



ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING

**BLENDING OF CINDER WITH FINE-GRAINED SOIL TO BE USED AS
SUB-BASE MATERIALS
(The Case of Butajira-Gubre Road)**

A THESIS SUBMITTED TO THE ADDIS ABABA INSTITUTE OF
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ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES

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SYMBOLS AND ABBREVIATION

AACRA – Addis Ababa City Road Authority`

AASHTO – American Association of Highway and Transportation Officials

ASTM – American Society for Testing and Materials

CBR – California Bearing Ratio

DD – Dry Density

E – Easting

E.C. –Ethiopian Calendar

ERA – Ethiopia Road Authority

GM – Grading Modules

GPS – Global Position System

GSB – Granular Sub-Base

LS – Linear Shrinkage

LAA – Los Angeles Abrasion

MDD – Maximum Dry Density

N – Northing

NNE – North-North East

NP – Non-Plastic

OMC – Optimum Moisture Content

PI – Plastic Index

SSW – South-South West

TRRL – Transport and Road Research Laboratory

UCS – Unconfined Compressive Strength

UK – United Kingdom

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ABSTRACT

In road construction, the use of locally available materials should be made as much as possible. However, these materials are not always suitable and when appropriate material cannot be found in areas close to the construction site, very high prices have to be paid with significant time delays and cost increases.

In many parts of Ethiopia, there is widely distributed cinder gravel. However, this material has the problem of compaction due to its light weight, its rough circular surface and its high porosity.

This thesis deals with investigating the performance of mechanically stabilized natural cinder gravels to be used as road sub-base material. To achieve the Ethiopia Road Authority manual specification, the cinder gravel was blended with some trial proportion of 0, 5, 10, 15, 20, and 25 % of fine-grained soil by mass and different tests including grain size distribution, Atterberg Limit, compaction, CBR, LAA, absorption and linear Shrinkage are conducted in the laboratory.

Based on the laboratory test results it is shown that, from both MDD- percent of fine-grained soil curve and CBR-percent of fine-grained soil curve, the optimum amount of fine-grained soil required in order to improve its properties is 19 % by mass proportion.

KEY WORDS: sub-base, Cinder Gravel, Blending of sub-base material, Optimum fine-grained soil Content, Gradation, Arithmetical proportioning.

CHAPTER 1

INTRODUCTION

1.1 General

Sub-base is an important load spreading layer in a completed pavement. It enables traffic stresses to be reduced to acceptable levels in the sub-grade, acts as a working platform for the construction of the upper pavement layers and serves as a separation layer between sub-grade and base course. The sub-base acts as a load spreading layer, also during construction, when it carries heavy site traffic and in addition protects to a certain extent the sub-grade (as long as conditions are not too wet) [14]. Under special circumstances, it may also act as a filter or as a drainage layer.

In wet climatic conditions, the most stringent requirements are dictated by the need to support construction traffic and paving equipment. In these circumstances, the sub-base material needs to be more tightly specified. In dry climatic conditions, in areas of good drainage, and where the road surface remains well sealed, un-saturated moisture conditions prevail and sub-base specifications may be relaxed. The selection of sub-base materials will therefore depend on the design function of the layer and the anticipated moisture regime, both in service and during construction.

Sub-base is the secondary load-spreading layer underlying the road base. It will normally consist of a material of lower quality than that used in the road base such as unprocessed natural gravel, gravel-sand, or gravel-sand-clay. This layer also serves as a separating layer preventing contamination of the road base by the sub-grade material and, under wet conditions; it has an important role to play in protecting the sub-grade from damage by construction traffic.

Material selection is an important component of pavement design. The selection of appropriate quality of materials for selected sub-grade, sub-base and road-base determines the capital and whole life costs of the road. The use of locally available materials is very much encouraged in most road construction especially in rural road engineering [20].

The materials used for the construction of sub-base layers shall be either [10, 19]:

1. Natural Gravel
2. Scoria (Cinder Gravel)
3. Weathered Rock
4. Crushed Gravel
5. Crushed Rock or Crushed Boulders
6. Recycled Pavement Material.

The component parts of a road consist of individual layers of specified materials laid and compacted to form a stable composite. A road foundation consisting of sub-base and capping layers is designed to achieve a satisfactory working platform for constructing the pavement. On thinner pavements it also contributes to the overall load spreading capability of the pavement structure [5]. A capping layer is a layer of selected fill material placed on the top of the topmost embankment layer or the bottom of excavation. These materials are often required to provide sufficient cover on weak sub-grades. 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 [9, 19].

The capping layer above the existing natural or compacted soil upgrades, the structural performance of the in-situ material whilst the sub-base provides a foundation for the base and wearing courses. Specifications of these materials for highway works are identified. Geographical variations occur in the application of specifications to take account of the variability of the locally available primary materials [5].

This research was conducted taking the case of Butajira-Gubre Road Project which is 87 km long and starts at Butajira station. In most of the area along the side of road, good type of sub-base material is not available. But in this area there is abundant cinder gravel material and is used as a sub-base material by mixing it with fine-grained soil but without any specified proportion. The objective of this thesis is to investigate the performance of cinder gravel blended with fine-grained soil as sub-base material and propose the best blending proportion. For this purpose, cinder samples and locally available fine-grained soil were collected from the Butajira- Gubre road which is under construction.

1.2 Background of the Problem

In road infrastructure projects, the quality of the sub-base material and its grain size distribution are very important. If appropriate sub-base material cannot be found in areas close to the construction site, then very high prices have to be paid in road construction process, which causes significant delays or cost increases. In such cases, sometimes work with low-quality materials affects the road quality and durability over time and results in very significant losses. Improving the quality of materials is very important for road construction works, in order to ensure that projects meet the necessary cost and quality criteria. If the material found close to the construction site does not meet the specifications, the materials may be improved with suitable chemicals such as lime, cement and fly ash, etc. All the additives may be advantageous to certain type of materials [18].

In many parts of Ethiopia, there is widely distributed cinder gravel. However, this material has the problem of compaction due to its light weight, its rough circular surface and its high porosity. Beside, in some regional part of Ethiopia, there is a scarcity of a good sub-base material such as natural gravel, crushed rock or recycled pavement material. Since the distribution of these sub-base materials is limited only in some parts of the country, using these materials everywhere incurs transportation cost and is time consuming. In those areas where cinder gravel is available, they are used mixed with fine-grained soils without having any research based out put and guideline about their proportion.

1.3 Objective of the Study

1.3.1 General objective

- To determine a good blending proportion of the cinder gravel with fine-grained soil used as sub-base materials.

1.3.2 Specific objectives

- To improve the compaction property of cinder gravel.
- To characterize the Grading limits, Grading Modules, Plasticity index, California Bearing Ratio, Compaction and Los Angeles Abrasion of the blended cinder materials.

1.4 Methodology

To achieve the objectives of this thesis, the following methodologies have been followed.

- Literature review.
- Field work, laboratory test; and analysis. On the field, the borrow soil and cinder gravel that are used for blending were collected. Disturbed samples were collected and tests were conducted in the laboratory. Based on AASHTO classification method, the soil type was classified. The cinder gravel was selected based on location, access and ease of workability. The cinder gravel was blended with fine-grained soil in some trial proportion to achieve the Ethiopia Road Authority (ERA) Manual requirement.

A series of tests conducted in the laboratory, include:

- Atterberg Limits
- Specific Gravity
- Gradation
- California Bearing Ratio

- Compaction
- Los Angeles Abrasion
- Linear Shrinkage
- Absorption

1.5 Organization of the Thesis

This thesis contains five Chapters and appendices. The first Chapter contains introduction of sub-base layer, back ground of the problem, objectives and methodology. Chapter two covers a literature review containing location and engineering properties of volcanic cinder gravels in Ethiopia; modification of existing materials and method of stabilization; soil survey for highway purpose; related works on cinder gravel in road construction and economical aspects of blending material. In the third Chapter, the general description of the study area and the proposed material source are discussed. In the fourth Chapter the laboratory test results and discussions are presented. In Chapter five conclusions and recommendations are given. Reference materials used in the research work are appropriately sited and listed. The thesis ends with appendices which contain detail experimental results of laboratory investigation.

CHAPTER 2

LITERATURE REVIEW

2.1 The Location and Engineering Properties of Volcanic Cinder Gravels in Ethiopia

2.1.1 Introduction

Volcanic cinder gravels occur extensively in Ethiopia, but in the past they have been used for road construction only to a limited extent, even though their use would substantially reduce road construction costs.

As part of a joint research project undertaken by the Ethiopian Road Authority and the United Kingdom Transport and Road Research Laboratory, research has been carried out previously to provide information on the occurrence and properties of the volcanic cinders with the objective of encouraging their wider use in road construction.

This joint research project was commenced in Ethiopia by the Ethiopian Road Authority (ERA) and Transport and Road Research Laboratory (TRRL, UK) in 1975. One of the objectives of the research program was to establish guidelines to enable highway engineers to make full use of locally-available materials for road construction. This was considered to be particularly relevant in Ethiopia and an extensive program of rural road construction is in progress which is expected to continue for a number of years [16].

2.1.2 Definition of Volcanic Cinders

Volcanic cinders are pyroclastic materials associated with recent volcanic activity. They occur in characteristically straight sided cone-shaped hills which frequently have large concave depressions in their tops or sides where mixtures of solids and gases were released during the formation of the cone [16]. Cinders vary in colour often within the same cone and may be red, brown, grey or black. The cinder particles also vary in size from large irregularly shaped lumps 50 cm in size, to sand and silt sizes. In some cones, however, particles may be more uniform with the largest size not exceeding 3 cm in diameter. Other characteristic features of cinders are their

light weight, their rough vesicular surface and their high porosity. Usually they are weak enough to be crushed under the heel [16].

An advantage of cinders as a road construction material is the relative ease with which they can be dug from the quarry; a mechanical shovel or hand tools are usually adequate for their extraction although occasionally a bulldozer may be required to open up a working face.

2.1.3 General location of cinder gravel in Ethiopia

A survey is made under TRRL and ERA which included the examination of aerial photographs and photo-mosaics and the preparation of a map showing the distribution of cinder deposits throughout Ethiopia [16].

Field visits in connection with the survey were all carried out within a distance of 150 km of Addis Ababa. They were concentrated in areas near to Debrezeit, Nazareth, Ziway, Butajira and Giyon (see Figure 2.1).

Cinder cones rarely support any vegetation other than grasses and examination of the exposed profiles showed that the depth of soil cover was not more than a few centimeters. A weathered cinder zone, however, usually extends down to a depth of about two meters. In some cone deposits of calcium carbonate coats the cinder gravels; these does not persist throughout the cones but in thin white bands parallel to and usually close to the surface [16].

The distinctive shape of the cinder cones made them easily identifiable on aerial photographs and aerial photographs were used both to plan the survey and subsequently in the field work [16].

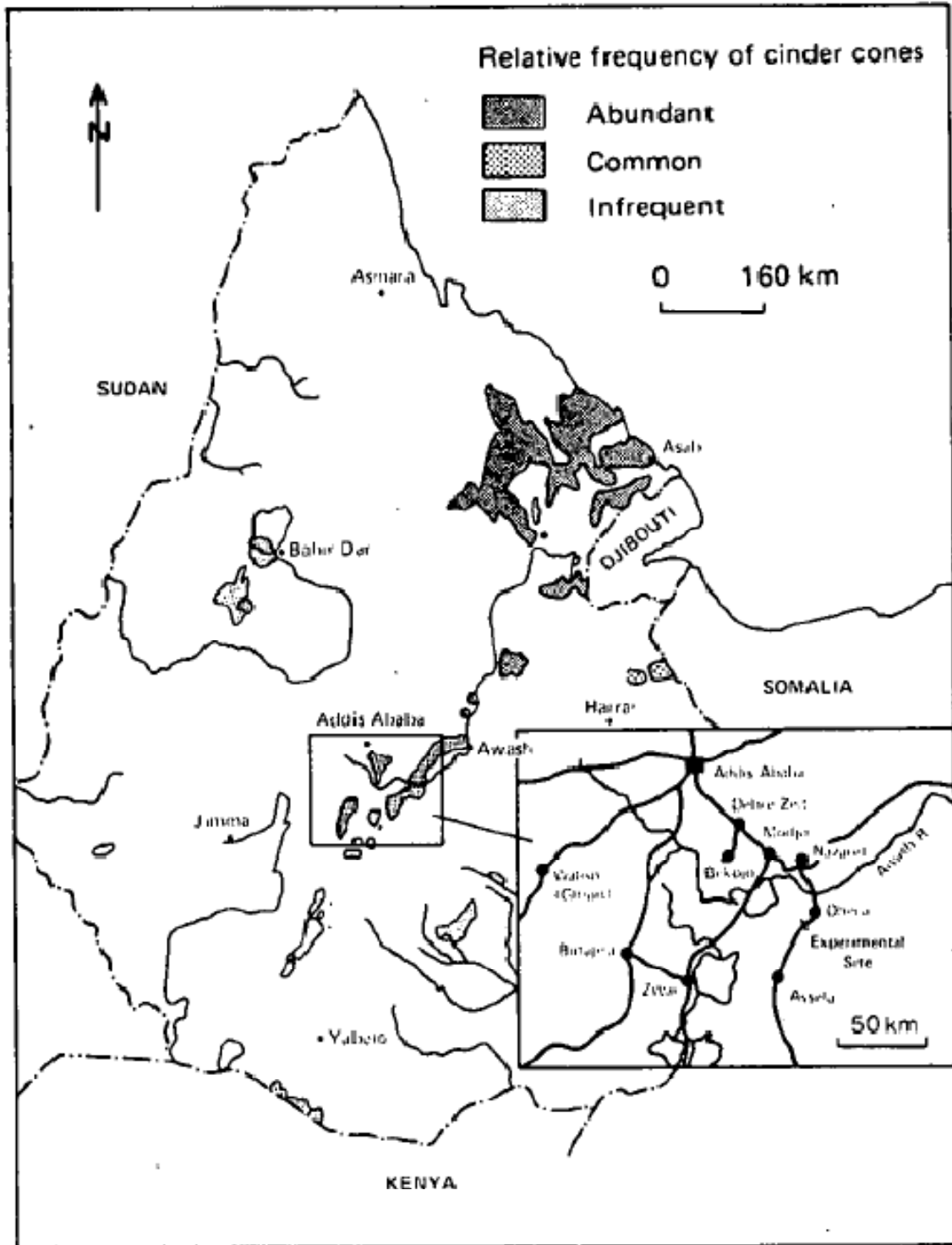


Figure 2.1: Cinder cone distribution in Ethiopia, survey area enlarged [16]

The examination of air-photos and print lay-downs was extended to cover the whole of Ethiopia and from these and a study of areas of recent (Quaternary) volcanic on the geological map, a preliminary map was compiled showing the occurrence of cones throughout the country (see Figure 2.1).

