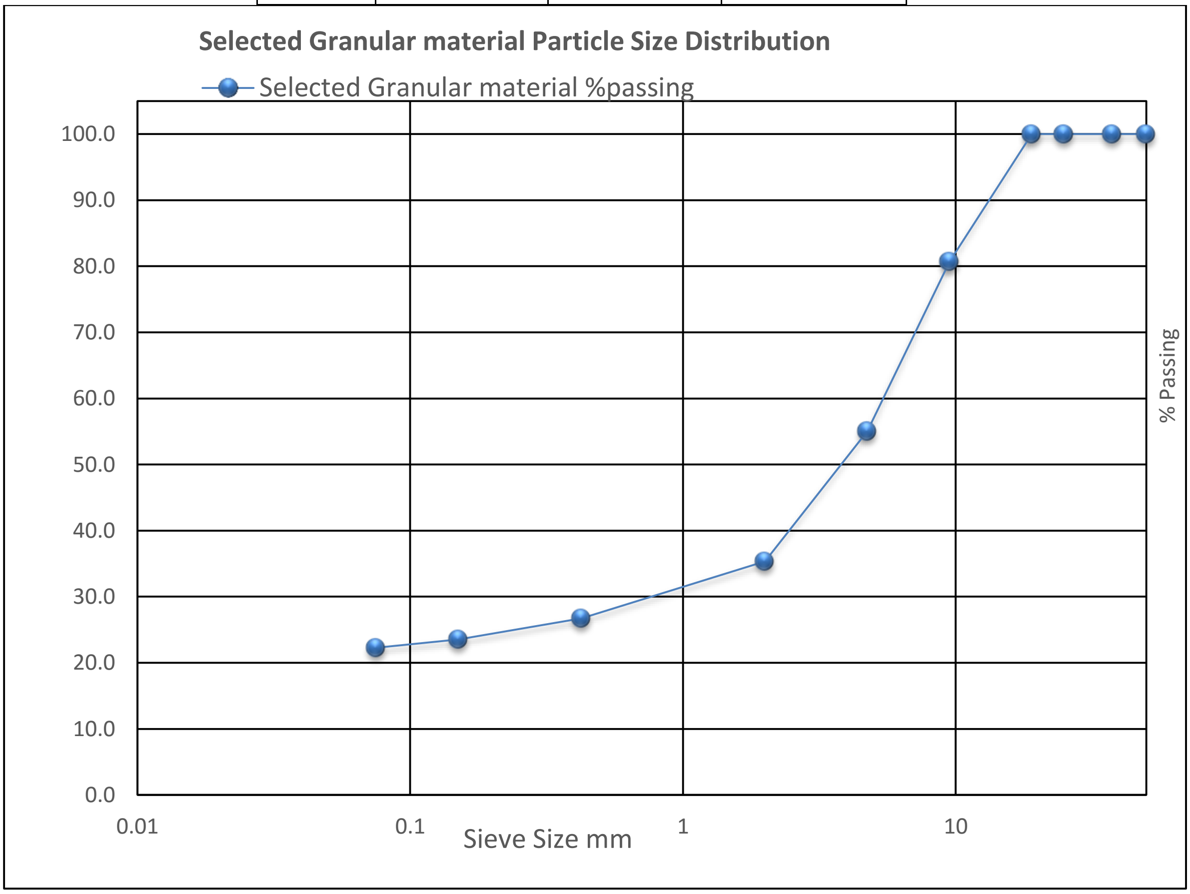
5.Clay smaller than 0.002mm = 53%

Hydrometer analysis Data **Specific Gravity= 2.72**

Time (min.)	Hydrometer Reading	Temp.	Corrected H.Reading		Correction factor(a)	Effe. Depth of Hydrometer(L)	Values of K	Diameter of soil Particle(mm)	% finer,P
			R'	R"					
2	45.5	21	47.35	41	0.9846	9.60	0.01323	0.0290	81.51
5	43.5	21	45.35	39	0.9846	9.90	0.01323	0.0186	77.57
15	40.5	21	42.35	36	0.9846	10.40	0.01323	0.0110	71.66
30	39.5	21	41.35	35	0.9846	10.60	0.01323	0.0079	69.69
60	37	21	38.85	33	0.9846	10.90	0.01323	0.0056	64.76
250	32.5	22	34.35	28	0.9846	11.70	0.01307	0.0028	55.89
1440	29.5	21	31.35	25	0.9846	12.20	0.01323	0.0012	49.97



Sieve Size (mm)	weight Retained	% Retained	Selected Granular material %passing
50	0	0.0	100.0
37.5	0	0.0	100.0
25	0	0.0	100.0
19	0	0.0	100.0
9.5	482.7	19.3	80.7
4.75	640.9	25.6	55.1
2	493.3	19.7	35.3
0.425	214.5	8.6	26.7
0.15	80.6	3.2	23.5
0.075	30.7	1.2	22.3
Pan	557.3	22.3	
Total	2500	100.0	



Appendix C. Atterberg Limit

SOIL CONSISTENCY TEST RESULT *(TEST METHOD: AASHTO T89, T90)*

Sampled and Tested by:- Ahmed Simeneh			
Material location :	*Agao Town*		
Sampling date :	*15/08/2019*	*Pit Number*	*Pit #1*
Testing date :	*05/09/2019*		
Material for :	*SUBGRADE*		

	Liquid Limit				Plastic Limit	
No. of Blows	32	24	17			
Container Number	LL-1	LL-2	LL-3		PL-1	PL-2
Wt. of Container + Wet Soil (g) = (W_1)	49.507	28.856	24.51		18.317	16.68
Wt. of Container + Dry Soil (g) = (W_2)	43.001	22.75	16.284		15.58	14.04
Wt. of Container (g) = (W_3)	34.64	14.947	6.341		6.47	5.31
Weight of Moisture (g) = (W_1 - W_2) = A	6.51	6.103	8.228		2.74	2.65
Weight of Dry Soil (g) = (W_2 - W_3) = B	8.36	7.81	9.94		9.11	8.72
Moisture Content (%) = (A / B)x 100	77.85	78.18	82.8		30.08	30.32
				AV. Plas. Lim.	30.2	

LIQUIDLIMIT	LL	79.25
PLASTIC LIMIT	PL	30.20
PLASTICITY INDEX =	LL - PL	49.05

SOIL CONS+F4+A1:I24+A1:I27+F4+A1:I24+A1:I40+F4+A1:I24+A1:I45+F4+A1:I24+A1:I45

Sampled and Tested by:- Ahmed Simeneh			
Material location	*Agao Town*		
Sampling date :	*15/08/2019*	*Pit Number*	*Pit #2*
Testing date :	*05/09/2019*		
Material for :	*SUBGRADE*		

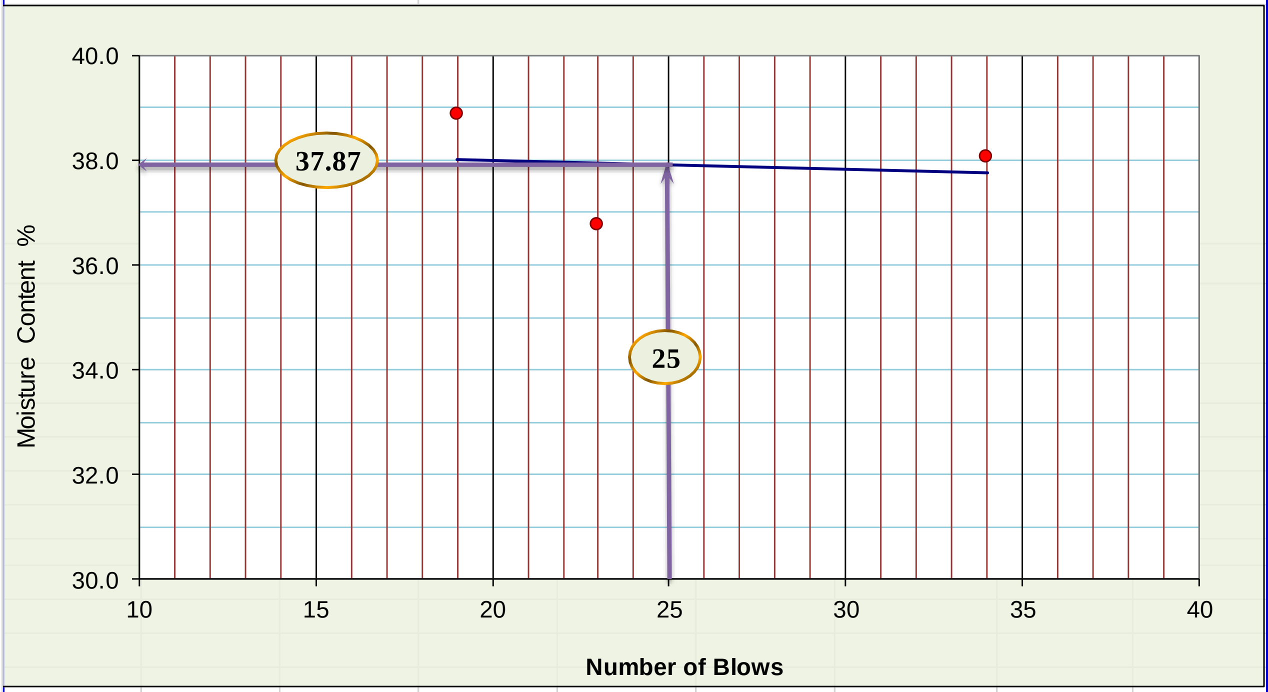
	Liquid Limit				Plastic Limit	
No. of Blows	35	24	16			
Container Number	LL-1	LL-2	LL-3		PL-1	PL-2
Wt. of Container + Wet Soil (g) = (W_1)	20.015	22.283	20.38		40.234	35.25
Wt. of Container + Dry Soil (g) = (W_2)	14.582	15.87	14.376		36.62	30.86
Wt. of Container (g) = (W_3)	6.43	6.452	5.578		26.28	18.37
Weight of Moisture (g) = $(W_1 - W_2)$ = A	5.43	6.412	6.001		3.62	4.39
Weight of Dry Soil (g) = $(W_2 - W_3)$ = B	8.15	9.42	8.80		10.33	12.49
Moisture Content (%) = (A / B)x 100	66.66	68.08	68.2		35.03	35.14
				AV. Plas. Lim.	35.1	

LIQUIDLIMIT	LL	67.5
PLASTIC LIMIT	PL	35.08
PLASTICITY INDEX =	LL - PL	32.42

SOIL CONSISTENCY TEST RESULT (TEST METHOD: AASHTO T89, T90)

Sampled and Tested by:- Ahmed Simeneh		
Material location	*Agao Town*	
Sampling date :	*15/08/2019*	*Selected Granular Materials (QM)*
Testing date :	*06/09/2019*	
Material for :	*SUBGRADE*	

	Liquid Limit				Plastic Limit	
No. of Blows	34	23	19			
Container Number	LL-1	LL-2	LL-3		PL-1	PL-2
Wt. of Container + Wet Soil (g) = (W_1)	32.67	46.855	31.95		35.968	36.44
Wt. of Container + Dry Soil (g) = (W_2)	28.515	43.15	27.963		32.85	33.18
Wt. of Container (g) = (W_3)	17.60	33.074	17.702		17.89	17.47
Weight of Moisture (g) = (W_1 - W_2) = A	4.16	3.705	3.989		3.12	3.26
Weight of Dry Soil (g) = (W_2 - W_3) = B	10.92	10.08	10.26		14.95	15.72
Moisture Content (%) = (A / B)x 100	38.07	36.77	38.9		20.89	20.75
				AV. Plas. Lim.	20.8	



LIQUIDLIMIT	LL	37.87
PLASTIC LIMIT	PL	20.82
PLASTICITY INDEX =	LL - PL	17.05

SOIL CONSISTENCY TEST RESULT (TEST METHOD: AASHTO T89, T90)

Sampled and Tested by:- Ahmed Simeneh

Material location	*Agao Town*	
Sampling date :	*15/08/2019*	*15%Soil+85%Selected Granular Materials (QM)*
Testing date :	*06/09/2019*	
Material for :	*SUBGRADE*	

	Liquid Limit				Plastic Limit	
No. of Blows	34	26	18			
Container Number	LL-1	LL-2	LL-3		PL-1	PL-2
Wt. of Container + Wet Soil (g) = (W_1)	31.039	34.991	51.15		70.598	53.32
Wt. of Container + Dry Soil (g) = (W_2)	26.906	29.53	46.829		66.77	49.72
Wt. of Container (g) = (W_3)	17.34	17.601	37.906		49.69	33.57
Weight of Moisture (g) = (W_1 - W_2) = A	4.13	5.466	4.32		3.82	3.60
Weight of Dry Soil (g) = (W_2 - W_3) = B	9.57	11.92	8.92		17.09	16.15
Moisture Content (%) = (A / B)x 100	43.20	45.84	48.4		22.38	22.30
				AV. Plas. Lim.	22.3	

LIQUID LIMIT	LL	46.12
PLASTIC LIMIT	PL	22.34
PLASTICITY INDEX =	LL - PL	23.78

SOIL CONSISTENCY TEST RESULT (TEST METHOD: AASHTO T89, T90)

Sampled and Tested by:- Ahmed Simeneh		
Material location	*Agao Town*	
Sampling date :	*15/08/2019*	*30%Soil+70%Selected Granular Materials (QM)*
Testing date :	*06/09/2019*	
Material for :	*SUBGRADE*	

	Liquid Limit			Plastic Limit	
No. of Blows	33	28	19		
Container Number	LL-1	LL-2	LL-3	PL-1	PL-2
Wt. of Container + Wet Soil (g) = (W_1)	49.454	45.496	30.16	42.264	45.25
Wt. of Container + Dry Soil (g) = (W_2)	45.755	41.27	25.604	37.03	39.49
Wt. of Container (g) = (W_3)	37.92	32.893	16.998	17.38	17.93
Weight of Moisture (g) = (W_1 - W_2) = A	3.70	4.229	4.553	5.24	5.76
Weight of Dry Soil (g) = (W_2 - W_3) = B	7.84	8.37	8.61	19.65	21.57
Moisture Content (%) = (A / B)x 100	47.19	50.50	52.9	26.64	26.68
			AV. Plas. Lim.	26.7	

LIQUID LIMIT	LL	50.75
PLASTIC LIMIT	PL	26.66
PLASTICITY INDEX =	LL - PL	24.09

SOIL CONSISTENCY TEST RESULT (TEST METHOD: AASHTO T89, T90)

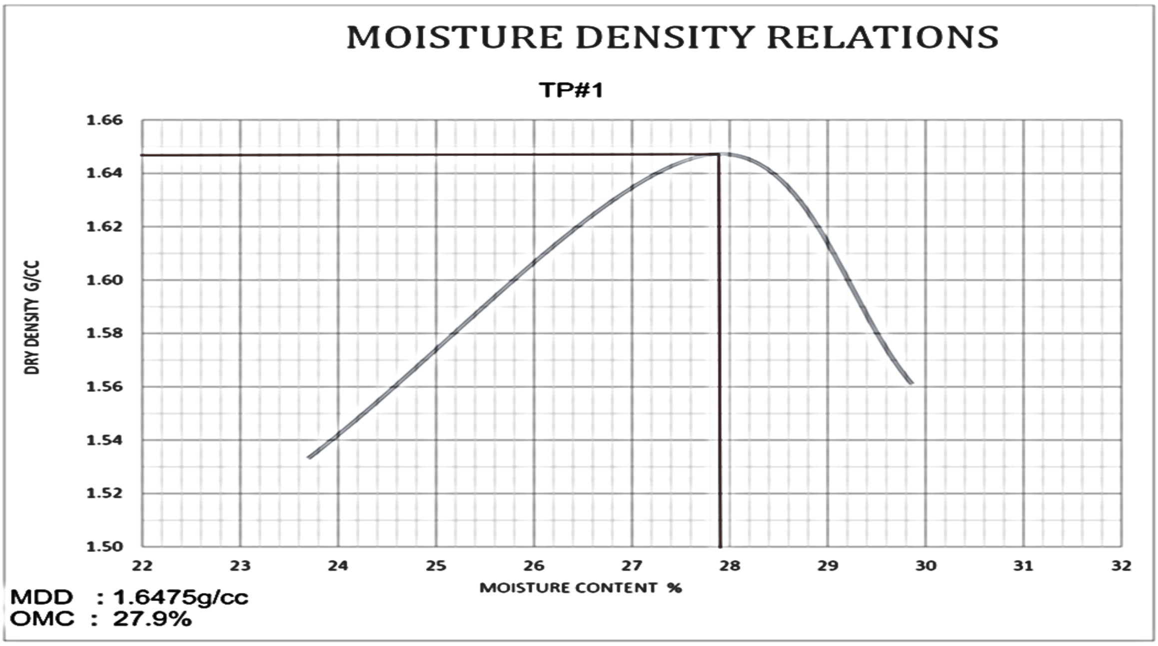
Sampled and Tested by:- Ahmed Simeneh		
Material location	*Agao Town*	
Sampling date :	*15/08/2019*	*45%Soil+55%Selected Granular Materials (QM)*
Testing date :	*06/09/2019*	
Material for :	*SUBGRADE*	

	Liquid Limit				Plastic Limit	
No. of Blows	32	24	17			
Container Number	LL-1	LL-2	LL-3		PL-1	PL-2
Wt. of Container + Wet Soil (g) = (W_1)	46.784	43.683	35.16		34.421	34.59
Wt. of Container + Dry Soil (g) = (W_2)	40.754	37.46	28.589		30.96	31.19
Wt. of Container (g) = (W_3)	28.32	25.404	17.158		17.73	18.23
Weight of Moisture (g) = (W_1 - W_2) = A	6.03	6.226	6.57		3.46	3.40
Weight of Dry Soil (g) = (W_2 - W_3) = B	12.43	12.05	11.43		13.23	12.96
Moisture Content (%) = (A / B)x 100	48.50	51.66	57.5		26.16	26.23
				AV. Plas. Lim.	26.2	

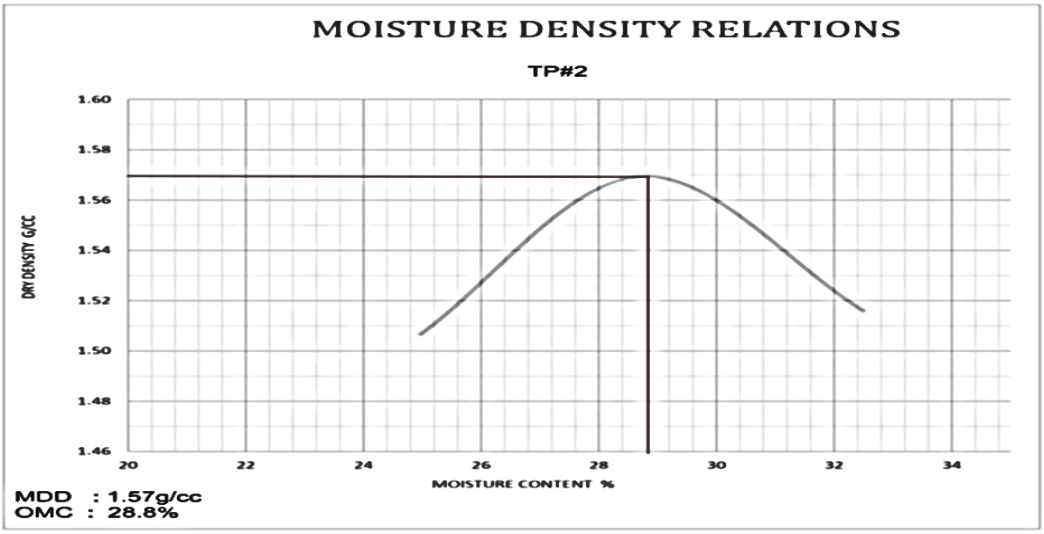
LIQUID LIMIT	LL	52
PLASTIC LIMIT	PL	26.20
PLASTICITY INDEX =	LL - PL	25.80

Appendex D. Compaction Tests

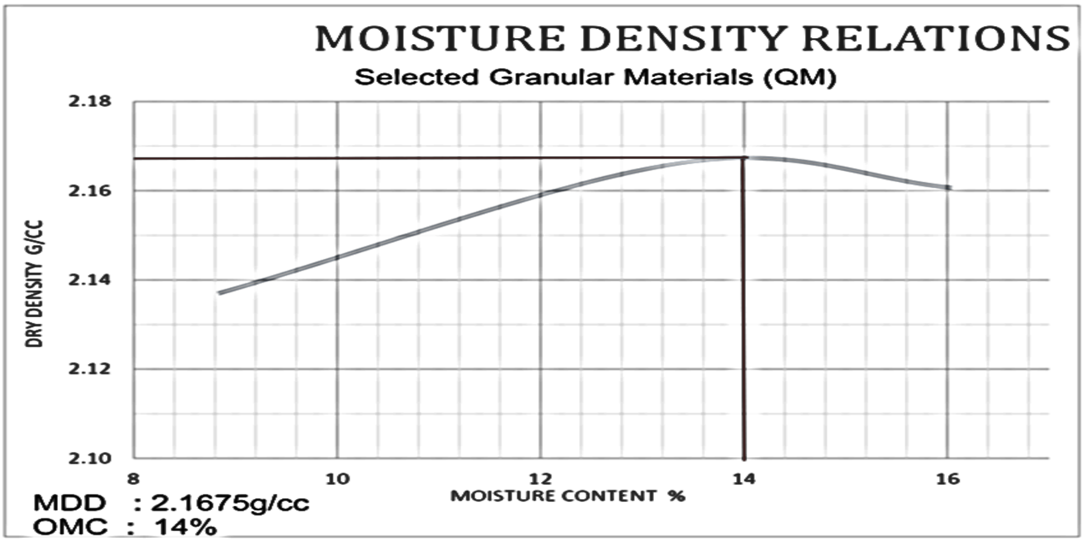
MOISTURE DENSITY RELATIONSHIP OF SOIL						
TEST METHOD: AASHTO T-180 METHOD D						
Sampled and Tested by- Ahmed Simeneh						
Material location	*Agao Town*					
Sampling date	*15/08/2019*		*Pit Number*		*Pit #1*	
Testing date	*10/09/2019*					
Material for	*SUBGRADE*					
DENSITY	TRIAL NUMBER		1	2	3	4
	WEIGHT OF SOIL + MOLD	g	10559.3	10988.9	10828.6	10702
	WEIGHT OF MOLD	g	6,653	6,653	6,653	6,653
	WEIGHT OF SOIL	g	3906.8	4336.4	4,176	4,050
	VOLUME OF MOLD	cc	2060.214	2060.214	2060.214	2060.214
	WET DENSITY OF SOIL	g/cc	1.90	2.10	2.03	1.97
MOISTURE	CONTAINER NUMBER		ZE	SG3	NB	P1
	WET SOIL + CONTAINER	g	108.0875	85.2115	63.628	86.681
	DRY SOIL + CONTAINER	g	93.87	71.364	53.057	68.991
	WEIGHT OF WATER	g	14.2175	13.8475	10.571	17.69
	WEIGHT OF CONTAINER	g	33.9	21.5	17.66	17.7705
	WEIGHT OF DRY SOIL	g	60.0105	49.815	35.397	51.2205
	MOISTURE CONTENT	**%**	23.69	27.80	29.86	34.54
DRY DENSITY OF SOIL		**g/cc**	1.53	1.65	1.56	1.46