Cinder gravels were mostly concentrated in the Rift Valley that bisects the country in a SSW-NNE direction; an indication of their frequency for each of the areas that were identified has been given in figure 2.1.

2.2 Soil Survey for Highway Purposes

Soil surveys are made in connection with highway location, design and construction. Many sources of information concerning soils generally are available for the area in which a highway project is to be carried out. These sources include geological and topographic maps and reports, agricultural soil maps and reports, aerial photographs, and the previous soil survey in the area. Information of such source is of importance in two general ways. First, a study of this information will aid in securing a broad understanding of soil conditions and associated engineering problems that may be encountered. Second, such information is of great value in planning, conducting and interpreting the results of detailed soil survey that are necessary for design and construction. Modern techniques emphasize the use of all available information about a given area in order to minimize the amount of detailed field and laboratory work necessary [17].

2.2.1 Modification of properties of existing materials

Existing materials may require modifications so as to improve their engineering properties. Also, locally available materials, which are otherwise not satisfying general specification requirement, can be suitably modified to become acceptable. This also serves the purpose of economy in terms of saving of haulage of costly materials from elsewhere. Sometimes, design may require special purpose material having specific properties which can be achieved through material modification [17].

2.2.2 Soil stabilized roads

Soil stabilization is the alteration of one or more soil properties, by mechanical or chemical means, to create an improved soil material possessing the desired engineering properties. Soils may be stabilized to increase strength and durability or to prevent erosion and dust generation. Regardless of the purpose for stabilization, the desired result is the creation of a soil material or soil system that will remain in place under the design use conditions for the design life of the project [4].

Soil stabilization for roads may be defined as the combinations and manipulation of soils, with or without admixture, to produce a firm mass that is capable of supporting traffic in all conditions. If a stabilized soil is to be truly “stable”, then it must have sufficient shearing strength to withstand the stresses imposed on it by traffic loads in all kinds of weather without excessive deformation.

In highway design and construction the emphasis is placed on the effective utilization of local materials, with a view towards decreased construction costs. The primary use of stabilized soil mixtures at the present time is in base and sub-base construction. A stabilized soil base and sub-base may provide the support for a relatively thin wearing surface that will be subjected to light or moderate amounts of traffic, or it may function as a base for a high type pavement that will be subjected to very heavy volumes of traffic [17].

The strength of crushed stone base material used for road infrastructure was increased after the addition of polymer, while deformations of the road surface were reduced. The polymer additive results demonstrate that using polymer in granular road materials can improve the soil's engineering properties and provide significant contributions to road infrastructure projects. The results indicate that addition of 1% polymer by dry weight results in improved geotechnical characteristics in the field applications and create a polymer soil composite [18].

2.3 Methods of Stabilization

2.3.1 Mechanical stabilization

Mechanical stabilization by compaction produces an interlocking of soil-aggregate particles. The grading of the soil-aggregate mixture must be such that a dense mass is produced when it is compacted. Mechanical stabilization can be accomplished by uniformly mixing the material and then compact the mixture. As an alternative, additional fines or aggregates may be blended before compaction to form a uniform, well graded, dense soil-aggregate mixture after compaction. The choice of methods should be based on the gradation of the material. In some instances, geotextiles can be used to improve a soil's engineering characteristics [4].

The three essentials for obtaining a properly stabilized soil mixture are:

- Proper gradation.
- A satisfactory binder soil.
- Proper control of the mixture content.

The mixture will normally be compacted at or near OMC to obtain satisfactory densities. The objective of mechanical stabilization is to blend available soils so that, when properly compacted, they give the desired stability. In certain areas, for example, the natural soil at a selected location may have low load-bearing strength because of an excess of clay, silt, or fine sand. Within a reasonable distance, suitable granular materials may occur that may be blended with the existing soils to markedly improve the soil at a much lower cost in manpower and materials than is involved in applying imported surfacing [4, 17].

2.3.1.1 Particle size and gradation

The particular packing arrangement for a material is normally represented by the particle size distribution (gradation) curve based on proportions (by mass) passing successive sieves. A lack of coarse or finer particles would produce an unbalanced gradation or distorted gradation curve resulting in poor mechanical stability and unsatisfactory compaction. Therefore, an improvement in gradation and in the reduction of oversized material will result in more uniform strength

development, uniform mixing and compaction. It is preferable to have a gradation with a continuously smooth curve from the maximum particle size to the smallest particle size with no excess or lack in certain particle fractions. The gradation depends on the amount of weathering of the material. This means that material close to the surface will most probably be finer graded than material that is retrieved at a greater depth [22].

An aggregate, with little or no fines content as shown in figure 2.2a, gains stability from grain-to-grain contact. An aggregate that contains no fines usually has a relatively low density but is pervious and not frost susceptible. This material is however difficult to handle during construction because of its non-cohesive nature.

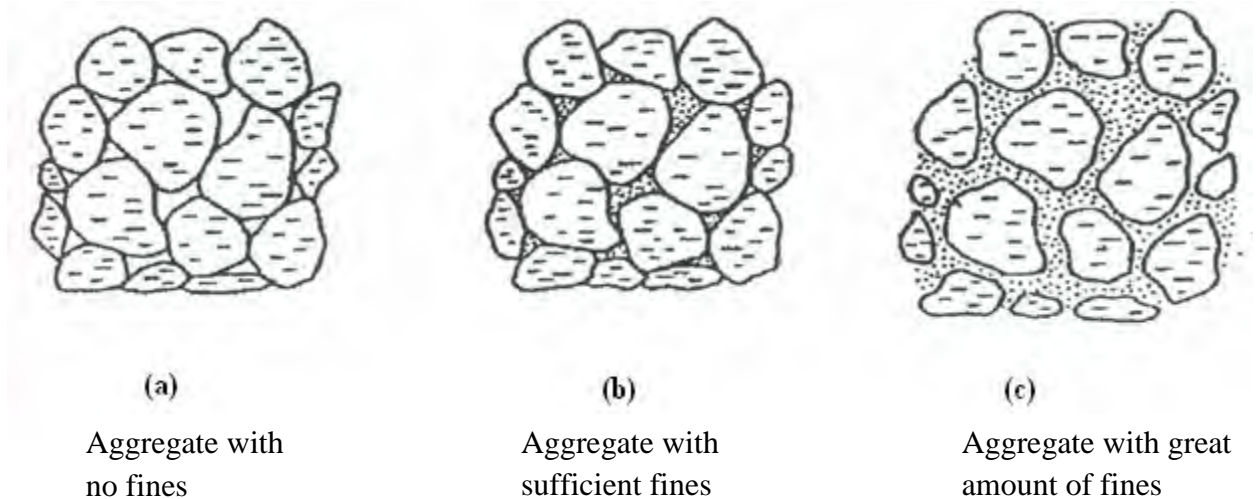


Figure 2.2: Physical States of Soil Aggregates Mixture

An aggregate that contains sufficient fines to fill all voids between the aggregate grains will still gain its strength from grain-to-grain contact but has increased shear resistance as shown in figure 2.2b. Its density is high and its permeability is low. This material is moderately difficult to compact but is ideal from the standpoint of stability. As shown in figure 2.2c, material that contains a great amount of fines has no grain-to-grain contact and the aggregate merely ‘float’ in the soil. Its density is low; it is practically impervious and it is frost susceptible. In addition, the stability of this type of material is greatly affected by adverse water conditions. Paradoxically, the material at times is quite easy to handle during construction and compacts quite readily [22].

2.3.1.2 Proportioning

Mixtures are difficult to design and build satisfactorily without laboratory control. A rough estimate of the proper proportions of available soils in the field is possible and depends on manual and visual inspection. Several trial mixtures should be made until this consistency is obtained. The proportion of each of the two soils should be carefully noted [4].

There are two methods of proportioning. These are:

- Graphical Proportioning
- Arithmetical Proportioning

Graphical Proportioning:

In this method, the actual gradations of the two soils to be blended (soil A & B, Figure 2.3) are plotted along the left and right axes of the graph, respectively. Once plotted, a line is drawn across the graph, connecting the percent passing of material A with the percent passing of material B for each sieve size. The shaded area of the chart represents the combinations of the two materials that will meet the specified gradation requirements [4]. The boundary on the left line is fixed by the upper limit of the requirement relating to the material passing the Number 200 sieve and the right line is established by the lower limit of the requirement relative to the fraction passing the Number 40 sieve.

The graphical method eliminates the need for precise blending under field conditions and the methodology requires less effort to use, its drawback is that it becomes very complex when blending more than two soils [4].

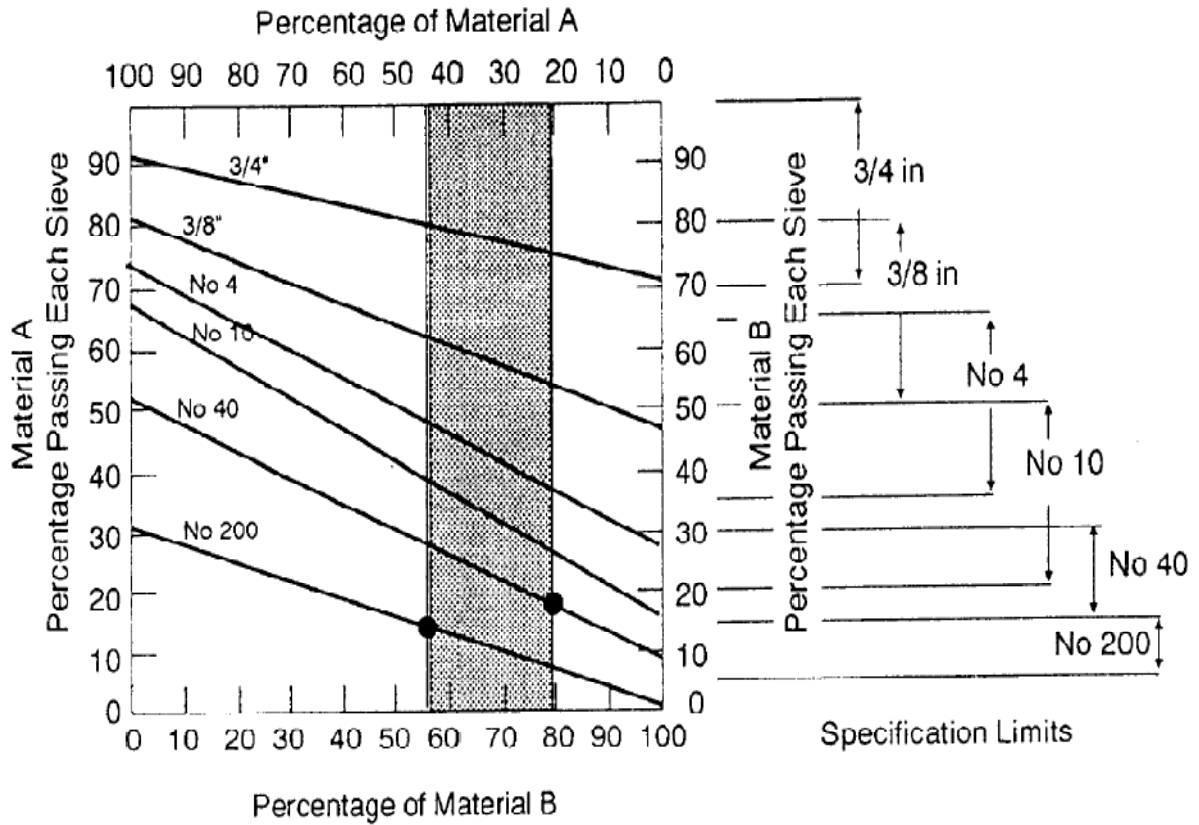


Figure 2.3: Graphical method of proportioning two soils to meet gradation requirements

Arithmetical Proportioning: In this method also, the actual gradation of soils A and B in their respective columns are recorded, average the gradation limits and record in the column labeled "S". Next, determine the absolute value of S-A and S-B for each sieve size and record in the columns labeled "(S-A)" and "(S-B)", respectively. Finally, sum columns (S-A) and (S-B) to determine $\sum A$ and $\sum B$. To determine the percent of soil A in the final mix, use the formula [4]:

$$\% A = \frac{\sum B}{\sum A + \sum B} \times 100 \quad \dots\dots\dots 2.1$$

And for soil B:

$$\% B = \frac{\sum A}{\sum A + \sum B} \times 100 \quad \dots\dots\dots 2.2$$

2.3.2 Chemical admixture stabilization

Chemical admixtures are often used to stabilize soils when mechanical methods of stabilization are inadequate and replacing an undesirable soil with a desirable soil is not possible or is too costly. Over 90 percent of all chemical admixture stabilization projects use cement, lime, fly ash & bituminous material [4, 9, 15, 17].