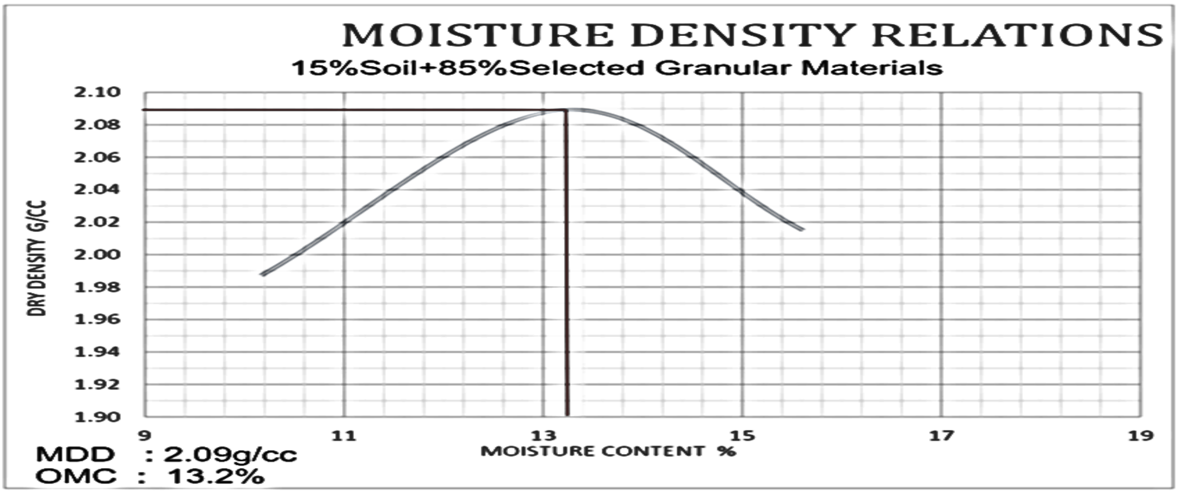
MOISTURE DENSITY RELATIONSHIP OF SOIL						
TEST METHOD: AASHTO T-180 METHOD D						
Sampled and Tested by- Ahmed Simeneh						
Material location	*Agao Town*					
Sampling date	*15/08/2019*		*Pit Number*		*Pit #2*	
Testing date	*10/09/2019*					
Material for	*SUBGRADE*					
DENSITY	TRIAL NUMBER		1	2	3	4
	WEIGHT OF SOIL + MOLD	g	10452.3	10736.9	10713	10660.4
	WEIGHT OF MOLD	g	6,575	6,575	6,575	6,575
	WEIGHT OF SOIL	g	3,877	4161.8	4,138	4085.3
	VOLUME OF MOLD	cc	2060.214	2060.214	2060.214	2060.214
	WET DENSITY OF SOIL	g/cc	1.88	2.02	2.01	1.98
MOISTURE	CONTAINER NUMBER		4=22	SS B1	6=12	10G
	WET SOIL + CONTAINER	g	84.709	93.114	104.495	134.428
	DRY SOIL + CONTAINER	g	71.321	76.249	83.118	102.365
	WEIGHT OF WATER	g	13.388	16.865	21.377	32.063
	WEIGHT OF CONTAINER	g	17.653	17.5	17.4	17.709
	WEIGHT OF DRY SOIL	g	53.668	58.731	65.731	84.656
	MOISTURE CONTENT	**%**	24.95	28.72	32.52	37.87
DRY DENSITY OF SOIL		**g/cc**	1.51	1.57	1.52	1.44



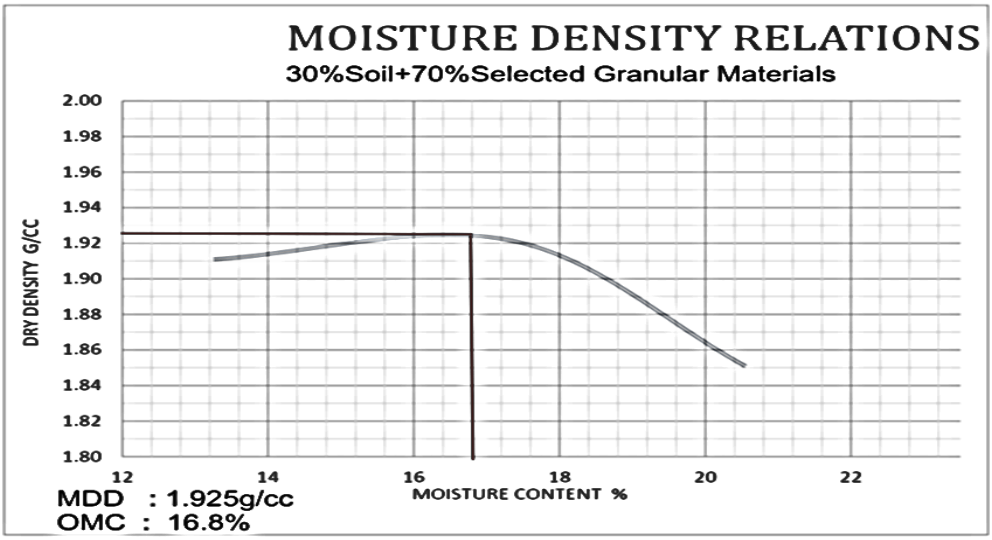
MOISTURE DENSITY RELATIONSHIP OF SOIL					
TEST METHOD: AASHTO T-180 METHOD D					
Sampled and Tested by- Ahmed Simeneh					
Material location	*Agao Town*				
Sampling date	*15/08/2019*	*Selected Granular Materials (QM)*			
Testing date	*10/09/2019*				
Material for	*SUBGRADE*				
DENSITY	TRIAL NUMBER	1	2	3	4
	WEIGHT OF SOIL + MOLD g	11340.2	11608.8	11714.6	11342.6
	WEIGHT OF MOLD g	6,549	6,549	6,549	6,549
	WEIGHT OF SOIL g	4791.4	5060	5165.8	4793.8
	VOLUME OF MOLD cc	2060.214	2060.214	2060.214	2060.214
	WET DENSITY OF SOIL g/cc	2.33	2.46	2.51	2.33
MOISTURE	CONTAINER NUMBER	G7	P3	N	M
	WET SOIL + CONTAINER g	110.15	178.269	242.186	245.357
	DRY SOIL + CONTAINER g	102.623	161.48	211.211	215.221
	WEIGHT OF WATER g	7.527	16.789	30.975	30.136
	WEIGHT OF CONTAINER g	17.389	36.0	18.2	18.031
	WEIGHT OF DRY SOIL g	85.234	125.485	193.018	197.19
	MOISTURE CONTENT %	8.83	13.38	16.05	15.28
DRY DENSITY OF SOIL	**g/cc**	2.14	2.17	2.16	2.02



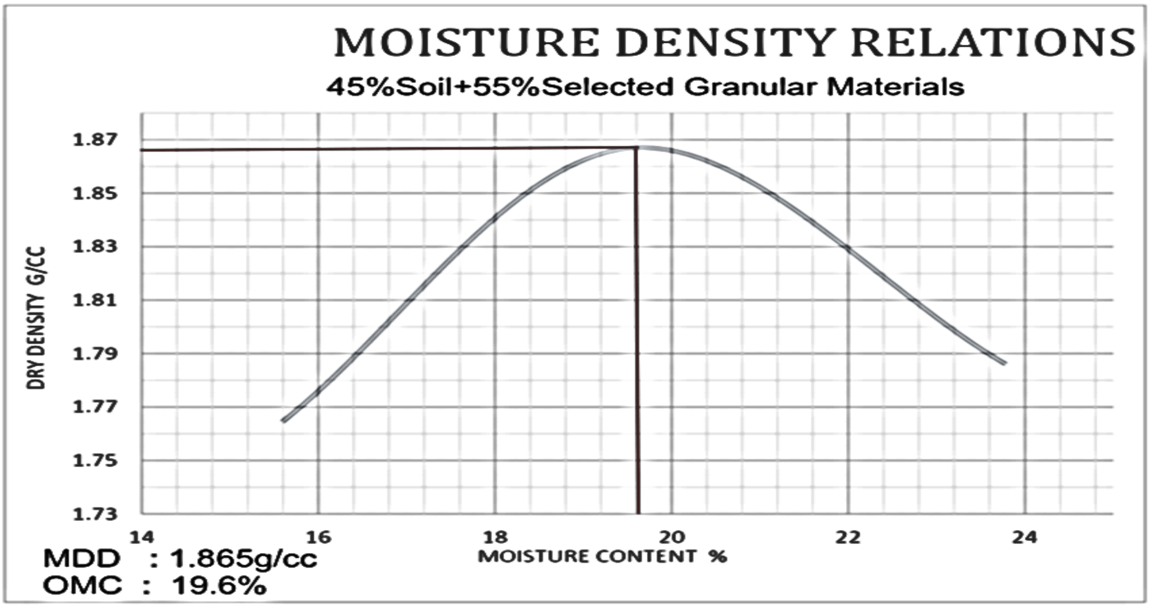
Moisture Density Relationship of Soil						
TEST METHOD: AASHTO T-180 METHOD D						
Sampled and Tested by- Ahmed Simeneh						
Material location	Agao Town					
Sampling date	15/08/2019		15%Soil+85%Selected Granular Materials (QM)			
Testing date	10/09/2019					
Material for	SUBGRADE					
DENSITY	TRIAL NUMBER		1	2	3	4
	WEIGHT OF SOIL + MOLD	g	11085.2	11446.1	11374.5	11375.8
	WEIGHT OF MOLD	g	6,575	6,575	6,575	6,575
	WEIGHT OF SOIL	g	4510.1	4871	4799.4	4800.7
	VOLUME OF MOLD	cc	2060.214	2060.214	2060.214	2060.214
	WET DENSITY OF SOIL	g/cc	2.19	2.36	2.33	2.33
MOISTURE	CONTAINER NUMBER		14	G19	15P3	2=2
	WET SOIL + CONTAINER	g	130.379	178.639	170.707	196.538
	DRY SOIL + CONTAINER	g	119.957	161.829	151.054	170.369
	WEIGHT OF WATER	g	10.422	16.81	19.653	26.169
	WEIGHT OF CONTAINER	g	17.423	34.3	25.2	28.747
	WEIGHT OF DRY SOIL	g	102.534	127.562	125.843	141.622
	MOISTURE CONTENT	%	10.16	13.18	15.62	18.48
DRY DENSITY OF SOIL		g/cc	1.99	2.09	2.01	1.97