2.4 Related Works on Cinder Gravel in Road Construction

2.4.1 Experimental use of cinder gravels for roads in Ethiopia

As part of the joint research project between ERA and TRRL, a full scale experiment has been carried out in Ethiopia to examine the performance of volcanic cinder gravels as surfacing material for unpaved roads and as road base under bituminous surfaced roads [6]. Compaction trials were carried out to determine the type of equipment to be used and an experimental road stretch comprising 20 different sections were constructed. Six sections were left unsurfaced and were monitored for 28 months during which they carried approximately 140,000 vehicles. A bitumen surface was provided for the remaining 14 sections and these carried 150-200 vehicles per day for 7 1/2 years giving a total of 440,000 equivalent single axles in one direction. Monitoring was carried out by taking quantitative measurements of the performance of the road pavement throughout this period. As a result of the study, recommendations were made for the use of cinders in both paved and unpaved roads. For unpaved roads, recommendations are made for a particle size distribution which provides a road surface that is resistant to corrugations. Improved performance can be obtained by mechanically stabilizing cinders with plastic fines. For paved roads, it is concluded that the types of materials used in this experiment are all capable of carrying in excess of 400,000 equivalent single axles when sealed with a surface dressing and designed. Road mixed asphalt is not a suitable surfacing for cinder gravels. In addition to the cinders, other materials also performed satisfactorily including dry bound macadam, agglomerate and tuff. Besides it was recommended that cinders are easier to compact when they are mechanically stabilized with 10 per cent of volcanic ash soil [6].

Preliminary investigation of cinder gravels by TRRL

The main conclusions from the preliminary investigation of cinder gravels by TRRL which covered a field survey, a laboratory study and an examination of a cinder gravel road, are given below [16]:-

- (i) Cinder gravels are more widespread in Ethiopia than was originally believed; this showed the value of using aerial photographs in survey work and enabled a preliminary map to be prepared giving the distribution of cinder cones.
- (ii) In order to obtain representative material from a cinder cone, it is important that samples are taken from below the weathered zone, which can extend to a depth of two meters.
- (iii) Although 'as dug' cinder gravels do not meet the recommended grading requirements for road base materials, the laboratory investigation revealed that, because of the weak nature of the aggregate particles, breakdown under compaction occurred with an improvement in both grading and strength properties.
- (iv) In the laboratory investigation, the cinder gravels were not affected by changes in moisture and even complete immersion in water only reduced their strength slightly.
- (v) The addition of locally available plastic volcanic ash soil, to make up for the deficiency of fine material in the grading, improved the mechanical stability of cinder gravels and indicated that this could be a valuable construction practice. However, unlike the natural cinders the mixed materials lost some of their strength when they were saturated with water.
- (vi) The gravel road study confirmed that an improvement in the grading and the strength of cinder gravels occurred under normal road conditions even when trafficking was used as the means of compaction.

The results from the preliminary investigations indicated that cinders could provide useful road construction materials especially for gravel roads. However, it was necessary to carry out further

work under known conditions of traffic and climate in bituminous surfaced roads, as well as in gravel roads, before limits could be recommended for their various uses. It was therefore decided to construct pilot scale compaction trials and then a full-scale road experiment to examine these aspects further.

2.4.2 Stabilization of cinder by means of bitumen or cement

Given the fact that cinder is a troublesome material, Efreem [8] conducted investigations to determine whether or not the characteristics could be enhanced by stabilizing cinder with foam + 1% cement or with cement [8]. The cinder was retrieved from the Addis Ababa – Awash and Nazareth – Assela roads.

Based on this study [8], it was suggested that in general cinder is not suited to act as a surfacing material for unsurfaced roads. This is because it lacks fines that act as a binder. Nevertheless it is used in Ethiopia as a surfacing material but those cinder roads are affected by corrugation and rutting in the track wheel as well as a high gravel loss.

Cinder is not considered to be suited as a base course material because of its lack of crushing resistance [8]. In the late 1970's, experiments were done in Ethiopia to use cinder as a base course covered with a double surface dressing or a 50 mm thick asphalt layer. It was shown that compaction with a vibratory roller together with a rubber tyre roller produced the best compaction results. The test sections carried 80 – 100 commercial vehicles per day and were designed to carry 0.64 million 80 kN equivalent single axles for a period of 15 years. After 2½ years in service, the test sections were still in good condition although significant crushing of the coarse particles was observed. In late 1999, however the sections were severely damaged by fatigue cracking and for some part the wearing course was totally worn out. The adjacent sections constructed with a conventional crushed stone base were still in a good condition except for some local potholes [8].

In conclusion one can say that one has to be careful with using cinder as surfacing or as base material. In order to be able to use it as a base, modification of the cinder by means of foam bitumen or cement might be a feasible option [8].

2.4.3 Stabilizing cinder gravels for heavily trafficked base course

Investigation into the improvement of natural gravels with the use of stabilization techniques was made using samples collected from quarry sites near Alemgena and Lake Chamo [11]. Mechanical and cement stabilization were investigated in two subsequent phases. In the first phase, optimum amount of fine soils that makes up the deficiency of the fine particles of natural cinder gravels was found to be 12%. In the second phase, natural cinder gravel sample without, and with 12% fine soils were stabilized with 3, 5, 7, and 10% of cement by mass. The result of investigation indicated that the optimum amount of cement required to achieve the minimum UCS of 3.0 MPa as specified in ERA and AACRA pavement design standard for heavily trafficked base course without adding fine soil is found to be 7% cement. However, this high cement requirement was reduced to 5% cement which is practical value by mechanically stabilizing cinder gravel with 12% of fine soils before cement stabilization. Nevertheless, it was recommends that the performance of cement stabilized cinder gravel should be investigated in a full-scale road experiment against cracking due to stresses induced by thermal, shrinkage and traffic [11].

2.5 Economical Aspects of Blending of Material

Economic necessity and the diversity of physical and climatic conditions require highway engineers to make the best use of available natural resources in developing countries, where the economic adaptation of a road to special needs and its technical adaptation to local conditions are two complementary aspects which greatly influence planning decisions. The excavation, haulage, and laying of satisfactory pavement materials must be accomplished as economically as possible for all highway projects. However, in developing countries, where a high percentage of the roads to be built and maintained are primarily unsurfaced and involve a gravel placement, investment in materials is normally higher. Therefore, in order to achieve the most cost-effective construction and lower subsequent maintenance costs, it is necessary for haulage distances, which form the major item of expense, to be minimized by making the best use of locally available materials [7].

Throughout many countries in the developing world locally occurring materials are often not used as extensively as would be possible due to the adaptation of specifications for road building materials that have been derived in industrialized countries with temperate climates and different geology.

When climatic conditions are favorable, more heavily trafficked roads may also be constructed if the type of material is carefully selected and controlled. Recent increases in the costs of road construction have further emphasized the need to make optimum use of these locally occurring resources, not only to reduce expenditure, but also to conserve those better quality materials which are frequently in short supply and of value for special projects that require construction to high specifications (e.g. airfields) [7].

There are very large potential savings indeed to be made from the implementation of research findings from work done by TRRL and others on design standards for low volume secondary and feeder roads. For example, in Malawi, the provision of a crushed stone base, which is often the preferred design option, is generally around 15% of the total project costs. The difference between the cost of 1km of crushed stone base and 1km of locally available lateritic gravel is at least 4: 1 [14]. Substantial savings on the cost of construction of low volume roads can be achieved if these locally abundant gravels can be utilized for the base construction. Where sources of crushed stone are not readily available, which is the case in many areas in southern Africa the difference in costs between crushed stone and the natural gravel base is much higher than the 4:1 quoted [14].

The use of recycled concrete materials for capping and road sub-base can be economically advantageous when compared with the use of primary materials for the same purposes. Cost comparisons with primary aggregates suggest that savings of 20 - 30% are possible [5]. These savings vary depending upon the geographical location and transport requirements [5].

In some countries, crushed concrete in defined circumstances can be used as a substitute material to primary material often with economic advantage. Such material is available from demolition operations and can be suitably prepared by appropriate processes such as screening, crushing,

grading and testing. The use of crushed concrete can offer benefits cost savings, primary resource conservation and transport impact reduction.

With correct selection and processing the material can have a performance that can match or even exceed the properties of conventional primary materials to strengthen foundation soils. There has been a limited use of the material in the construction of roads and paved areas over many years [5].

If the appropriate sub-base material cannot be found in areas close to the construction site, then very high prices have to be paid in this type of road construction process, which causes significant delays or cost increases. In such cases, sometimes low-quality materials are used which affects the road quality and durability over time and results in very significant losses. Therefore, improving the quality of materials is very important for road construction works, in order to ensure that projects meet the necessary cost and quality criteria.

CHAPTER 3

GENERAL DESCRIPTION OF THE STUDY AREA

3.1 Guraghe Zone

Guraghe zone has a land size of about 5,932 square kilometers and consists of 13 woredas. Namely: Abeshegae, Qebena, Ezia, Kokir -Gedebano, Sodo, Meskan, Mareko, Gumer, Cheha, Enemore, Aenere, Muhur- Aklile and Endegagn. Topographically the zone lies within an elevation ranging from 1000 to 3600 meters above sea level. The total population of the zone in 1996 E.C. was estimated about 1,533,279, which brings the population density to reach 258 inhabitants per square kilometer. The zone has three agro-ecological zones dega (35%), wenadega (62%) and kolla (3%). The annual average temperature of the zone ranges from 13⁰ C to 30⁰ C and the mean annual rainfall ranges from 600-1600 mm [2].

The research was conducted as a case study on the Butajira – Gubre Road which is 87 km long and starts at Butajira station. This asphalt road is under construction in the region which connected different woredas such as Meskan woreda, Cheha woreda, Gubre town, etc.

3.2 Climatic Condition around Butajira Area

Butajira is an Administrative center of Meskan Woreda. It is located 135km from Addis Ababa to the south via Addis Ababa-Sodo road and 50 km west of Ziway in the Rift Valley, with a latitude 8.2°N and longitude 38.5°E. The climate condition varies from arid lowland areas at altitudes of around 1500 m above sea level (tropical climate) to cool mountainous areas of up to 3500 m above sea level (temperate climate). The main wet season occurs between June and October, with the remaining months predominantly dry. Daytime temperatures are typically 20–30°C, with night time temperatures falling close to freezing at higher altitudes. The lowland areas are drought prone and have been affected during the main droughts in Ethiopia [2].

The recent data values of the temperature and average annual rainfall of Meskan Woreda is [13]:

Annual average high temperature: 24.0 °C

Annual average low temperature: 10.3 °C

Average temperature: 17.2 °C

Average annual precipitation: 1062mm

3.3 Proposed Material Sources

An important prerequisite for selecting the best of the locally available materials for road construction is knowledge of their properties and behavior and how these vary in time and space. Although some materials may perform satisfactorily in a road pavement, though not conforming to normally accepted selection criteria, the opposite may also apply where the results are just as unpredictable.

The samples in this study were collected around Butajira area. The available cinder gravels are located 10 km (GPS indicated N= 8982274.24, E= 436190.454) far from Butajira and the fine-grained soil of the blending materials are located 3km (GPS indicated N= 896355.629, E= 428169.729) far from Butajira town. The fines borrow materials are also used in the sub-grade construction of the road. The status of the project, when this study was conducted, was under construction of the sub-base layer (See Figure 3.4). The blended cinder gravels with fine-grained soils being used to construct the sub- base layer is without any specified proportion of the blended sub-base materials.



Figure 3.1: Location of Butajira area



Figure 3.2: Material site for visually yellowish silty soils



Figure 3.3: Material site for cinder gravel



Figure 3.4: Sub-base layer under construction

CHAPTER 4

LABORATORY TEST RESULTS AND DISCUSSIONS

In this section a description of the testing procedures followed as a part of this research will be discussed. Standard procedures of AASHTO and ASTM have been used wherever possible.

4.1 Material Used

Road construction materials are an important component of pavement design. The selection of appropriate quality of materials for selected sub-base layer affects determination of the capital and whole life costs of the road. The use of locally available materials is very much encouraged in rural road engineering. As part of this research, fine-grained soil obtained near the project site was blended with cinder gravel samples in order to make up for the deficiency of fine materials. For the borrow material that is used in this research the whole descriptive test results are shown below.

Table 4.1: Descriptive test results of fine-grained soil used for mechanical stabilization

Type of test	Test results
Gradation : % passing	
9.5 mm	100
4.75 mm	97.79
2.0 mm	92.24
0.425 mm	81.81
0.075 mm	55.18
Liquid Limit	38 %
Plastic Limit	26 %

Plasticity Index	12 %
Specific Gravity	2.5
Free well	35 %
CBR %	18

The fine-grained soils obtained near Butajira are visually classified as yellowish silty and based on the laboratory test results and AASHTO classification; it is classified as clay with in medium plasticity (A-6). These materials were also used in the embankment and sub-grade road construction in the Butajira-Gubre road project.

The cinder gravel is classified as A-1-a material since in gradation test of cinder gravel less than 15 % passes through No.200 sieve, less than 30 % passes through No.40 sieve and less than 50 % passes through No.10 sieve and its PI is less than 6%. The cinder gravel is a weak material and has a high water absorption capacity because of high porosity.

4.2 Laboratory Test Results of Blended Cinder

4.2.1 Index property tests

Particle size distribution and gradation

This method is used primarily to determine the grading of materials proposed for use as aggregates or being used as aggregates. The results are used to determine compliance of particle size distribution with applicable specification requirements and to provide necessary data for control of the production of various aggregate products and mixtures containing aggregates.

A key property of aggregate used for highway bases and surfaces is distribution of particles sizes in the aggregate mix. The gradation of cinder gravel, that is, the blend of the particle sizes in the mix, affects the density, strength, and economy of the pavement structure.

A grain size analysis is used to determine the relative proportions of various particle sizes in cinder gravels. To do this analysis, a wet preparation method is performed which is given in AASHTO T-146 and weighed sample of dried aggregate is shaken over a nest of sieves having selected sizes of square openings. The cinder sample is shaken with a mechanical sieve shaker, and the weight of material retained on each sieve is determined and expressed as a percentage of the original sample. Detailed procedures for performing a grain size analysis of coarse and fine aggregates are given in AASHTO Method T-27 [1].

According to the ERA grading limit, the material shall have a smooth continuous grading within the limits for grading A, B or C and the sub-base material shall comply with one of the grading given below in Table 4.2.

Table 4.2: Ethiopia Road Authority grading limits for sub-base material

SIEVE SIZE(mm)	Mass Percent Passing			
	A	B	C	D
63	100			
50	90 – 100	100	100	
37.5			80 – 100	
25	51 – 80	55 – 85		100
20			60 – 100	
9.5		40 – 70		51 – 85
5			30 – 100	
4.75	35 – 70	30 – 60		35 – 65
2.0		20 – 51		25 – 51
1.18			17 – 75	
0.425		10 – 30		15 -30

0.3			9 – 50	
0.075	5 – 15	5 – 15	5 – 25	5 – 15

The complete sub-base shall contain no material having a maximum dimension exceeding two-third of the compacted layer thickness.

Among the grading limits, grade B has almost a complete specification of percent mass passing for each sieve size. Moreover, it has a practical implementation as practiced by AACRA. Taking these advantages this thesis work is made based on grading B specification.

The Specification of grain size analysis of grade B data is plotted on an aggregate grading chart, exemplified by Figure 4.1.

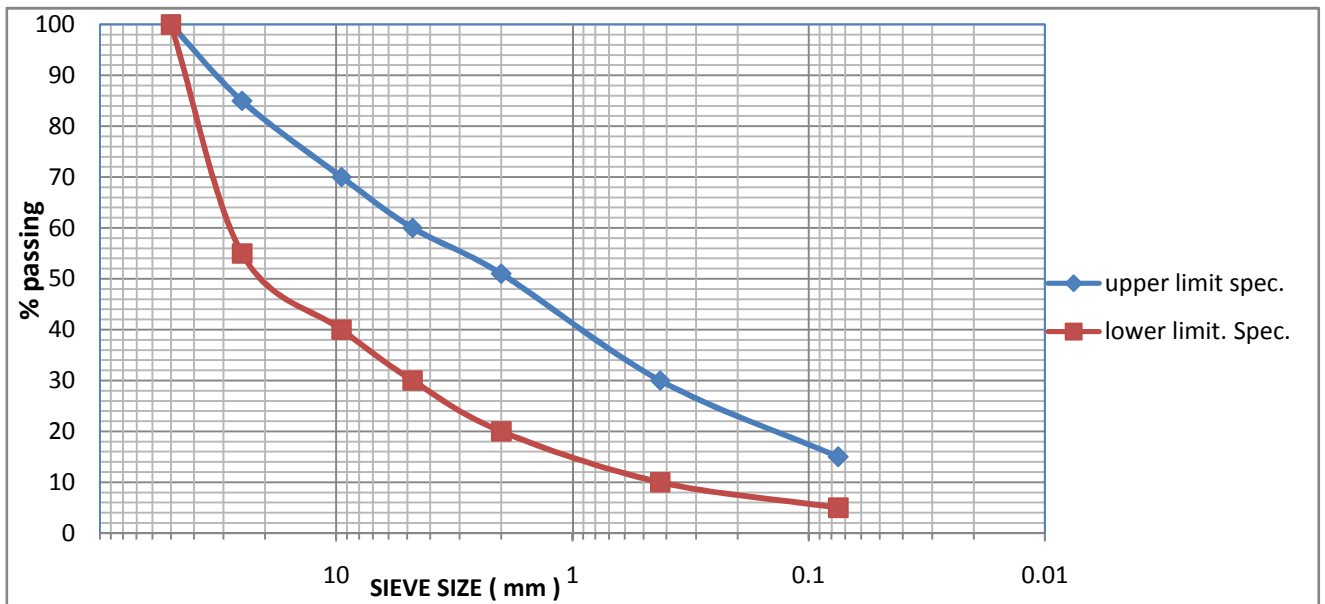


Figure 4.1: ERA Specification grade limits for grade B

Figures 4.2 to 4.7 show gradation before and after compaction of cinder gravel only and cinder gravel blended with different proportion of fine-grained soils.

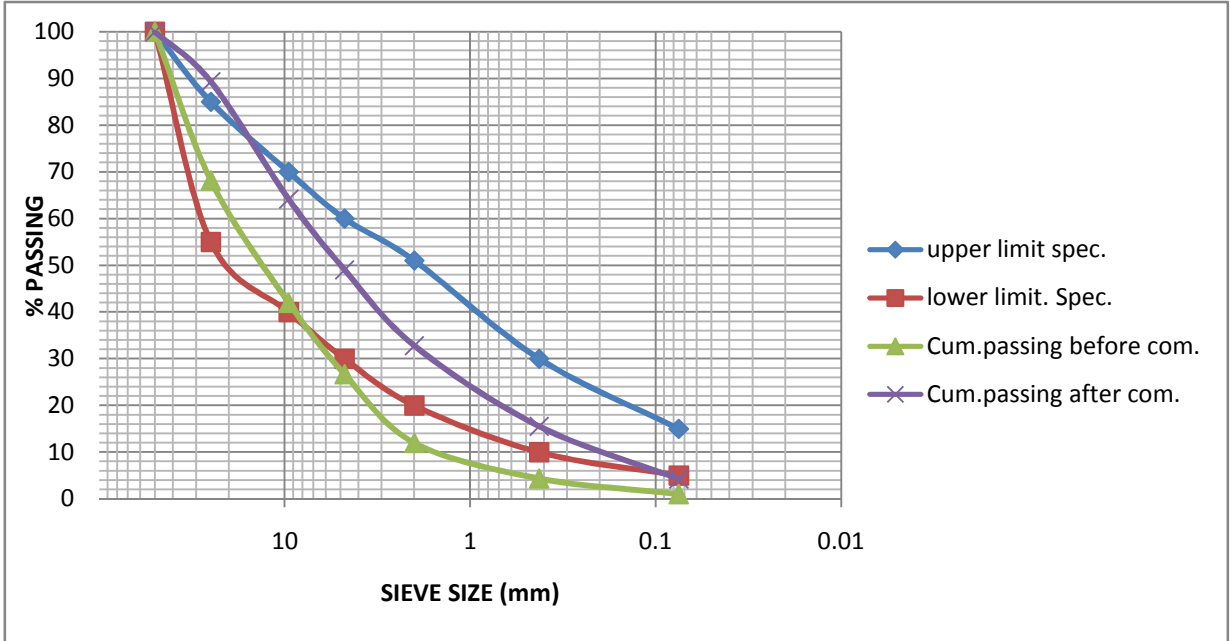


Figure 4.2: Gradation before and after compaction of natural cinder gravel only

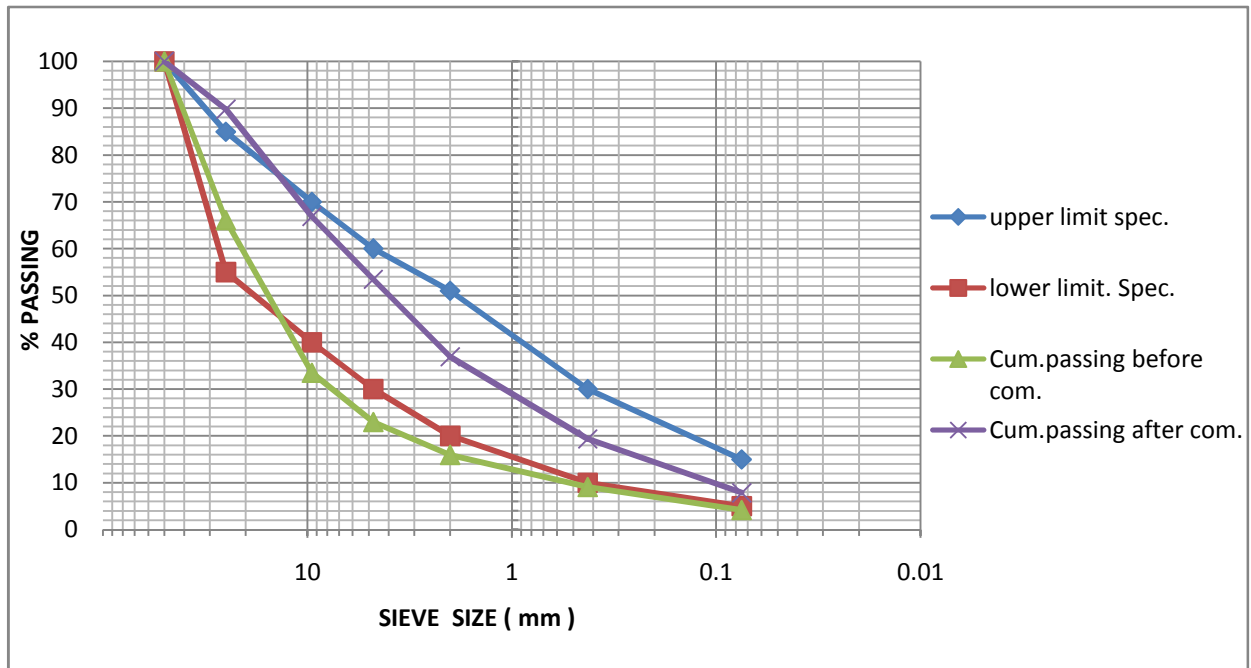


Figure 4.3: Gradation before and after compaction of cinder with 5 % fine-grained soil

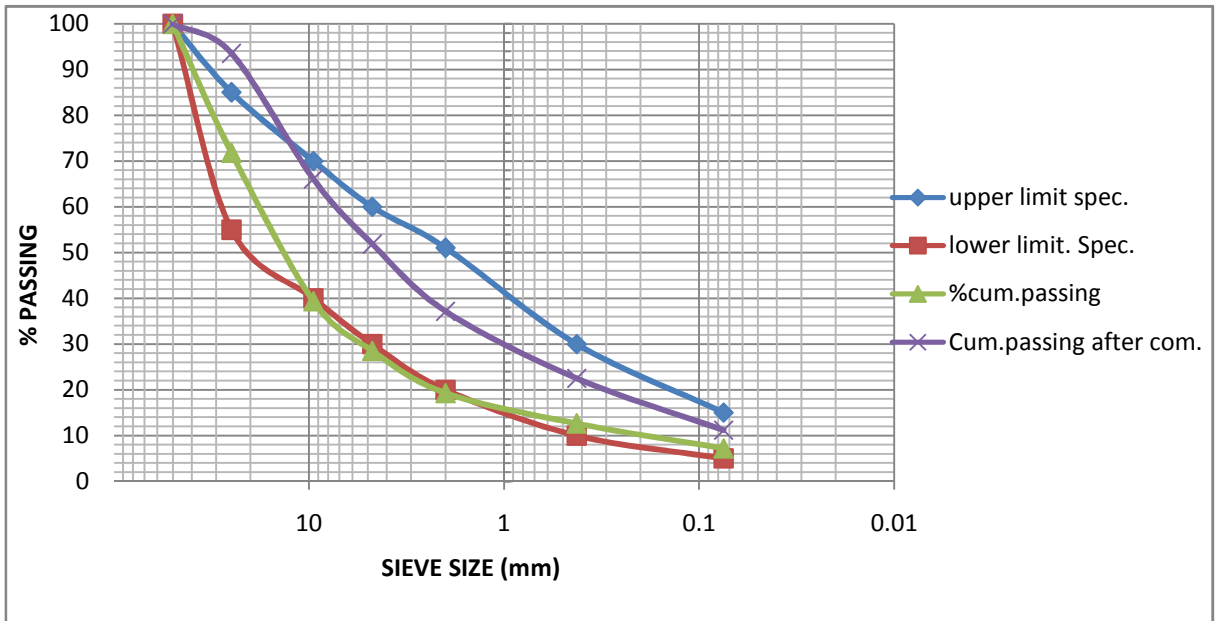


Figure 4.4: Gradation before and after compaction of cinder with 10 % fine-grained soil