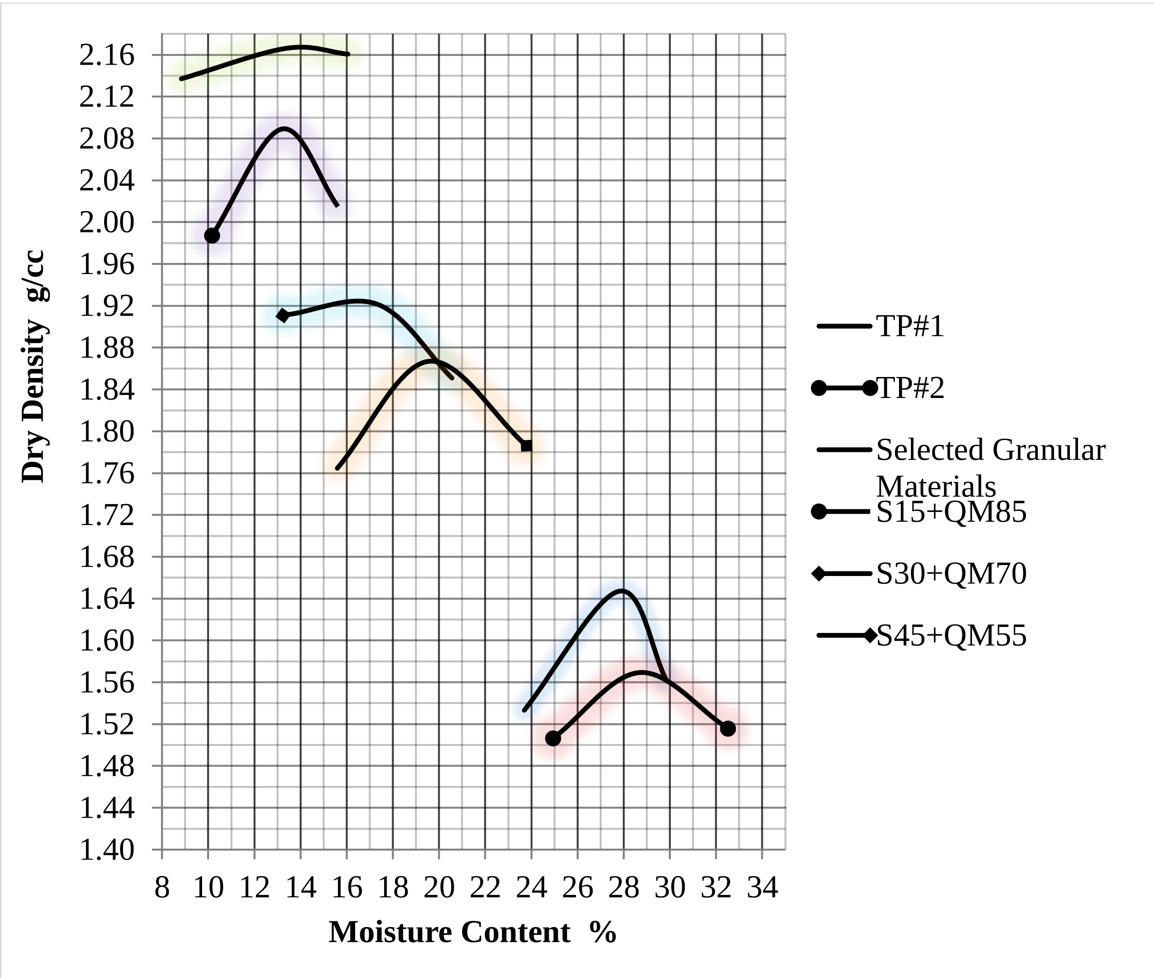


MOISTURE DENSITY RELATIONSHIP OF SOIL						
TEST METHOD: AASHTO T-180 METHOD D						
Sampled and Tested by- Ahmed Simeneh						
Material location	*Agao Town*					
Sampling date	*15/08/2019*		*30%Soil+70%Selected Granular Materials (QM)*			
Testing date	*10/09/2019*					
Material for	*SUBGRADE*					
DENSITY	TRIAL NUMBER		1	2	3	4
	WEIGHT OF SOIL + MOLD	g	11033.5	11220.04	11172	11170.4
	WEIGHT OF MOLD	g	6,575	6,575	6,575	6,575
	WEIGHT OF SOIL	g	4,458	4644.94	4596.9	4595.3
	VOLUME OF MOLD	cc	2060.214	2060.214	2060.214	2060.214
	WET DENSITY OF SOIL	g/cc	2.16	2.25	2.23	2.23
MOISTURE	CONTAINER NUMBER		AT	CS3	II	2
	WET SOIL + CONTAINER	g	78.332	132.372	115.861	130.2805
	DRY SOIL + CONTAINER	g	71.218	117.9585	99.1495	110.5655
	WEIGHT OF WATER	g	7.114	14.4135	16.7115	19.715
	WEIGHT OF CONTAINER	g	17.526	34.8	17.8	26.1785
	WEIGHT OF DRY SOIL	g	53.692	83.1565	81.3075	84.387
	MOISTURE CONTENT	**%**	13.25	17.33	20.55	23.36
DRY DENSITY OF SOIL		**g/cc**	1.91	1.92	1.85	1.81



MOISTURE DENSITY RELATIONSHIP OF SOIL					
TEST METHOD: AASHTO T-180 METHOD D					
Sampled and Tested by- Ahmed Simeneh					
Material location	*Agao Town*				
Sampling date	*15/08/2019*	*45%Soil+55%Selected Granular Materials (QM)*			
Testing date	*10/09/2019*				
Material for	*SUBGRADE*				
DENSITY	TRIAL NUMBER	1	2	3	4
	WEIGHT OF SOIL + MOLD g	10777.1	11173.2	11129.9	10611.6
	WEIGHT OF MOLD g	6,575	6,575	6,575	6,575
	WEIGHT OF SOIL g	4202	4598.1	4554.8	4036.5
	VOLUME OF MOLD cc	2060.214	2060.214	2060.214	2060.214
	WET DENSITY OF SOIL g/cc	2.04	2.23	2.21	1.96
MOISTURE	CONTAINER NUMBER	C2	36=3	3	G10
	WET SOIL + CONTAINER g	135.673	118.513	157.646	122.62
	DRY SOIL + CONTAINER g	119.743	101.938	130.738	113.253
	WEIGHT OF WATER g	15.93	16.575	26.908	9.367
	WEIGHT OF CONTAINER g	17.6	17.1	17.611	23.253
	WEIGHT OF DRY SOIL g	102.2	84.801	113.127	90
	MOISTURE CONTENT %	15.59	19.55	23.79	10.41
DRY DENSITY OF SOIL g/cc		1.76	1.87	1.79	1.77


MOISTURE DENSITY RELATIONS
45%Soil+55%Selected Granular Materials
DRY DENSITY G/CC
1.87
1.85
1.83
1.81
1.79
1.77
1.75
1.73
14 16 18 20 22 24
MOISTURE CONTENT %
MDD : 1.865g/cc
OMC : 19.6%


Dry Density g/cc
2.16
2.12
2.08
2.04
2.00
1.96
1.92
1.88
1.84
1.80
1.76
1.72
1.68
1.64
1.60
1.56
1.52
1.48
1.44
1.40
8 10 12 14 16 18 20 22 24 26 28 30 32 34
Moisture Content %
TP#1
TP#2
Selected Granular Materials
S15+QM85
S30+QM70
S45+QM55

Appendix E. California Bearing Ratio (CBR) Tests

Standard Method of Test for CBR: AASHTO T-193			
Sample date:	15/08/2019		
Soak date:	*13/9/2019*		
Test Date:	17/9/2019	Type of Material:	TP#1

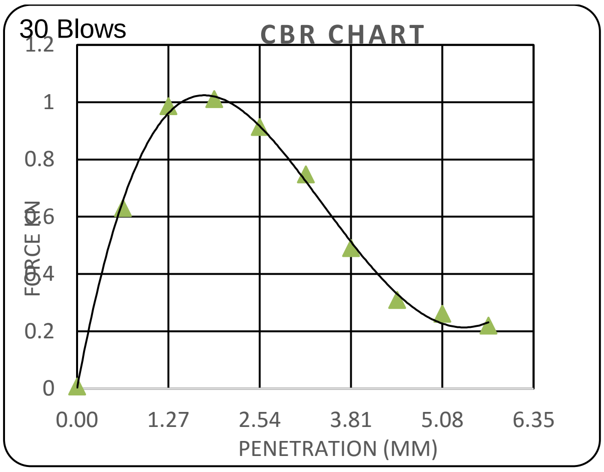
COMPACTION DATA		65 Blows		30 Blows		10 Blows	
		Before soak	After soak	Before soak	After soak	Before soak	After soak
Mould No.		CB1	CB1	CB2	CB2	CB3	CB3
Mass of soil + Mould	g	10693.8	10974.8	10468.4	10783.7	10053.9	10478.7
Mass Mould	g	6609.5	6609.5	6571.1	6571.1	6622.2	6622.2
Mass of Soil	g	4084.3	4365.3	3897.3	4212.6	3431.7	3856.5
Volume of Mould	g	2060.21	2060.21	2060.21	2060.21	2060.21	2060.21
Wet density of soil	g/cc	1.982	2.119	1.892	2.045	1.666	1.872
Dry density of soil	g/cc	1.570	1.573	1.498	1.521	1.316	1.334

Moisture Determination												
MOISTURE CONTENT DATA	65 Blows				30 Blows				10 Blows			
	Before soak		After soak		Before soak		After soak		Before soak		After soak	
Container no.	14	3	P15	F	PH	F	A13	ZE	P3	ZF	14	1A
Mass of wet soil + Container g	103.8	109.3	187.8	253.6	155.2	131.4	199.0	258.4	143.4	173.9	171.0	152.8
Mass of dry soil + Container g	86.0	90.1	148.5	197.1	129.8	111.7	157.3	200.8	121.0	144.2	126.4	114.5
Mass of container g	17.4	17.6	33.6	36.4	33.6	36.4	36.6	33.1	36.0	33.1	17.4	17.7
Mass of water g	17.8	19.2	39.4	56.5	25.4	19.7	41.7	57.6	22.4	29.8	44.6	38.4
Mass of drysoil g	68.5	72.4	114.9	160.7	96.2	75.3	120.7	167.7	85.0	111.0	109.0	96.7
Moisture content %	26.0	26.5	34.3	35.2	26.4	26.2	34.6	34.3	26.3	26.8	40.9	39.7
Average moisture content %	26.3		34.7		26.3		34.4		26.6		40.3	

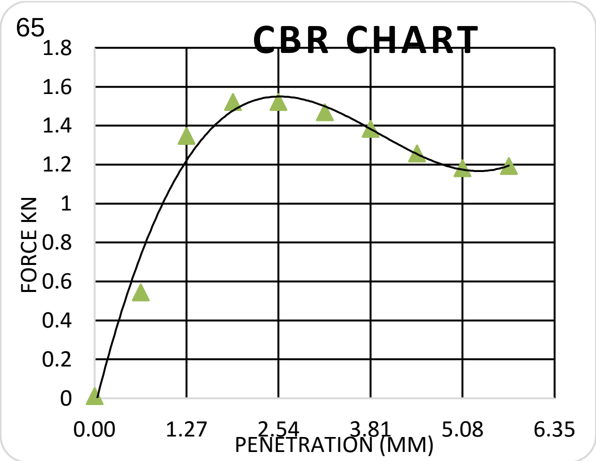
CBR Penetration Determination								
Penetration after 96 hrs Soaking Period			Surcharge Weight: 4.55 KG					
65 Blows			30 Blows			10 Blows		
Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %
0.00	0.012		0.00	0.008		0.00	0.001	

0.64	0.144		0.64	0.431		0.64	0.149	
1.27	0.848		1.27	0.786		1.27	0.167	
1.91	0.922		1.91	0.711		1.91	0.18	
2.54	0.922	6.91	2.54	0.614	4.60	2.54	0.192	1.44
3.18	1.069		3.18	0.548		3.18	0.201	
3.81	0.885		3.81	0.491		3.81	0.208	
4.45	0.757		4.45	0.31		4.45	0.213	
5.08	0.682	3.41	5.08	0.261	1.31	5.08	0.218	1.09
5.72	0.694		5.72	0.22		5.72	0.223	