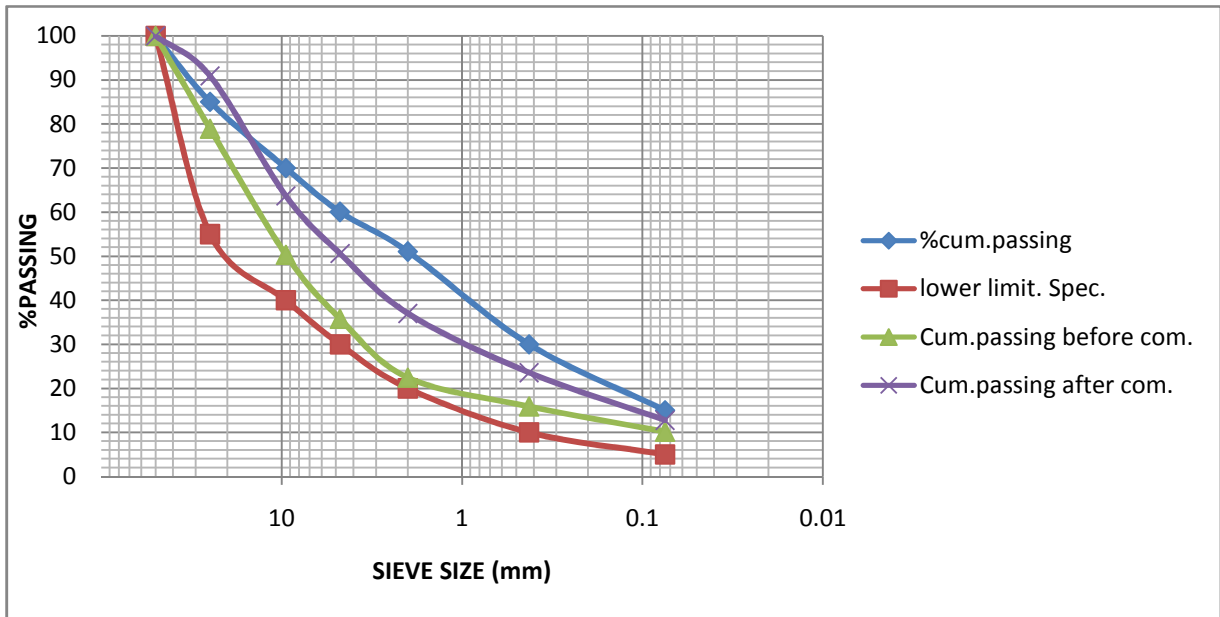


Figure 4.5: Gradation before and after compaction of cinder with 15 % fine-grained soil

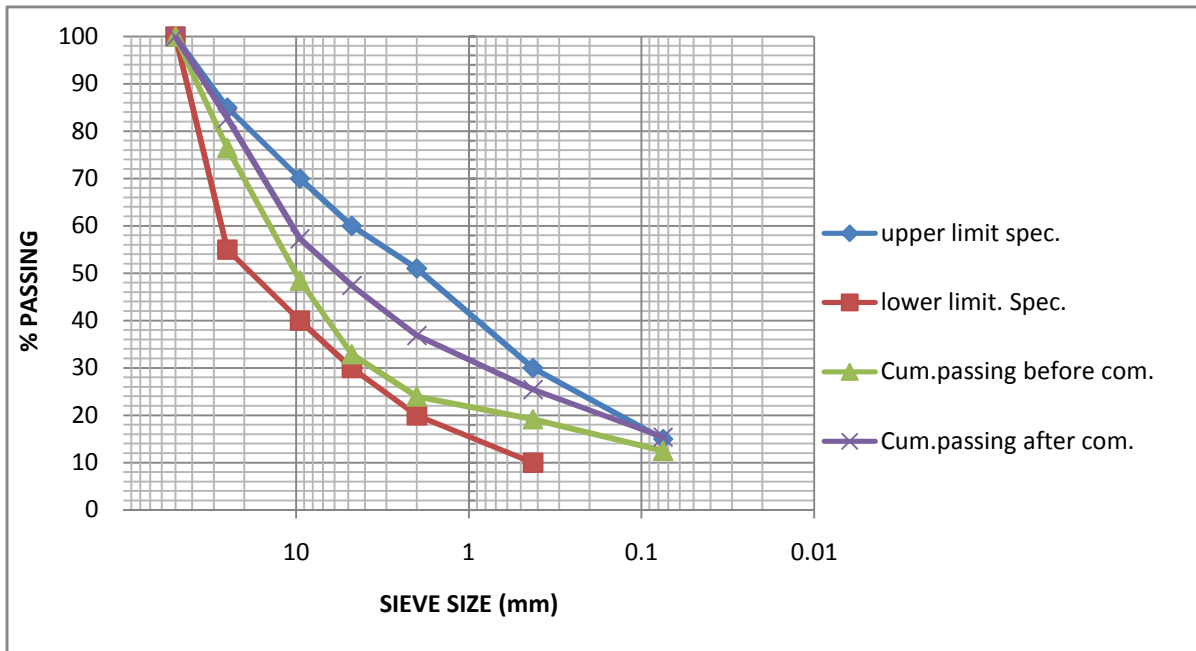


Figure 4.6: Gradation before and after compaction of cinder with 20 % fine-grained soil

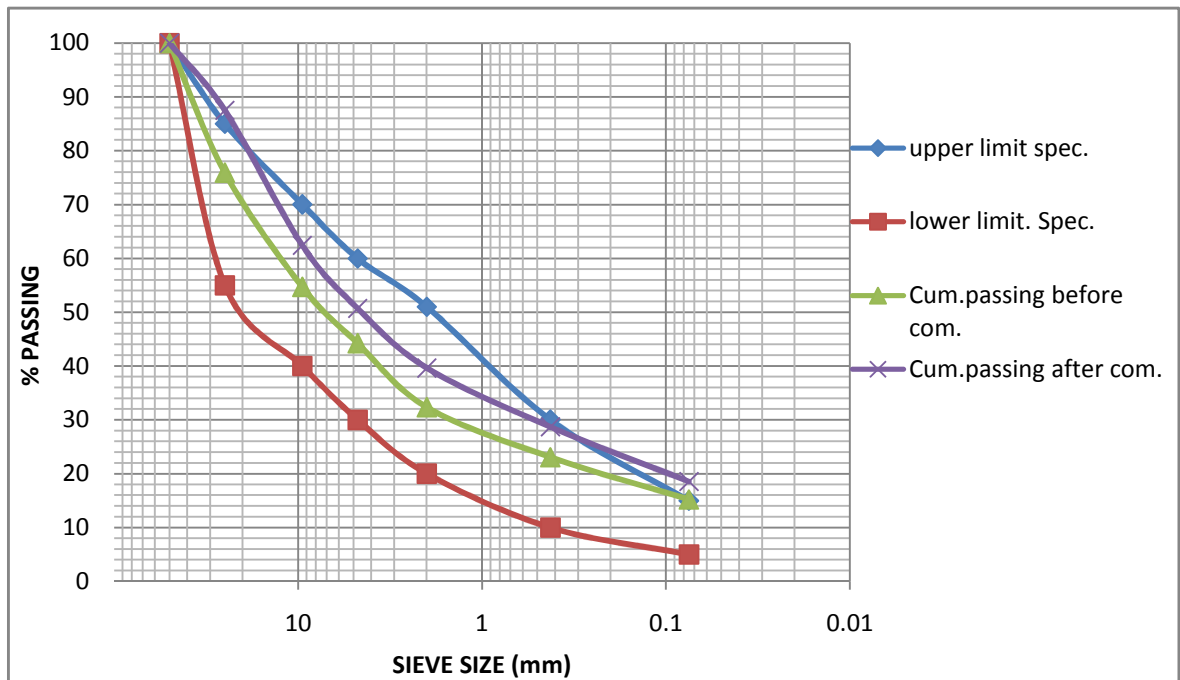


Figure 4.7: Gradation before and after compaction of cinder with 25 % fine-grained soil

The grading chart of natural cinder gravel in Figure 4.2 shows the deficiency in fine particle. Therefore, it is out of the specification. Blending of a trial proportion of fine-grained soil improves the gradation and the optimum proportioning is found to fulfill the ERA specification manual requirement. Among the trial proportioning of fine-grained soil 20 % and 15 % are good proportioning from gradation point of view.

From figures 4.2 to 4.7 also clearly shows that the modified AASHTO compaction method resulted in the largest amount of crushing. The laboratory investigation shows that because of the weak nature of cinder gravels, break down under compaction occurred with an improvement of gradation but still needs an additional fine-grained soil to achieve the ERA specification.

For construction purposes, optimum amount of fine-grained soils are required [12]. This is needed to:

- i. Ensure that the finished granular surface is stable enough to carry the paving equipment without disruption of the surface.
- ii. Ensure that the material can be compacted adequately. Some fines are required so that when the sub-base is compacted, the granular material will bind and the particles will inter lock. Other wise they would shear when loaded.

If a material has a continuous grading following the full curve, then the voids between the larger particles are filled with fine material resulting in a dense packing. This immediately results in a larger number of contact points between the particles. The resistance to shear of the skeleton depends on the sum of the frictional forces that are generated between the particles. When the number of contact points increases, also the total friction in the skeleton increases which in turn results into a higher resistance to shear.

The arithmetic proportioning as discussed in section 2.3.1.2 is a rough approximation to determine the proportions of the blending material. It can serve as a reference to make a quick blending without making many trial proportioning tests. It minimizes the number of trial proportioning tests. This method is used in this study to preliminary estimate the optimum proportions of cinder gravel and fine-grained soil.

Table 4.3 below is tabulated based on the gradation of cinder gravel and fine-grained soil before compaction. Therefore, arithmetically the optimum proportion is calculated to be 78% cinder gravel and 22% fine-grained soil.

Table 4.3: Determination of the proportions of cinder and fine-grained soil

Sieve size(mm)	Percent passing				$ \bar{S}-A $	$ \bar{S}-B $	0.78A (4)	0.22B (5)	Percent passing final mix (4 + 5)
	Cinder(A) (1)	Fine-grained soil(B) (2)	Specs(S)	\bar{s} (3)					
50	100	100	100	100	0	0	78	22	100
25	68.11	100	55-85	70	1.89	30	53.12	22	75.12
9.5	41.95	100	40-70	55	13.05	45	32.72	22	54.72
4.75	26.68	97.79	30-60	45	18.32	52.79	20.9	21.51	42.41
2	11.98	92.24	20-51	35.5	23.52	56.74	9.344	20.29	29.63
0.425	4.34	81.81	10-30	20	15.66	61.81	3.385	18	21.38
0.075	0.98	55.18	5-15	10	9.02	45.18	0.764	12.13	12.89
					$\sum A$ 81.46	$\sum B$ 291.52			

Grading Modules (GM)

Grading Modules (GM) is the cumulative percentages by mass of material in a representative sample of aggregate, gravel or soil retained on the 2.00 mm, 0.425 mm and 0.075 mm sieves, divided by 100.

According to ERA specification, the minimum Grading Modulus shall be 1.5 except where a material, having a lower Grading Modulus but not less than 1.2, is approved for use by the Engineer [10, 19].

In table below the grading modules is calculated for different proportions of cinder gravel and fine-grained soil.

Table 4.4: Determination of Grading Modules values

Percent of cinder gravel	Percent of fine-grained Soil	GM
100	0	2.83
95	5	2.71
90	10	2.61
85	15	2.51
80	20	2.44
75	25	2.29

The particle distribution of the finer material passing the 2 mm sieve has a marked effect on the compact ability and bearing strength of the material.

Table 4.4 and figure 4.8 show that the grading modules of all the blending proportions lie within ERA specification manual requirements.

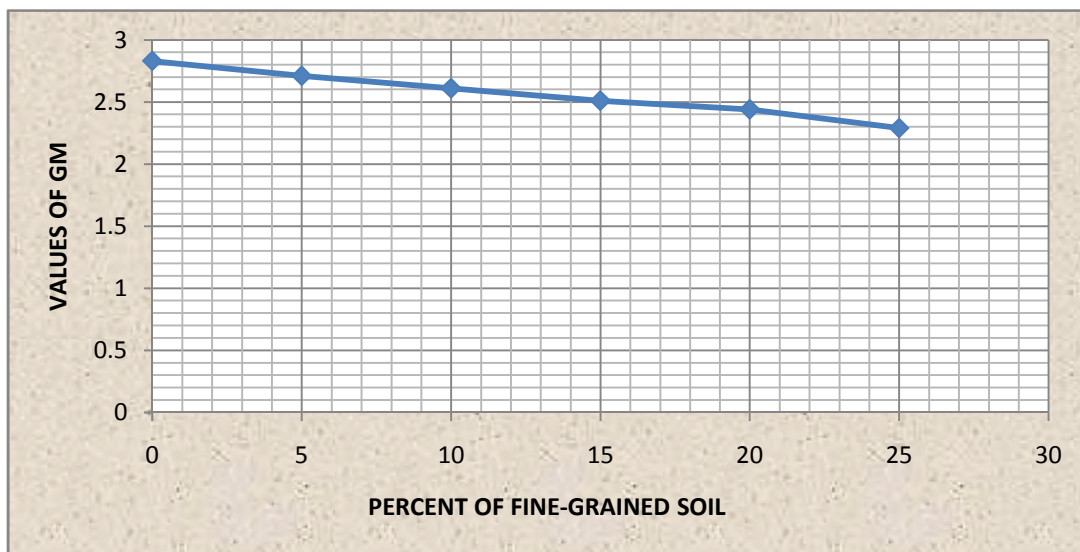


Figure 4.8: The Grading Modules with different proportion of fine-grained soil

The Grading Modules for all mix proportions is found to be above the minimum required value of 1.5.

Atterberg Limits

The liquid limit may be defined as the minimum moisture content at which the soil will flow under the application of a very small shear force. At this moisture content the soil is assumed to behave practically as a liquid. The plasticity limit may be defined in general terms, as the minimum moisture content at which the soil remains in plastic condition. The plastic limit is further described as the lowest moisture content at which the soils can be rolled in to thread of 3.2mm diameter without crumbling. The “Plasticity index” (PI) of a soil is defined as the numerical difference between the liquid and plastic limits. It thus indicates the range of moisture content over which the soil is in a plastic condition.

Plasticity is an important factor in the performance of a gravel wearing course for the following reasons. Material with plasticity that is too low tends to loosen quickly as a result of diminished bonding and the rate of gravel loses is generally very high. Loose material is pushed off into the drains or washed away by run-off or blown away by wind when dry. High gravel lose reduces re-gravelling cycle periods causing high maintenance cost and general whole life costs. High plasticity on the other hand causes the wearing course to be slippery when wet and the material may soften to an extent where the gravel layer may actually deform and fail instantly under traffic.

According to ERA specification, all sub-base materials shall have a maximum plasticity index of 6 when determined in accordance with AASHTO T-90. The plasticity product ($PP = PI \times$ percentage passing the 0.075mm sieve) shall not be greater than 75% [10, 19].

Table 4.5 shows that 0, 5,10,15,20 and 25 % of the blending proportion of material by mass with cinder has no plasticity characteristics and the plastic product is also zero. Therefore, all the proportions satisfy the ERA manual specification.

Table 4.5: Determination of plastic index and plastic product for blended material

Percent of fine-grained soil	Plastic index	Plasticity product
0	NP	Zero
5	NP	Zero
10	NP	Zero
15	NP	Zero
20	NP	Zero
25	NP	Zero

4.2.2 Moisture – Density relations by modified proctor test

Practically most soils exhibit a similar relationship between moisture content and density (dry unit weight) when subjected to dynamic compaction. That is, practically the cohesive soils have an optimum moisture content at which the soil attains maximum density under a given compacting effort but the granular soils difficult to define. This fact, which was first stated by R.R. Proctor in a series of articles published in Engineering News-Record in 1933, forms the basis for modern construction process commonly used in the formation of highway sub-grades, bases, embankments, and earthen dams. In laboratory, dynamic compaction is achieved by use of a freely falling weight on confined soil mass; in the field, similar compaction is secured through the use of rollers or vibratory compactors applied to relatively thin layers of soil during construction process. Compaction is a process by means of which the soil can be densified. In soils there is some amount of air and water besides solid grains. Theoretically the density of soil can be increased by:

- By reducing the space occupied by the air.
- By elastic compression of soil grains.

Compaction takes place due to expulsion of air from the voids of the soil mass by applying any mechanical means [7]. It is the process by which the solid particles are packed more closely together, usually by mechanical means, there by increasing the dry density of the soil. The dry density, which can be achieved, depends on the degree of compaction applied and on the amount of water present in the soil. For a given degree of compaction of a given cohesive soil there is an optimum moisture content at which the dry density obtained reaches a maximum value. For cohesion less soils optimum moisture content might be difficult to define.

The determination of the relationship between water content and density of soils is used in determining the compaction of the material. The purpose of compaction is to arrange the particles in such a way as to achieve the highest possible density for the layer with minimum voids. By achieving high densities, not only is the shear strength and elastic modulus improved but also the ingress of water is reduced or eliminated.

In this research, a heavily trafficked asphalt road was considered hence the modified proctor test is used. The Ethiopia Road Authority recommends using AASHTO T-180 method D. In this test, a specimen is prepared by compacting soil in 152.4 mm mold in five approximately equal layers to give a total compacted depth of about 127 mm, each layer being compacted by 56 uniformly distributed blows from the rammer.

Results of the compaction tests are summarized in table 4.6 and figure 4.9 below.

Table 4.6: Determination of MDD & OMC with different fine-grained soil proportions

Percent of fine-grained soil	MDD(g/cc)	OMC (%)
0	1.62	1.71
5	1.64	5.71
10	1.68	8.36
15	1.73	13.94
20	1.75	14.6
25	1.67	16.1

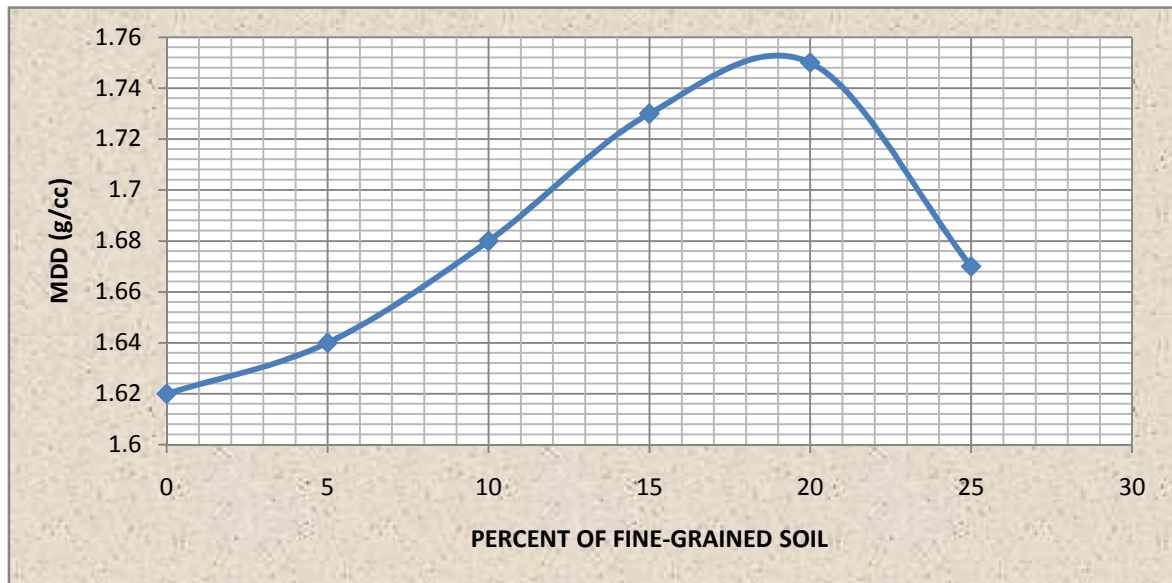


Figure 4.9: Variation of MDD with percentage of fine-grained soil mixed with cinder

Figure 4.9 shows that the maximum dry density increases up to 19 % of fine-grained soil proportion by mass and after that MDD goes down. This shows that the arithmetic method can be used as a good indicator, which result 22% of fine-grained soil.

The cinder gravel has little fine-grained soil content and gains its stability from grain-to-grain contact; consequently it usually has relatively low density. Adding fine-grained soil to the cinder gravel still gains its strength from grain-to-grain contact and leads to the increment of density up to an optimum point. The cinder gravel that contains optimum amount of fine-grained soil fills all the voids. This results in high density. Beyond this optimum fine-grained soil grain-to-grain contact gradually decreases leading to the decrement of density.

4.2.3 California Bearing Ratio (CBR) test

This test method covers the determination of the CBR of pavement sub-grade, sub-base, and base course materials from laboratory compacted specimens. The method uses soil particles that pass 19 mm size and provides CBR value of a material at optimum water content. The specimen shall be soaked prior to penetration. This test simulates the prospective actual condition at the surface of the sub-base. A surcharge is placed on the surface to represent the mass of pavement material above sub-base. The sample is soaked to simulate its weakest condition in the field. Expansion of the sample is measured during soaking to check for potential swelling.

This method covers the laboratory determination of the California Bearing Ratio (CBR) of a compacted or undisturbed sample of soil. The principle is to determine the relation between force and penetration when a cylindrical plunger with a standard cross-section area is made to penetrate the soil at a given rate. At certain values of penetration the ratio of the applied force to a standard force, expressed as a percentage, is defined as the California Bearing Ratio (CBR).

According to ERA manual, for the sub-base material the minimum soaked California Bearing Ratio (CBR) shall be 30% when determined in accordance with the requirements of AASHTO T-193. The Californian Bearing Ratio (CBR) shall be determined at a density of 95% of the maximum dry density when determined in accordance with the requirements of AASHTO T-180 method D [10, 19].

The determination of the CBR and percent swell of cinder gravel only are presented in tables 4.7 to 4.9 and figures 4.10 to 4.12 below.

Laboratory Test Result for Cinder Gravels Only

Table 4.7: Determination of dry density before & after soaking

BLOWS	BEFORE SOAKING		AFTER SOAKING	
	DD (g/cc)	MOISTURE (%)	DD (g/cc)	MOISTURE (%)
10	1.41	1.35	1.42	19.39
30	1.55	1.63	1.57	18.46
65	1.63	1.36	1.66	17.79

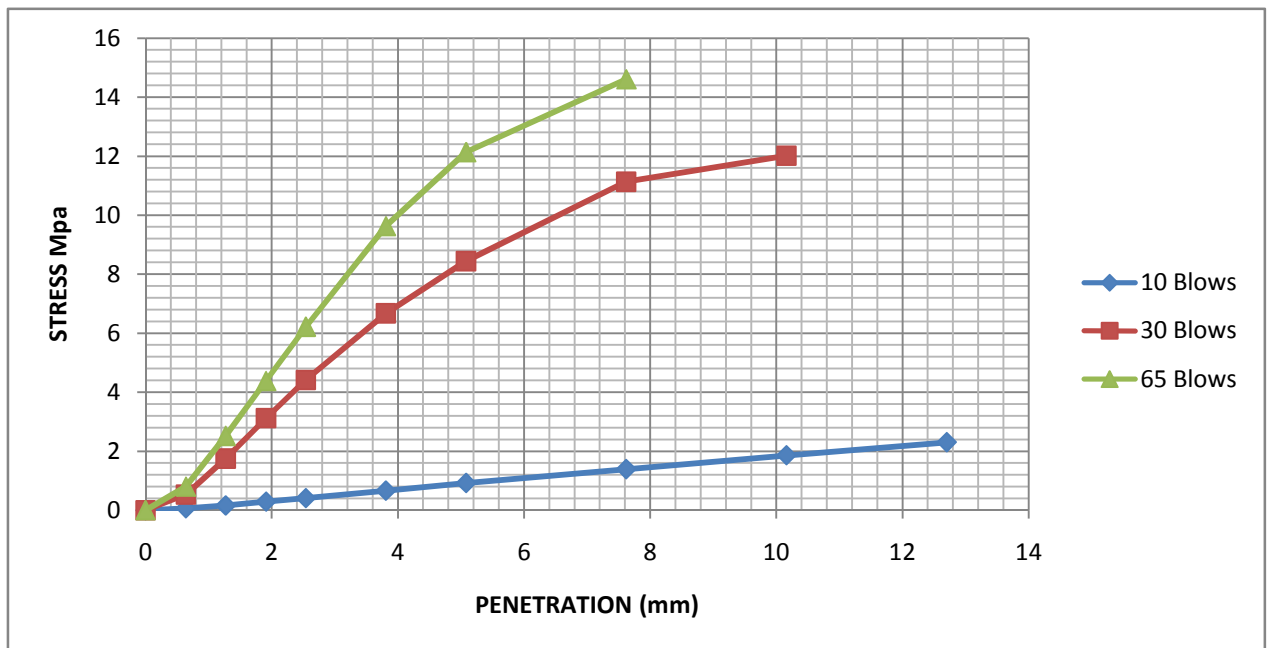


Figure 4.10: Load-penetration curves of cinder gravel only

Table 4.8: Determination of CBR & Swell (%)

BLOWS	STRESS (MPa)		CBR (%)		CBR %		SWELL (%)
	2.54mm	5.08mm	2.54	5.08	2.54	5.08	
10	0.41	0.92	5.6	8.95	5.94	8.93	0.18
30	4.41	8.44	64.2	82.0	63.9	81.94	0.026
65	6.22	12.14	89.5	117.4	90.1	117.9	0.01