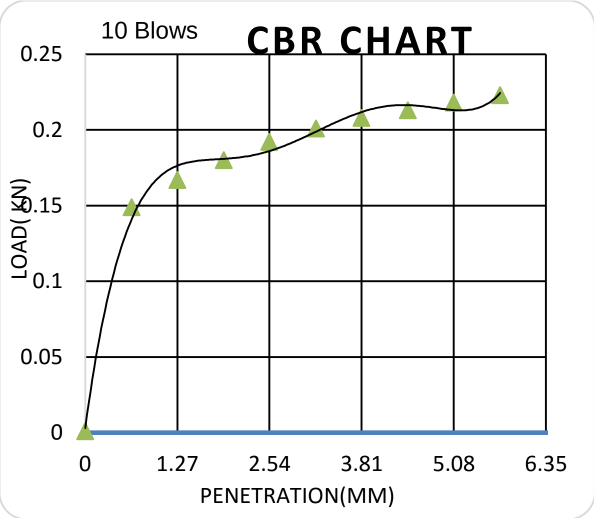
Modified Max.Dry Density g/cc		1.648	OMC %	27.9			
Swell Determination							
Date		65 Blows		30 Blows		10 Blows	
		Gauge rdg	Swell in %	Gauge rdg	Swell in %	Gauge rdg	Swell in %
		mm		mm		mm	
13/9/2019	Initial	19.42	4.73	20.455	4.30	20.46	4.30
17/9/2019	Final	24.92		25.46		25.46	



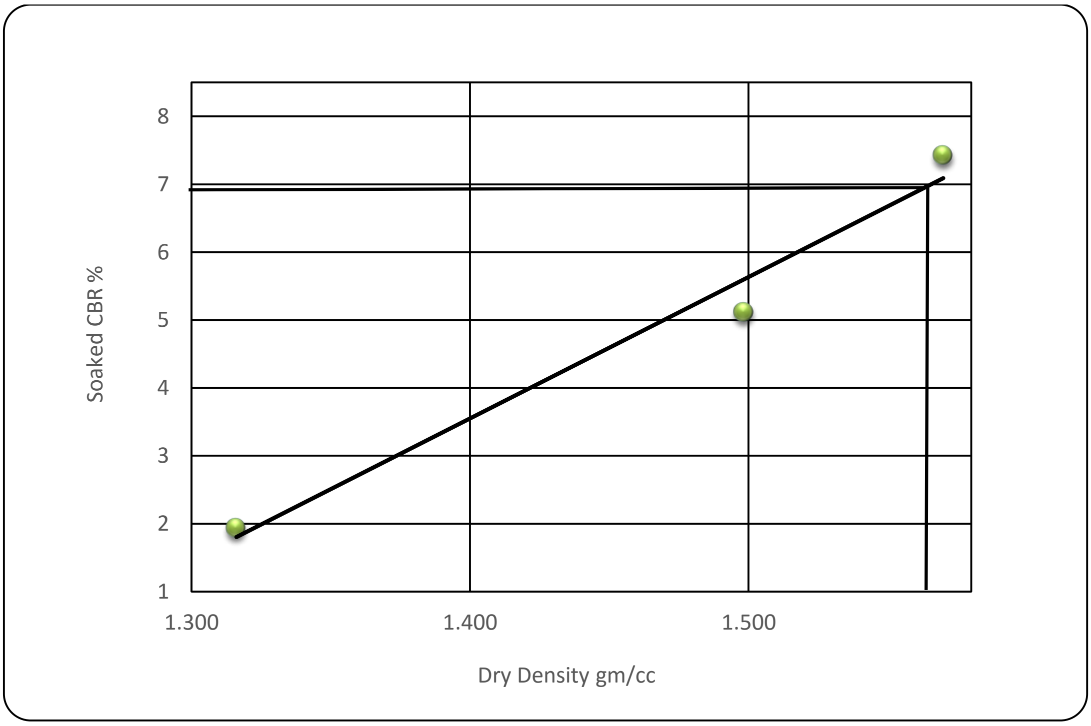
Penetration (mm)	Load KN		Corr. CBR %	Swell %
	Top	Bottom		
2.54mm		0.9	6.9	4.73
5.08mm		0.7	3.4	



Penetration (mm)	Load KN		Corr. CBR %	Swell %
	Top	Bottom		
2.54mm		0.6	4.6	4.30
5.08mm		0.2	1.1	



Penetration (mm)	Load KN		Corr. CBR %	Swell %
	Top	Bottom		
2.54mm		0.2	1.4	4.30
5.08mm		0.2	1.1	



Dry Density at 95% of MDD: 1.565

No.of blows	MCBS %	DDBS g/cm3	Correcrt CBR %	% of Compaction
10	26.6	1.316	1.4	80
30	26.3	1.498	4.6	91
65	26.3	1.570	6.9	95

CBR % at 95 % MDD	6.9	Swell %	4.30

Standard Method of Test for CBR: AASHTO T-193			
Sample date:	15/08/2019		
Soak date:	13/9/2019		
Test Date:	17/9/2019	Type of Material:	TP#2

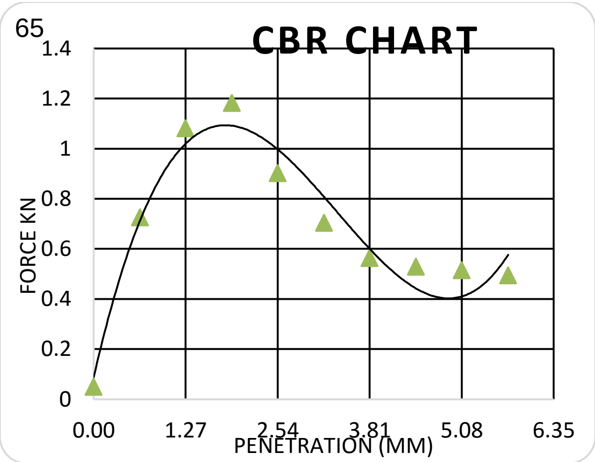
Compaction Determination						
COMPACTION DATA	65 Blows		30 Blows		10 Blows	
	Before soak	After soak	Before soak	After soak	Before soak	After soak
Mould No.	F1	F1	F2	F2	F3	F3
Mass of soil + Mould g	10865.5	11039.5	10186.7	10557.3	10118.3	10489.2
Mass Mould g	6668.8	6668.8	6595.2	6595.2	6653.5	6653.5
Mass of Soil g	4196.7	4370.7	3591.5	3962.1	3464.8	3835.7
Volume of Mould g	2060.21	2060.214	2060.21	2060.21	2060.21	2060.21
Wet density of soil g/cc	2.037	2.121	1.743	1.923	1.682	1.862
Dry density of soil g/cc	1.595	1.603	1.365	1.389	1.316	1.327

Moisture Determination												
MOISTURE CONTENT DATA	65 Blows				30 Blows				10 Blows			
	Before soak		After soak		Before soak		After soak		Before soak		After soak	
Container no.	9	A16	DH	AT	G7	HC12	SB	B3	A13	C2	36-3	G21
Mass of wet soil + Container g	145.7	161.1	96.7	163.2	101.5	106.3	141.8	181.1	143.5	98.4	132.8	134.7
Mass of dry soil + Container g	120.9	133.5	77.8	126.5	83.4	87.0	107.9	135.1	120.3	80.8	99.3	101.6
Mass of container g	32.4	32.9	17.0	17.6	17.4	18.1	18.4	17.4	36.6	17.6	17.2	18.0
Mass of water g	24.8	27.6	18.8	36.7	18.1	19.2	33.9	46.0	23.2	17.6	33.6	33.1
Mass of drysoil g	88.5	100.6	60.8	108.9	66.0	68.9	89.5	117.7	83.7	63.2	82.1	83.6
Moisture content %	28.0	27.4	31.0	33.7	27.4	27.9	37.8	39.1	27.7	27.9	40.9	39.6
Average moisture content %	27.7		32.3		27.7		38.5		27.8		40.3	

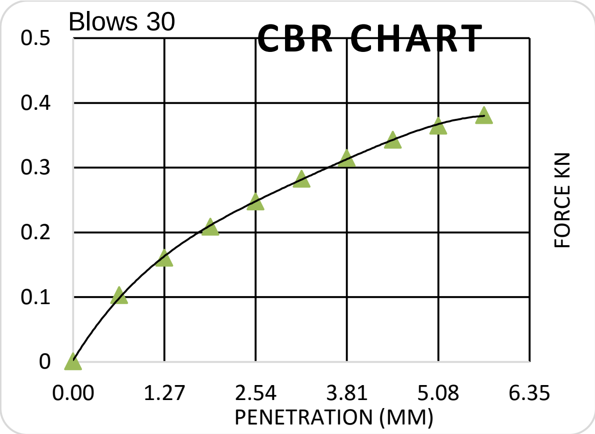
CBR Penetration Determination								
Penetration after 96 hrs Soaking Period				Surcharge Weight:4.55 KG				
65 Blows			30 Blows			10 Blows		
Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %
0.00	0.049		0.00	0.001		0.00	0.004	
0.64	0.726		0.64	0.103		0.64	0.091	
1.27	1.081		1.27	0.161		1.27	0.102	
1.91	1.182		1.91	0.209		1.91	0.116	
2.54	0.902	6.76	2.54	0.248	1.86	2.54	0.13	0.97
3.18	0.704		3.18	0.283		3.18	0.147	
3.81	0.563		3.81	0.315		3.81	0.157	
4.45	0.529		4.45	0.343		4.45	0.17	
5.08	0.515	2.58	5.08	0.365	1.83	5.08	0.183	0.92
5.72	0.495		5.72	0.381		5.72	0.198	

Modified Max.Dry Density g/cc	1.570	OMC %	28.8

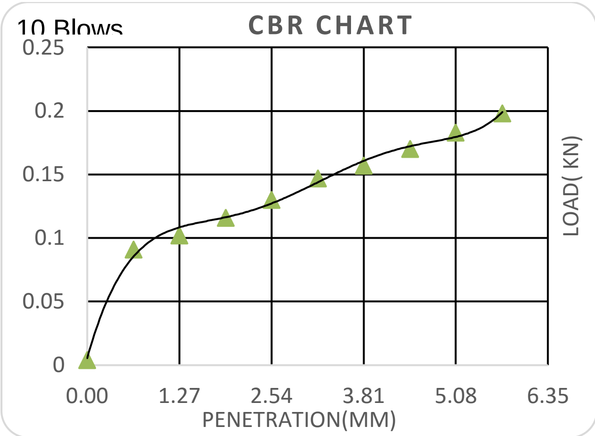
Swell Determination							
Date		65 Blows		30 Blows		10 Blows	
		Gauge reading mm	Swell in %	Gauge reading mm	Swell in %	Gauge reading mm	Swell in %
13/9/2019	Initial	18.92	5.15	20.455	4.30	23.28	1.87
17/9/2019	Final	24.92		25.46		25.46	



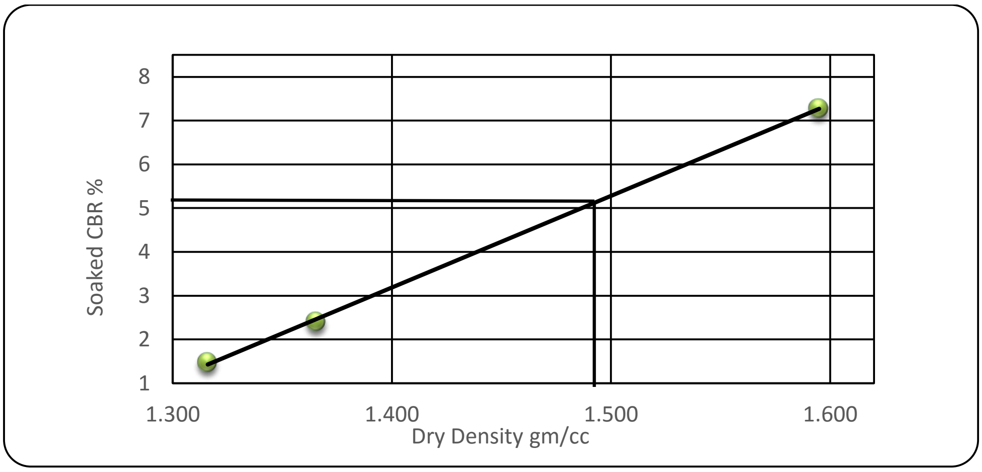
Penetration (mm)	Load KN		Corr. CBR %	Swell %
	Top	Bottom		
2.54mm		0.9	6.8	5.15
5.08mm		0.5	2.6	