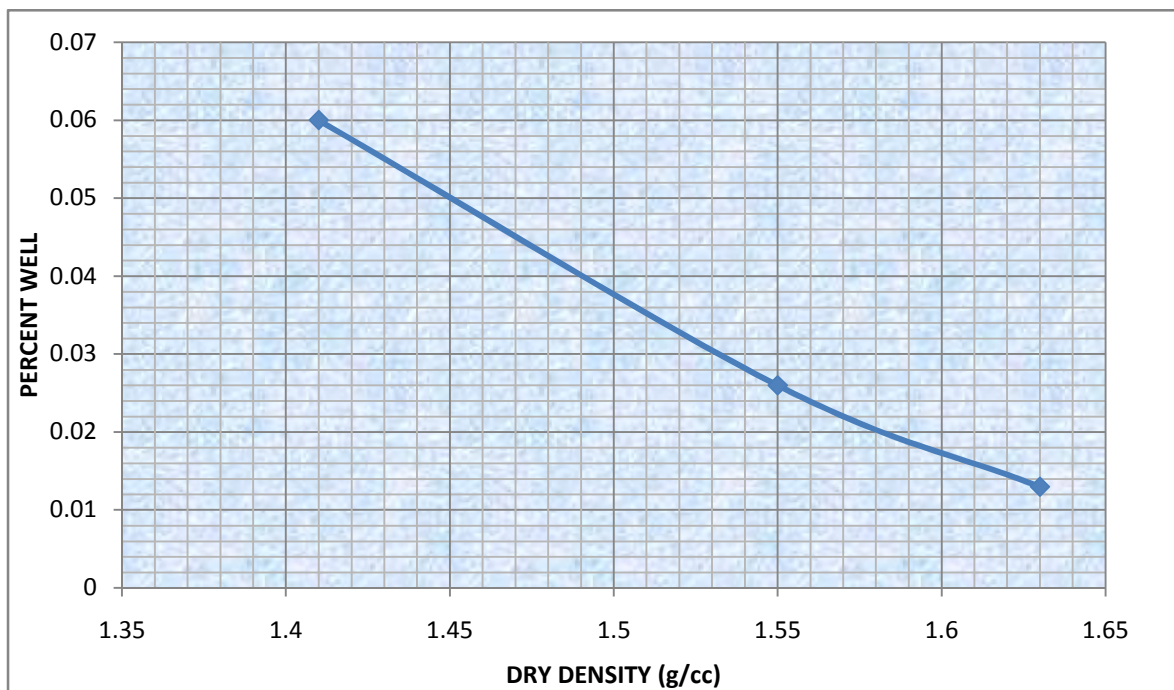


Figure 4.11: Dry density Vs percent swell

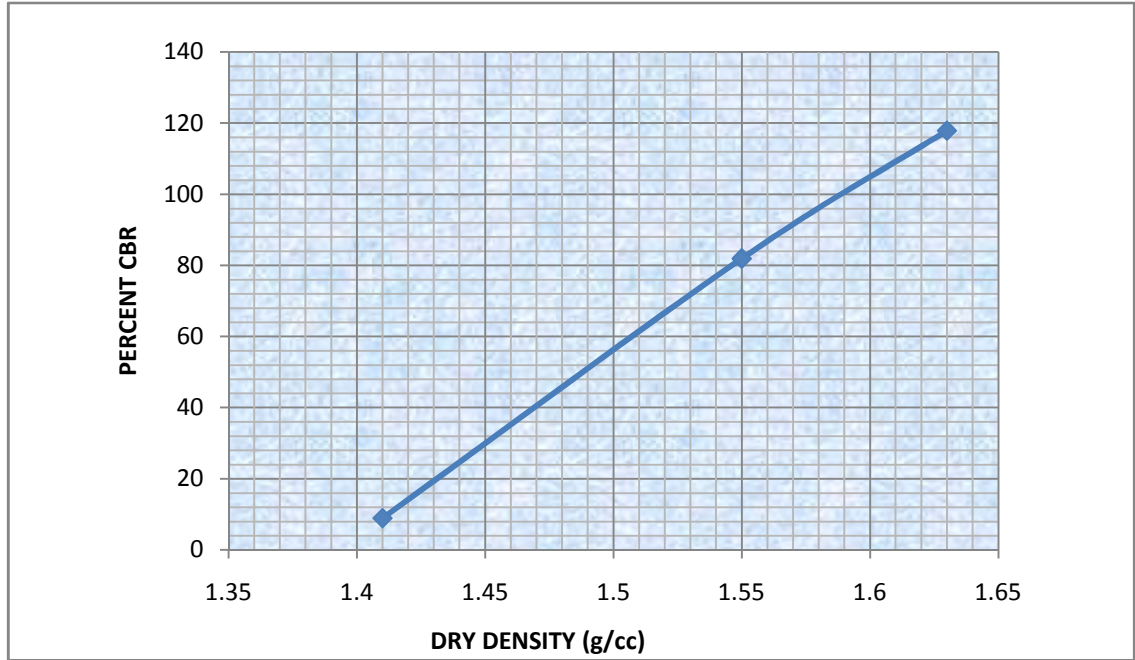


Figure 4.12: Dry Density Vs Percent CBR

Table 4.9: Determination of CBR & Swell at 95 % of MDD of cinder gravel only

BLOW	DRY DENSITY (g/cc)	CBR (%)	SWELL (%)
10	1.41	8.93	0.18
30	1.55	81.94	0.026
65	1.63	117.9	0.01
CBR AT 95% OF MDD		76 %	
SWELL AT 95% OF MDD		0.028 %	

To obtain the true stress-strain relationships in Figure 4.10 above, correct the curve having concave upward shape near the origin by adjusting location of origin by extending line portion of stress-strain curve downward until it intersect the abscissa.

Results of CBR and Swell percent values of the different mix proportions are given in table 4.10 and figure 4.13 below.

Table 4.10: Values of CBR and Swell percent of different proportion of fine-grained soil

Percent of fine-grained soil	CBR %	% Swell
0	76	0.028
5	88	0.035
10	95	0.05
15	110	0.075
20	120	0.12
25	55	0.26

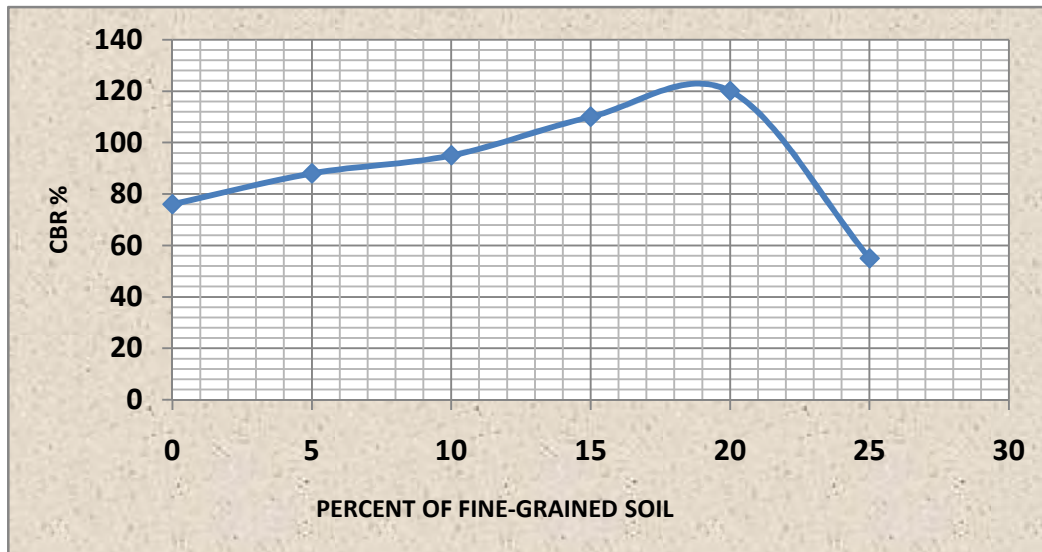


Figure 4.13: Variation of CBR with percentage of fine-grained soil mixed with cinder

According to ERA specification, for the sub-base materials the minimum soaked California Bearing Ratio (CBR) shall be 30% when determined in accordance with the requirements of AASHTO T-193.

All the results show on the above figure for 0, 5,10,15,20 & 25 % of fine-grained soil by mass satisfies the ERA manual requirements. Figure 4.13 also shows that the percentage of CBR values increases up to 19 % of fine-grained soil by mass and decreases afterwards. This trend agrees with the compaction test results.

4.2.4 Abrasion resistance

This method is used to measure the hardness of the natural cinder gravel and to know its resistance during compaction. The Ethiopia Road Authority Manual Specification recommends that the Los Angeles Abrasion value shall not exceed 51% when determined in accordance with the requirements of AASHTO T-96.

Los Angeles Abrasion test measures the hardness of aggregate. A sample is placed in a drum with steel balls. The drum is rotated and the balls grind down the aggregate particles. Soft aggregates are quickly ground to dust, while hard aggregates lose little mass [3].

Abrasion resistance applies only to coarse aggregates. Aggregates vary in their resistance to fracturing under impact (toughness) and breaking down into smaller pieces from abrasive action (hardness). The acceptable limits are set by the Los Angeles Abrasion Test AASHTO T-96. The limits vary from 30.0 to 50.0 percent, depending on the classification of the aggregate. The percentage is a measure of the degradation or loss of material as a result of impact and abrasive actions [21].

Los Angeles Abrasion Test Procedure (Grade B)

1. The sample was sieved with 19 mm sieve & retained material discarded.
2. The material passing 19 mm & retained on sieve 12.5 mm was weighed.
3. The material passing 12.5 mm & retained on sieve 9.5 mm was weighed.

4. The material obtained on steps 2 & 3 was washed & placed in an oven for 24 hrs.
5. 2500 gms of the material retained on sieve 12.5mm & 2500 gms of material retained on sieve 9.5 mm was weighed.
6. The two materials (5000 g total) were mixed & placed inside the LAA machine with 11 balls together.
7. The LAA machine was set to rotate at 500 revolutions for 15 minutes.
8. The material was sieved with 1.7 mm & retained weight was recorded.

The LA Abrasion value (LAA) is then calculated using:

$$LAA = \frac{(Total\ weight\ of\ material\ tested - Material\ retained\ on\ 1.7\ mm\ sieve\ size\ after\ testing) \times 100}{Total\ weight\ of\ material\ tested}$$

..... 4.1

Table 4.11 below shows the average calculated values LAA for natural cinder by use of the Los Angeles Abrasion machine is 48 %. This value satisfies the ERA manual specification but shows that cinder gravel is a weak material.

Table 4.11: Resistance of Abrasion of cinder by use of the Los Angeles Abrasion machine

Designation	Trail 1	Trail 2
No. of revolutions	500	500
Total Wt.of sample tested(gm)	5000	5000
Wt.of tested sample retained on 1.7mm sieve	2610	2588
Percent Loss (%)	47.8	48.2

Average : 48 %

4.3 Additional Tests

4.3.1 Absorption and specific gravity

The determination of absorption potential and specific gravity of natural cinder gravel is important to know some of the especial characteristics cinder gravel.

AASHTO T-85 defines absorption as the increase in the weight of aggregate because of water in the pores of the material, but not including water adhering to the outside surface of the particles. Absorption values are used to calculate the change in the mass of an aggregate due to water absorbed in the pore spaces within the constituent particles, compared to the dry condition, when it deemed that the aggregate has been in contact with water long enough to satisfy most of the absorption potential. The laboratory standard for absorption is that obtained after submerging dry aggregate for approximately 15 hours in water. For an aggregate that has been in contact with water and that has free moisture on the particle surfaces, the percentage of free moisture can be determined by deducting the absorption from the total moisture content determined by AASHTO T-255 drying.

Bulk specific gravity is the characteristic generally used for calculation of the volume occupied by the aggregate in various mixtures containing aggregate including Portland cement concrete and other mixtures that are proportioned or analyzed on an absolute volume basis. The bulk specific gravity determined on the saturated surface-dry basis is used if the aggregates is wet, that is, if its absorption has been satisfied.

Since cinder gravel is lightweight aggregate, the pores may or may not become essentially filled with water after immersion for 15 hours. In fact, many such aggregates can remain immersed in water for several days without most of the aggregates' absorption potential. Therefore, this method for both Absorption and Specific gravity is not intended for coarser cinder gravel that retained 4.75 mm sieve size.

Laboratory Test for Fine Aggregate (material < 4.75 mm)

- i. Mass of saturated, surface dry (M_{ssd}) Cinder gravel in air = 500.0 g

$$\text{Mass of dry Cinder gravel (M}_d) = 449.0 \text{ g}$$

$$\text{Mass of absorbed water (M}_{wa}) = M_{ssd} - M_d$$

$$\text{Absorption} = \text{Mass of absorbed water} / \text{Mass of dry cinder}$$

$$= M_{wa} / M_d$$

$$= (500 - 449) \text{ g} / 449 \text{ g}$$

$$= 0.1135$$

$$\text{Absorption (\%)} = 11.35$$

- ii. Mass of saturated, surface dry (M_{ssd}) Cinder gravel in air (A) = 500 gm

$$\text{Mass of pycnometer + Soil + Water (M}_2) \text{ (g)} = 1561.9 \text{ gm}$$

$$\text{Mass of pycnometer + water (M}_1) \text{ (g)} = 1269.9 \text{ gm}$$

$$\text{Specific gravity of Cinder } 23^\circ\text{c} = A / (M_1 + A - M_2)$$

$$= 500 / (1269.9 + 500 - 1561.9)$$

$$\text{Specific Gravity} = 2.4$$

Laboratory test results showed that the absorption & specific gravity of the cinder gravel that passes sieve 4.75 mm is 11.35 % and 2.4 respectively. Therefore, the cinder gavel has high water absorption capacity because of its high porosity and it is a lightweight aggregate.

4.3.2 Linear shrinkage

The linear shrinkage test offers a convenient method to confirm that the test results for the plasticity index are reasonable. Most types of soil exhibit a relationship between the plasticity index and the linear shrinkage of the material. The linear shrinkage is considered a more reliable indicator than the plasticity index for materials with very low plasticity.

Shrinkage due to drying is significant in clays, but less so in silts and sands. If the drying process is prolonged after the plastic limit has been reached, the soil will continue to decrease in volume, which is also relevant to the converse condition of expansion due to wetting.

Linear shrinkage is found by determining the change in length of semi-cylindrical bar sample of soil when it dries out, starting from near the liquid limit. The linear shrinkage value is a way of quantifying the amount of shrinkage likely to be experienced by clayey material.

The method covers the determination of the total linear shrinkage from linear measurements on a bar of soil of the fraction of a soil sample passing a 425 μm test sieve, originally having the moisture content of the liquid limit. For highly plastic material even 3 days of air drying may be deemed necessary. The sample should not be placed too early in the oven. Therefore, due to the long time required for air drying, linear shrinkage is a time consuming test. However, it is important to take the time required in order to produce reliable results. Finally, the linear shrinkage of the soil is calculated as a percentage from the equation given below.

If the original length when made up at about the L.L is denoted by L_0 , & the dried length by L_D , the change length is equal to $L_0 - L_D$, the linear shrinkage, LS, is given by:

$$LS = \left(1 - \frac{L_D}{L_0}\right) \times 100 \dots\dots\dots 4.2$$

Where: L_D is the length of the oven-dry specimen (in mm).

L_0 is the original length of the specimen (in mm).



Figure 4.14: Oven-dried specimen for different proportion of fine-grained soil

Table 4.12 and figure 4.15 show results of linear shrinkage different proportions of fine-grained soil.