Penetration (mm)	Load KN		Corr. CBR %	Swell %
	Top	Bottom		
2.54mm		0.2	1.9	4.30
5.08mm		0.4	1.9	



Penetration (mm)	Load KN		Corr. CBR %	Swell %
	Top	Bottom		
2.54mm		0.1	1.0	1.87
5.08mm		0.2	0.9	



Dry Density at 95% of MDD	1.492

No.of blows	MCBS %	DDBS g/cm3	Correcrt CBR %	% of Compaction
10	27.8	1.316	1.0	84
30	27.7	1.365	1.9	87
65	27.7	1.595	6.8	102

CBR % at 95 % MDD	5.3	Swell %	4.30

Standard Method of Test for CBR: AASHTO T-193			
Sample date:	15/08/2019		
Soak date:	13/9/2019		
Test Date:	17/9/2019	Type of Material:	Selected Granular Materials (QM)

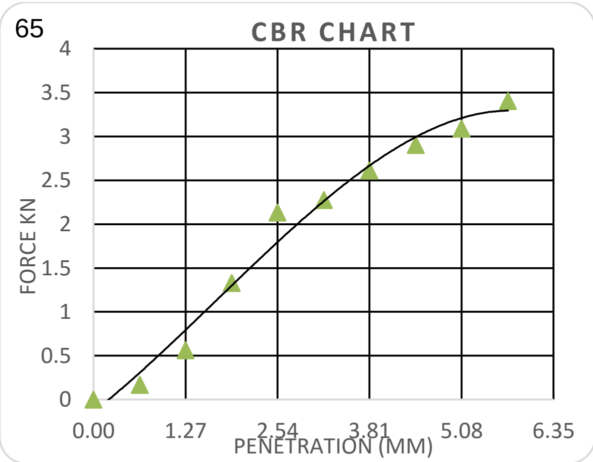
Compaction Determination						
COMPACTION DATA	65 Blows		30 Blows		10 Blows	
	Before soak	After soak	Before soak	After soak	Before soak	After soak
Mould No.	A1	A1	A2	A2	A3	A3
Mass of soil + Mould g	11502.2	11543.4	11601.7	11641.5	11338.3	11403.3
Mass Mould g	6674.2	6674.2	6548.4	6548.4	6638.3	6638.3
Mass of Soil g	4828	4869.2	5053.3	5093.1	4700	4765
Volume of Mould g	2060.21	2060.21	2060.21	2060.21	2060.21	2060.21
Wet density of soil g/cc	2.343	2.363	2.453	2.472	2.281	2.313
Dry density of soil g/cc	2.092	2.052	2.199	2.166	2.035	1.996

MOISTURE CONTENT DATA	65 Blows				30 Blows				10 Blows			
	Before soak		After soak		Before soak		After soak		Before soak		After soak	
Container no.	II	G10	F	ZE	P3	P25	A13	P15	ZZ	NB	36	29
Mass of wet soil + Container g	181.8	191.3	303.9	262.8	294.9	230.3	308.7	330.6	194.7	214.9	188.3	233.3
Mass of dry soil + Container g	164.5	172.4	266.2	234.7	267.5	208.7	275.3	293.6	175.6	193.6	164.2	204.6
Mass of container g	18.0	17.2	36.4	33.2	26.0	25.5	36.6	33.6	17.6	17.6	17.2	17.6
Mass of water g	17.4	18.9	37.7	28.1	27.4	21.6	33.5	37.0	19.1	21.3	24.1	28.7
Mass of drysoil g	146.4	155.3	229.7	201.6	241.5	183.2	238.7	260.0	158.0	176.0	147.0	187.0
Moisture content %	11.8	12.2	16.4	13.9	11.3	11.8	14.0	14.2	12.1	12.1	16.4	15.4
Average moisture content	12.0		15.2		11.6		14.1		12.1		15.9	

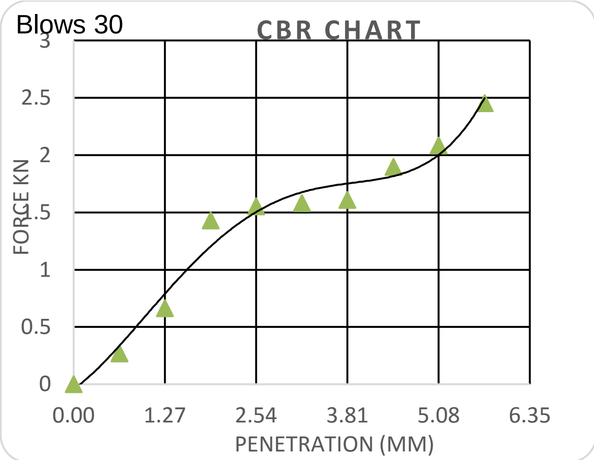
CBR Penetration Determination								
Penetration after 96 hrs Soaking Period				Surcharge Weight:4.55 KG				
65 Blows			30 Blows			10 Blows		
Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %
0.00	0.001		0.00	0.002		0.00	0.004	
0.64	0.168		0.64	0.268		0.64	0.201	
1.27	0.563		1.27	0.663		1.27	0.323	
1.91	1.331		1.91	1.431		1.91	0.516	
2.54	2.135	16.00	2.54	1.553	11.64	2.54	1.146	8.59
3.18	2.277		3.18	1.582		3.18	1.194	
3.81	2.612		3.81	1.608		3.81	1.228	
4.45	2.907		4.45	1.896		4.45	1.492	
5.08	3.091	15.46	5.08	2.083	10.42	5.08	1.575	7.88
5.72	3.407		5.72	2.453		5.72	2.067	

Modified Max.Dry Density g/cc	2.168	OMC %	14.0

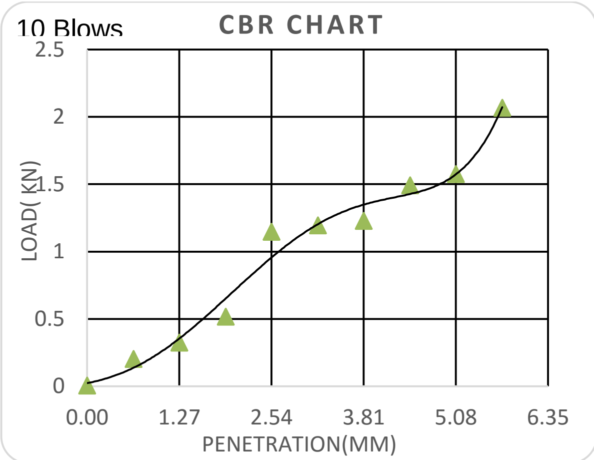
Swell Determination							
Date		65 Blows		30 Blows		10 Blows	
		Gauge reading	Swell in %	Gauge reading	Swell in %	Gauge reading	Swell in %
		mm		mm		mm	
13/9/2019	Initial	24.46	0.15	19.455	0.20	24.46	0.20
17/9/2019	Final	24.64		19.69		24.69	



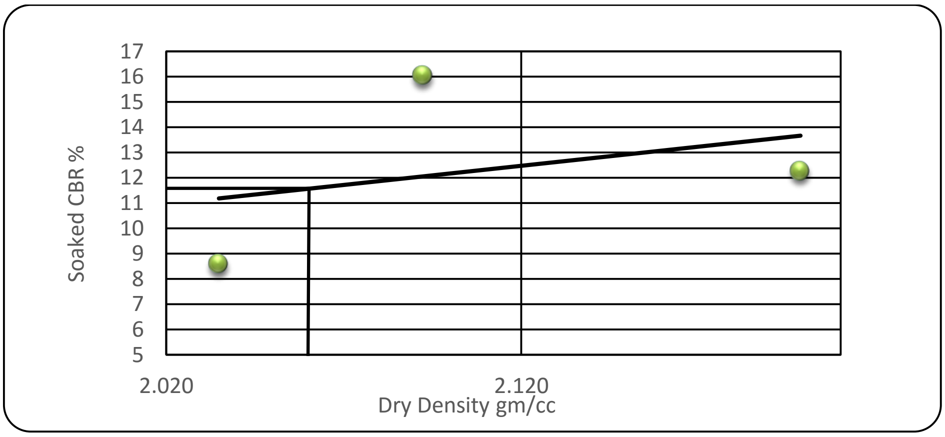
Penetration (mm)	Load KN		Corr. CBR %	Swell %
	Top	Bottom		
2.54mm		2.1	16.1	0.15
5.08mm		3.1	15.5	



Penetration (mm)	Load KN		Corr. CBR %	Swell %
	Top	Bottom		
2.54mm		1.6	11.7	0.20
5.08mm		2.5	12.3	



Penetration (mm)	Load KN		Corr. CBR %	Swell %
	Top	Bottom		
2.54mm		1.1	8.6	0.20
5.08mm		1.6	7.9	



Dry Density at 95% of MDD:	2.059

No.of blows	MCBS %	DDBS g/cm3	Correcrt CBR %	% of Compaction
10	12.1	2.035	8.6	94
30	11.6	2.199	12.3	101
65	12.0	2.092	16.1	97

CBR % at 95 % MDD	11.6	Swell %	0.20

Standard Method of Test for CBR: AASHTO T-193			
Sample date:	15/08/2019		
Soak date:	14/9/2019		
Test Date:	18/9/2019	Type of Material:	15%Soil +85%Selected Granular Materials

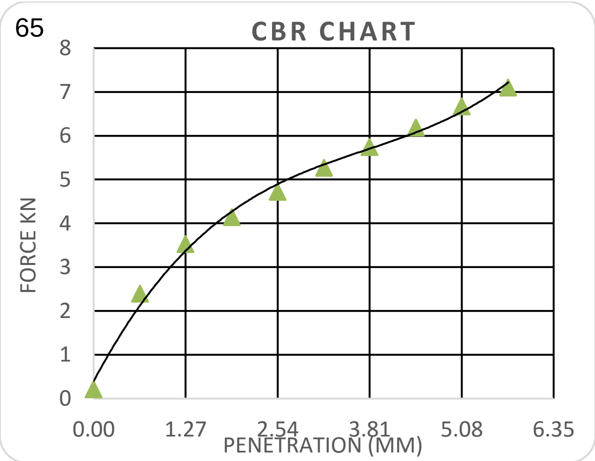
Compaction Determination						
COMPACTION DATA	65 Blows		30 Blows		10 Blows	
	Before soak	After soak	Before soak	After soak	Before soak	After soak
Mould No.	B1	B1	B2	B2	B3	B3
Mass of soil + Mould g	11411.4	11510.3	11137.1	11273.1	10902.9	11123.1
Mass Mould g	6595.1	6595.1	6572.3	6572.3	6736.8	6736.8
Mass of Soil g	4816.3	4915.2	4564.8	4700.8	4166.1	4386.3
Volume of Mould g	2060.21	2060.21	2060.21	2060.21	2060.21	2060.21
Wet density of soil g/cc	2.338	2.386	2.216	2.282	2.022	2.129
Dry density of soil g/cc	2.097	2.033	2.010	1.931	1.804	1.787

Moisture Determination												
MOISTURE CONTENT DATA	65 Blows				30 Blows				10 Blows			
	Before soak		After soak		Before soak		After soak		Before soak		After soak	
Container no.	C3	NC1	B3	SB	G3	P66	1A	DH1	C2	52	G21	P5
Mass of wet soil + Container g	214.1	102.8	198.2	190.7	239.5	231.5	183.3	192.1	179.8	155.9	174.6	187.0
Mass of dry soil + Container g	195.6	93.7	168.2	168.5	220.4	213.8	155.9	167.3	162.3	141.1	149.4	159.7
Mass of container g	26.7	17.5	17.4	18.4	37.9	37.4	17.8	17.0	17.7	17.9	18.1	17.3
Mass of water g	18.4	9.2	30.0	22.2	19.1	17.7	27.4	24.8	17.6	14.8	25.1	27.3
Mass of drysoil g	168.9	76.2	150.8	150.1	182.5	176.4	138.1	150.2	144.6	123.2	131.3	142.4
Moisture content %	10.9	12.0	19.9	14.8	10.5	10.0	19.8	16.5	12.1	12.0	19.1	19.2
Average moisture content %	11.5		17.3		10.3		18.2		12.1		19.1	

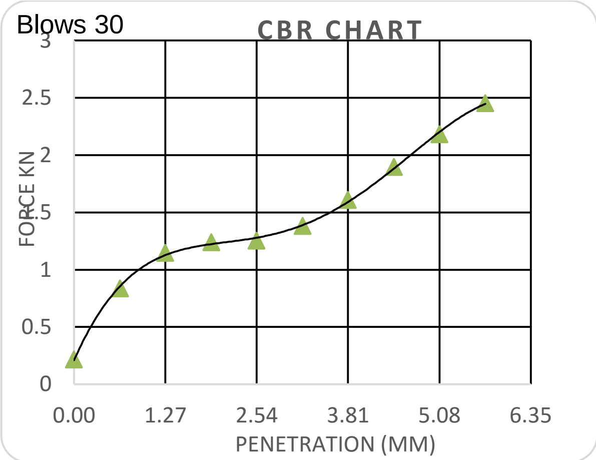
CBR Penetration Determination								
Penetration after 96 hrs Soaking Period				Surcharge Weight:4.55 KG				
65 Blows			30 Blows			10 Blows		
Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %
0.00	0.209		0.00	0.216		0.00	0	
0.64	2.394		0.64	0.836		0.64	0.2	
1.27	3.524		1.27	1.148		1.27	0.201	
1.91	4.141		1.91	1.239		1.91	0.223	
2.54	4.718	35.37	2.54	1.253	9.39	2.54	0.453	3.40
3.18	5.265		3.18	1.382		3.18	0.454	
3.81	5.741		3.81	1.608		3.81	0.583	
4.45	6.186		4.45	1.896		4.45	0.725	
5.08	6.668	33.34	5.08	2.183	10.92	5.08	0.877	4.39
5.72	7.097		5.72	2.453		5.72	0.96	

Modified Max.Dry Density g/cc	2.090	OMC %	13.2

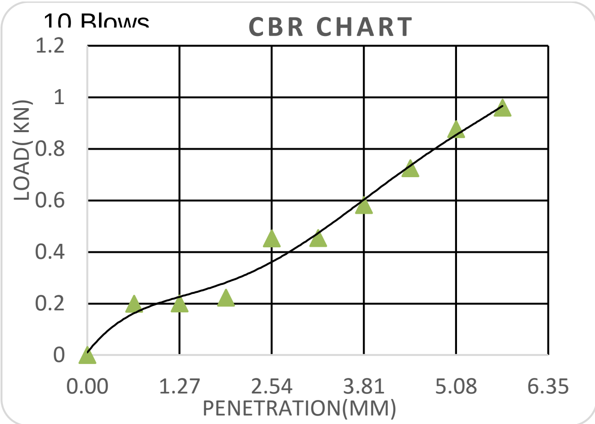
Swell Determination							
Date		65 Blows		30 Blows		10 Blows	
		Gauge reading	Swell in %	Gauge reading	Swell in %	Gauge reading	Swell in %
		mm		mm		mm	
14/9/2019	Initial	23.47	0.40	27.3	0.58	23.23	1.60
18/9/2019	Final	23.93		27.97		25.09	



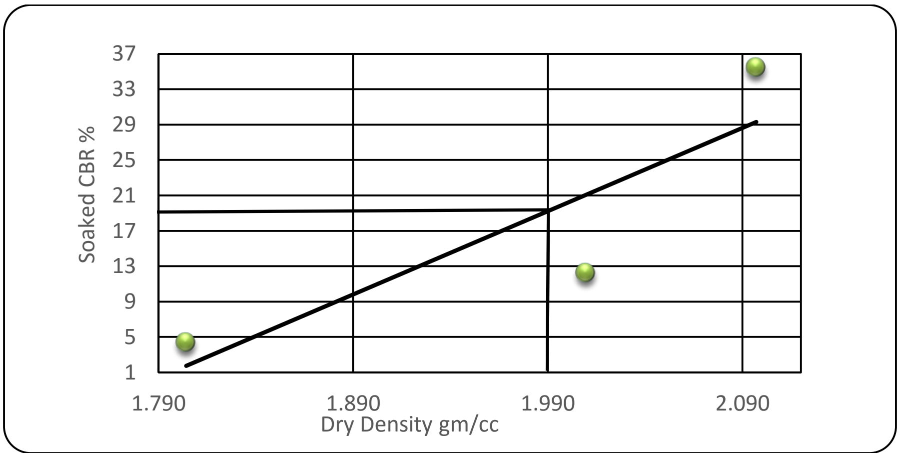
Penetration (mm)	Load KN		Corr. CBR %	Swell %
	Top	Bottom		
2.54mm		4.7	35.5	0.40
5.08mm		6.7	33.3	



Penetration (mm)	Load KN		Corr. CBR %	Swell %
	Top	Bottom		
2.54mm		1.3	9.4	0.58
5.08mm		2.5	12.3	



Penetration (mm)	Load KN		Corr. CBR %	Swell %
	Top	Bottom		
2.54mm		0.5	3.4	1.60
5.08mm		0.9	4.4	



Dry Density at 95% of MDD:	1.986

No.of blows	MCBS %	DDBS g/cm3	Correcrt CBR %	% of Compaction
10	12.1	1.804	4.4	86
30	10.3	2.010	12.3	96
65	11.5	2.097	35.5	100

CBR % at 95 % MDD	19.0	Swell %	0.58

Standard Method of Test for CBR: AASHTO T-193			
Sample date:	15/08/2019		
Soak date:	14/9/2019		
Test Date:	18/9/2019	Type of Material:	30%Soil +70%Selected Granular Materials

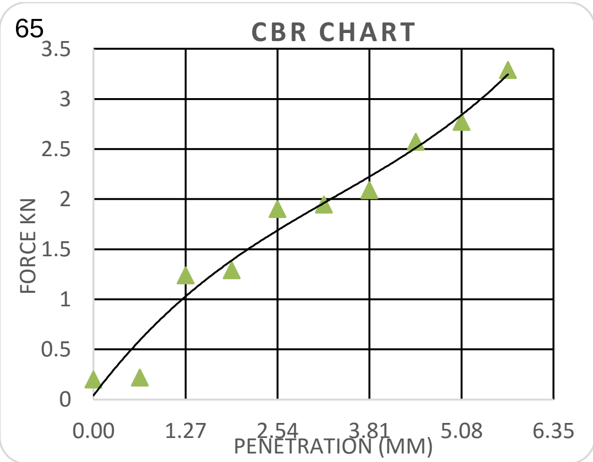
Compaction Determination						
COMPACTION DATA	65 Blows		30 Blows		10 Blows	
	Before soak	After soak	Before soak	After soak	Before soak	After soak
Mould No.	C1	C1	C2	C2	C3	C3
Mass of soil + Mould g	11258.2	11346.2	11196.7	11306	10885.4	11085.3
Mass Mould g	6598.5	6598.5	6628.1	6628.1	6613.5	6613.5
Mass of Soil g	4659.7	4747.7	4568.6	4677.9	4271.9	4471.8
Volume of Mould g	2060.21	2060.21	2060.21	2060.21	2060.21	2060.21
Wet density of soil g/cc	2.262	2.304	2.218	2.271	2.074	2.171
Dry density of soil g/cc	1.964	1.932	1.954	1.887	1.800	1.771

Moisture Determination												
MOISTURE CONTENT DATA	65 Blows				30 Blows				10 Blows			
	Before soak		After soak		Before soak		After soak		Before soak		After soak	
Container no.	F	P15	2	G19	DH	P5	31	P65	1A	HC51	SG1	69
Mass of wet soil + Container g	179.8	228.6	245.7	240.6	152.4	170.9	264.9	263.7	172.0	163.0	269.1	247.1
Mass of dry soil + Container g	161.5	202.2	209.9	208.9	135.9	153.2	223.9	228.1	152.6	142.9	225.0	205.9
Mass of container g	36.4	33.6	34.7	34.3	17.0	17.2	36.7	37.8	17.7	17.7	26.7	25.4
Mass of water g	18.3	26.4	35.8	31.7	16.5	17.8	41.0	35.6	19.3	20.1	44.1	41.2
Mass of drysoil g	125.0	168.6	175.2	174.6	118.9	136.0	187.1	190.3	134.9	125.3	198.2	180.5
Moisture content %	14.6	15.6	20.4	18.2	13.9	13.1	21.9	18.7	14.3	16.0	22.2	22.8
Average moisture content %	15.1		19.3		13.5		20.3		15.2		22.5	

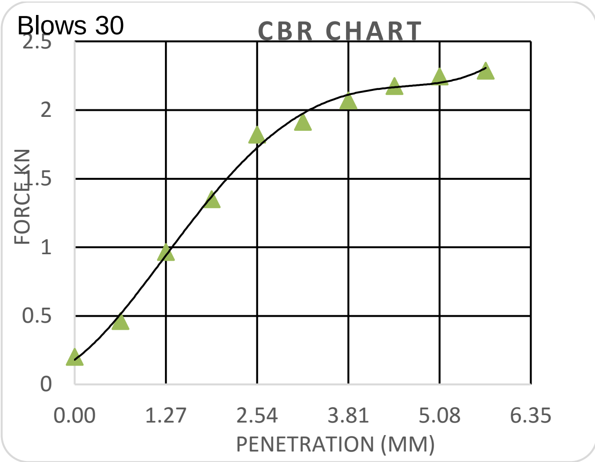
CBR Penetration Determination								
Penetration after 96 hrs Soaking Period			Surcharge Weight:4.55 KG					
65 Blows			30 Blows			10 Blows		
Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %
0.00	0.201		0.00	0.201		0.00	0.201	
0.64	0.221		0.64	0.462		0.64	0.441	
1.27	1.239		1.27	0.965		1.27	0.476	
1.91	1.292		1.91	1.351		1.91	0.527	
2.54	1.901	14.25	2.54	1.82	13.64	2.54	1.419	10.64
3.18	1.947		3.18	1.915		3.18	1.429	
3.81	2.089		3.81	2.07		3.81	1.432	
4.45	2.574		4.45	2.176		4.45	1.514	
5.08	2.771	13.86	5.08	2.244	11.22	5.08	1.524	7.62
5.72	3.29		5.72	2.286		5.72	1.558	

Modified Max.Dry Density g/cc	1.925	OMC %	16.8

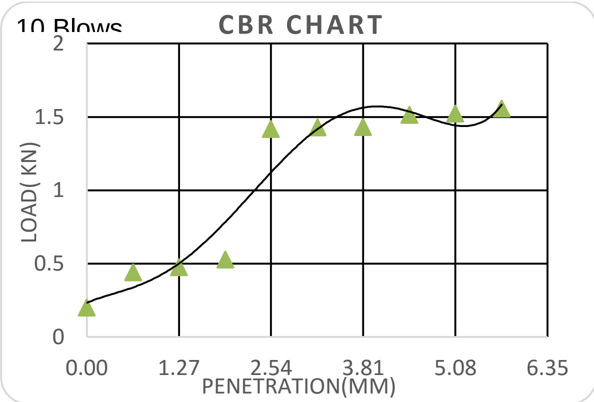
Swell Determination							
Date		65 Blows		30 Blows		10 Blows	
		Gauge reading	Swell in %	Gauge reading	Swell in %	Gauge reading	Swell in %
		mm		mm		mm	
14/9/2019	Initial	24.86	0.05	24.38	0.06	24.46	0.70
18/9/2019	Final	24.92		24.46		25.28	



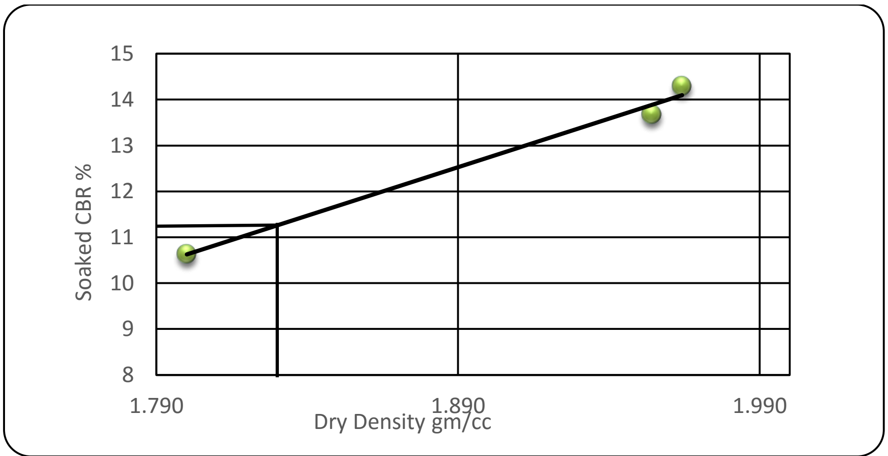
Penetration (mm)	Load KN		Corr. CBR %	Swell %
	Top	Bottom		
2.54mm		1.9	14.3	0.05
5.08mm		2.8	13.9	



Penetration (mm)	Load KN		Corr. CBR %	Swell %
	Top	Bottom		
2.54mm		1.8	13.7	0.06
5.08mm		2.3	11.4	



Penetration (mm)	Load KN		Corr. CBR %	Swell %
	Top	Bottom		
2.54mm		1.4	10.6	0.70
5.08mm		1.5	7.6	



Dry Density at 95% of MDD:	1.829

No.of blows	MCBS %	DDBS g/cm3	Correcrt CBR %	% of Compaction
10	15.2	1.800	10.6	94
30	13.5	1.954	13.7	102
65	15.1	1.964	14.3	102

CBR % at 95 % MDD	11.3	Swell %	0.06

Standard Method of Test for CBR: AASHTO T-193			
Sample date:	15/08/2019		
Soak date:	14/9/2019		
Test Date:	18/9/2019	Type of Material:	45%Soil +55%Selected Granular Materials

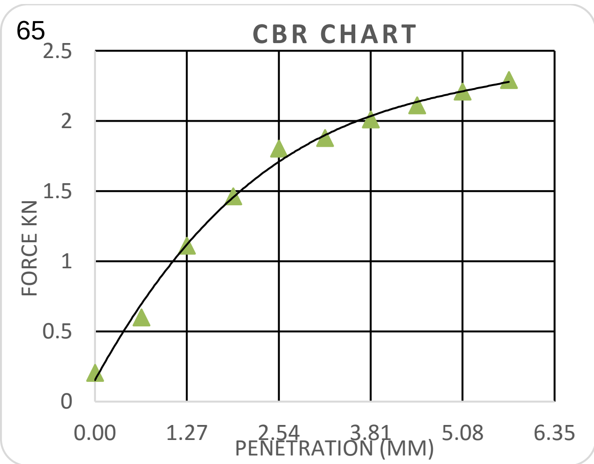
Compaction Determination						
COMPACTION DATA	65 Blows		30 Blows		10 Blows	
	Before soak	After soak	Before soak	After soak	Before soak	After soak
Mould No.	D1	D1	D2	D2	D3	D3
Mass of soil + Mould g	11149.9	11241.6	11068	11212.2	10748.2	10955.6
Mass Mould g	6596.1	6596.1	6707.1	6707.1	6642.3	6642.3
Mass of Soil g	4553.8	4645.5	4360.9	4505.1	4105.9	4313.3
Volume of Mould g	2060.21	2060.21	2060.21	2060.21	2060.21	2060.21
Wet density of soil g/cc	2.210	2.255	2.117	2.187	1.993	2.094
Dry density of soil g/cc	1.892	1.832	1.801	1.764	1.684	1.687

Moisture Determination												
MOISTURE CONTENT DATA	65 Blows				30 Blows				10 Blows			
	Before soak		After soak		Before soak		After soak		Before soak		After soak	
Container no.	A13	ZE	NB	NC1	14	3	ZZ	5LS	G7	HC12	II	G3T3
Mass of wet soil + Container g	191.1	214.5	160.6	169.9	179.5	140.8	173.9	317.9	184.5	179.8	208.5	210.5
Mass of dry soil + Container g	169.1	188.0	132.6	142.6	155.8	122.0	142.8	263.1	159.2	154.0	169.7	175.0
Mass of container g	36.6	33.2	17.6	17.5	17.4	17.6	17.6	26.0	17.4	18.2	17.5	18.0
Mass of water g	22.0	26.5	28.0	27.3	23.7	18.8	31.1	54.8	25.2	25.8	38.8	35.6
Mass of drysoil g	132.5	154.9	115.0	125.1	138.4	104.3	125.2	237.1	141.8	135.9	152.2	157.0
Moisture content %	16.6	17.1	24.3	21.9	17.1	18.0	24.8	23.1	17.8	19.0	25.5	22.7
Average moisture content %	16.8		23.1		17.6		24.0		18.4		24.1	

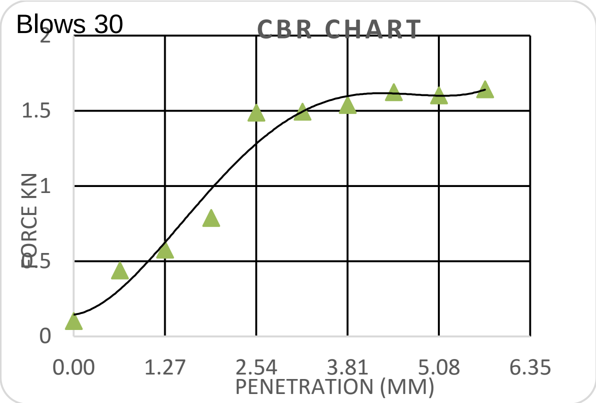
CBR Penetration Determination								
Penetration after 96 hrs Soaking Period			Surcharge Weight:4.55 KG					
65 Blows			30 Blows			10 Blows		
Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %
0.00	0.205		0.00	0.102		0.00	0.101	
0.64	0.6		0.64	0.437		0.64	0.337	
1.27	1.113		1.27	0.576		1.27	0.476	
1.91	1.463		1.91	0.786		1.91	0.686	
2.54	1.803	13.52	2.54	1.486	11.14	2.54	1.02	7.65
3.18	1.879		3.18	1.493		3.18	1.045	
3.81	2.011		3.81	1.539		3.81	1.082	
4.45	2.113		4.45	1.624		4.45	1.097	
5.08	2.211	11.06	5.08	1.602	8.01	5.08	1.15	5.75
5.72	2.293		5.72	1.643		5.72	1.175	

Modified Max.Dry Density g/cc	1.865	OMC %	19.6

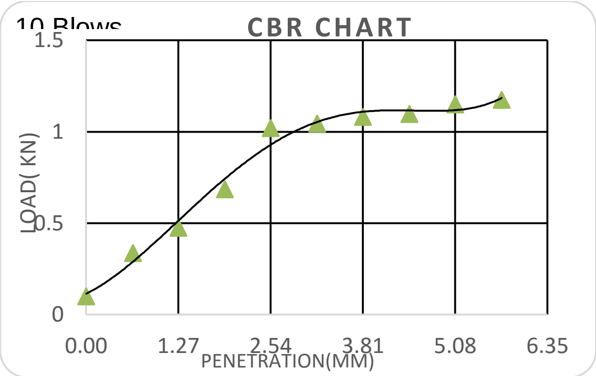
Swell Determination								
Date		65 Blows		30 Blows		10 Blows		
		Gauge reading	Swell in %	Gauge reading	Swell in %	Gauge reading	Swell in %	
		mm		mm		mm		
14/9/2019	Initial	25.42	0.22	24.06	0.34	23.28	0.36	
18/9/2019	Final	25.68		24.46		23.70		



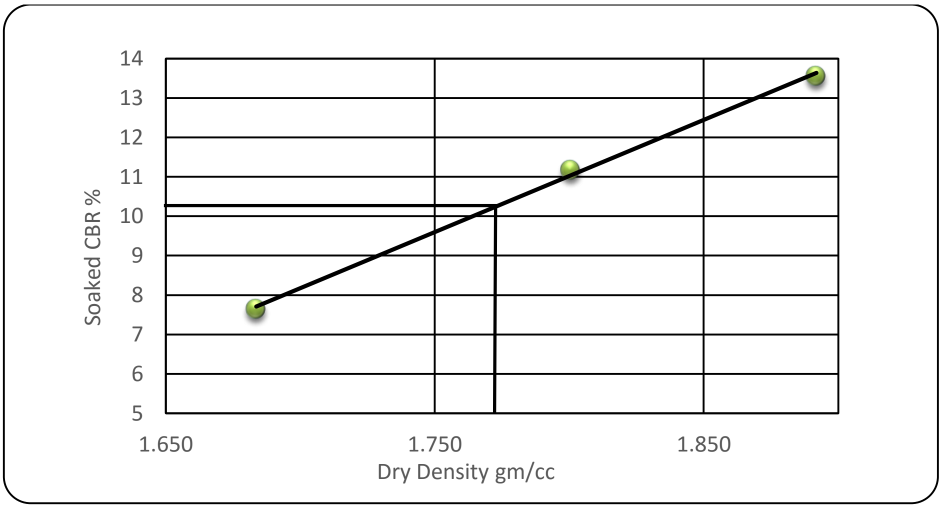
Penetration (mm)	Load KN		Corr. CBR %	Swell %
	Top	Bottom		
2.54mm		1.8	13.6	0.22
5.08mm		2.2	11.1	



Penetration (mm)	Load KN		Corr. CBR %	Swell %
	Top	Bottom		
2.54mm		1.5	11.2	0.34
5.08mm		1.6	8.2	



Penetration (mm)	Load KN		Corr. CBR %	Swell %
	Top	Bottom		
2.54mm		1.0	7.6	0.36
5.08mm		1.2	5.8	



Dry Density at 95% of MDD:	1.772

No.of blows	MCBS %	DDBS g/cm3	Correcrt CBR %	% of Compaction
10	18.4	1.684	7.6	90
30	17.6	1.801	11.2	97
65	16.8	1.892	13.6	101

CBR % at 95 % MDD	10.3	Swell %	0.34

Appendix F. Photographs of Laboratory Tests

Atterberg Limit Test

Specific Gravity Test

Mechanical Sieve Analysis for the Sand and Coarser Fraction.

Hydrometer Test for the Silt and finer Clay material.

Compaction Test

California Bearing Ratio (CBR) Test