Table 4.12: Determination of Linear shrinkage

Percent of fine-grained soil	Linear shrinkage
0	0.14%
5	0.36%
10	0.57%
15	0.86%
20	1.14%
25	1.43%

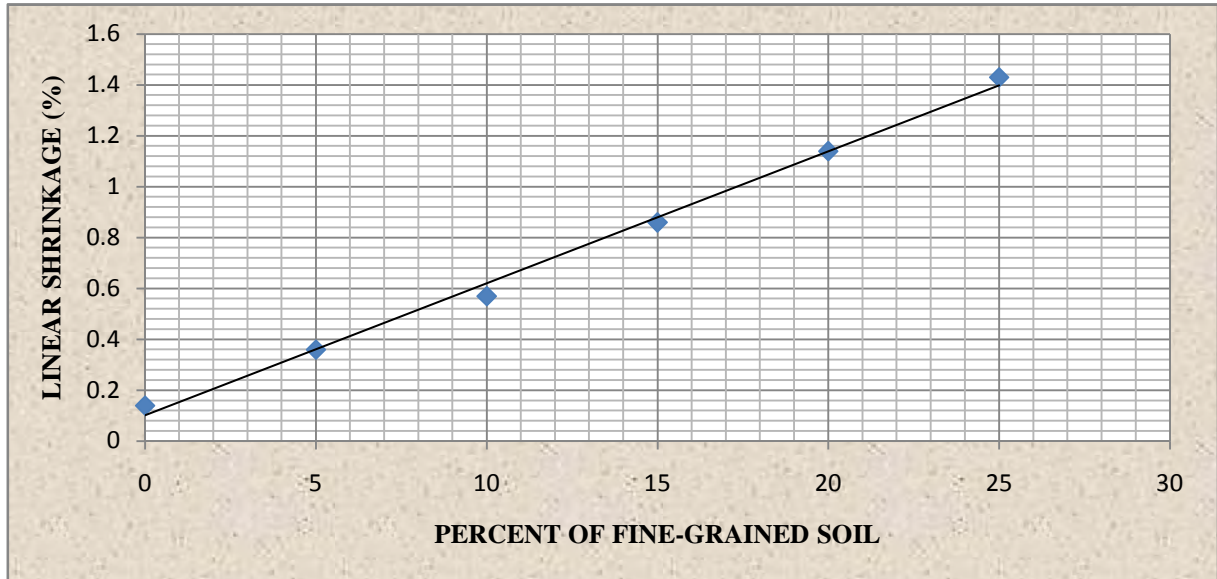


Figure 4.15: Variation of linear shrinkage with percentage of fine-grained soil mixed with cinder

Figure 4.15 shows that the percentage of linear shrinkage increases as fine-grained soil material proportion increases. All values of linear shrinkage for the 0, 5, 10, 15, 20 & 25 % of fine-grained soil proportions by mass are very low and are less than 3%, satisfying ERA manual specification.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

This thesis attempted to investigate the mechanical stabilization of cinder gravel with fine-grained soil to be used as road sub-base material. From this thesis work the following conclusions were drawn:

1. Results of sieve analyses that were performed after compaction clearly show significant amount of breaking down of coarser cinder gravel particles.
2. Because of the weak nature of cinder gravels, break down under compaction occurred with an improvement in gradation of cinder but still needs an additional fine-grained soil to achieve the ERA specification.
3. In order to minimize time and effort, arithmetic proportioning method can be used for the first trial proportioning.
4. The blended sub-base material has no plasticity characteristics and the plastic product is zero.
5. Blending of cinder with an optimum fine-grained soil improves its properties.
6. From MDD- percent of fine-grained soil curve, the optimum amount of fine-grained soil is 19 % by mass.
7. From CBR-percent of fine-grained soil curve, the optimum amount of fine-grained soil is 19 % by mass.

5.2 Recommendations for future work

The present work has attempted to obtain the optimum blending proportion of cinder with fine-grained soil only in the case of Butajira-Gubre asphalt road project. However, due to financial constraints and time limitations the present research work did not cover the whole cinder gravel in Ethiopia. In view of this, it would be desirable to consider the following recommendations on better use of cinder gravels in road construction.

1. The country is endowed with widely distributed cinder gravel deposits. In order to prepare proportioning guide-line on cinder gravel with fine-grained material, it is better to take samples from different areas of Ethiopia.
2. A full scale experiments should be carried out in Ethiopia to examine the performance of volcanic cinder gravels blended with cement or bitumen as surfacing materials for different areas of cinders in Ethiopia.

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APPENDICES

Appendix-A
Descriptive test results of borrow soil

Table A- 1: Atterberg Limits of fine-grained Soil

Item Description	Liquid Limit Trails				Plastic Limit trails	
	#1	#2	#3	#4	#1	#2
Test no.	#1	#2	#3	#4	#1	#2
Number of blows	17	22	29	35		
Container no.	103	D-4	62	C-19	95	A-14
Mass of Wet Soil + Container (m ₂)g	30.64	31.44	31.28	27.04	18.99	19.17
Mass of Dry Soil + Container (m ₃)g	26.45	27.16	27	23.49	18.28	18.43
Mass of Container (m ₁)g	15.57	15.91	15.6	13.93	15.57	15.64
Mass of moisture (m ₂ -m ₃)	4.19	4.28	4.28	3.55	0.71	0.74
Mass of dry Soil (m ₃ -m ₁)g	10.88	11.25	11.4	9.56	2.71	2.79
Moisture Content = (m ₂ -m ₃)/(m ₃ -m ₁)*100	38.51	38.05	37.54	37.14	26.2	26.5
Liquid Limit/Plastic Limit	37.8				26.35	

L.L=37.8
 P.L=26.35
 P.I=11.45

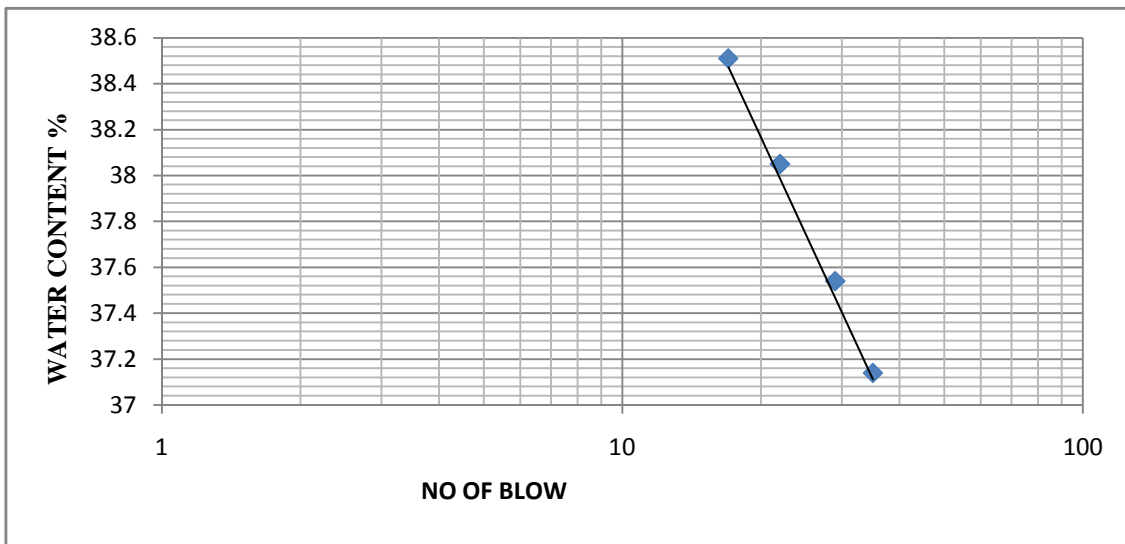


Figure A- 1: Flow curve for borrow material

Table A- 2: Sieve Analysis (wet preparation method)

Sieve size (mm)	Wt Retained	%Retained	Cum.%passing
75	0	0	100
50	0	0	100
25	0	0	100
9.5	0	0	100
4.75	20	2.21	97.79
2	50	5.55	92.24
0.425	94	10.43	81.81
0.075	240	26.63	55.18
pan	497	55.16	0

Before Wash =901 gm

After Wash = 410 gm

Loss = 491 gm

Table A- 3: Determination of specific gravity

Trail no	#1	#2	#3
pycnometer no	P2	P20	PB
Mass of Dry Soil	25	25	25
Mass of Pycnometer + Soil +water (M1)(g)	164.53	159.8	162.9
Mass of pycnometer + water (M1)	149.6	144.74	147.89
Test temperature (°C)	20	20.5	20.6
Specific gravity of soil at test temperature	2.48	2.52	2.5
correction factor	1	0.9995	0.9994
specific gravity of soil at 20 °C	2.48	2.52	2.5
Average specific gravity of cinder at 20 °C	2.5		

Table A- 4: Determination of free swell index of borrow soil

Trail no	#1	#2
Initial volume (ml)	10	10
Final volume (ml)	14	13
Free Swell Index (%)	40	30
Average	35 %	

Table A- 5: Hydrometer analysis for borrow Soil

Elapsed Time [min]	Test Temp. [C°]	Actual Hydrometer Reading	Composite Correction	Corrected Hydrometer Reading	Effective Depth [cm]	Coefficient K	Percent Finer	Grain Size [mm]	Percent Finer for combined analysis
0.5	16.9	1.03	0.00332	1.02668	8.36	0.014878	88.93	0.06	49.0
1	16.9	1.029	0.00332	1.02568	8.63	0.014878	85.6	0.0437	47.23
2	16.4	1.0245	0.00342	1.02108	9.92	0.01497	70.26	0.0331	38.77
5	16.3	1.02	0.00344	1.01656	11.01	0.01499	55.2	0.0222	30.46
8	16.4	1.017	0.00342	1.01358	11.80	0.01497	45.26	0.018	24.97
15	16.3	1.014	0.00344	1.01056	12.59	0.01499	35.2	0.0137	19.42
30	16.5	1.0105	0.0034	1.0071	13.52	0.01495	23.66	0.01	13.05
60	16.3	1.0091	0.00344	1.00566	13.89	0.01499	18.86	0.0072	10.40
80	16.7	1.0087	0.00336	1.00534	13.99	0.01491	17.80	0.0062	9.82
120	16.9	1.007	0.00332	1.00368	14.44	0.01487	12.26	0.0051	6.76
240	17.4	1.0065	0.00322	1.00328	14.58	0.01478	10.93	0.0036	6.03
480	18.1	1.006	0.00308	1.00292	14.71	0.01465	9.73	0.0025	5.36
1440	16.6	1.005	0.00338	1.00162	14.97	0.01493	5.4	0.0015	2.9

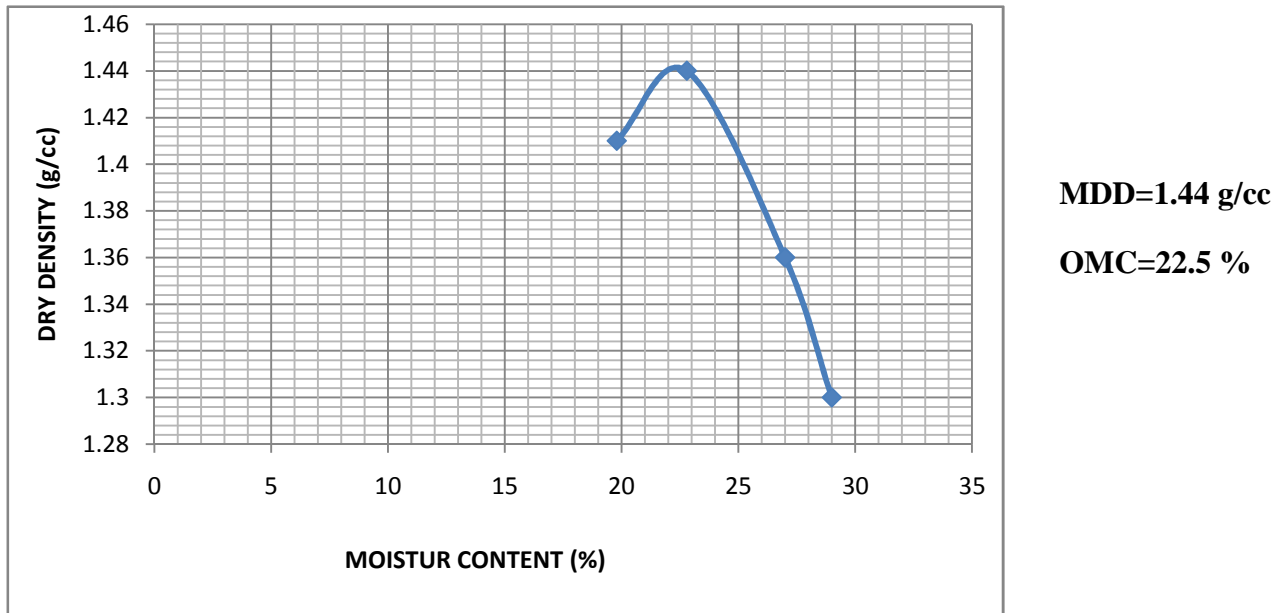


Figure A- 3: Moisture-Dry Density curve of borrow soil

California Bearing Ratio (CBR) Test

(Test Method of AASHTO T-193)

Table A- 7: Determination of dry density before & after soaking of fine-grained soil only

BLOWS	BEFORE SOAKING		AFTER SOAKING	
	DD (g/cc)	MOISTURE (%)	DD (g/cc)	MOISTURE (%)
10	1.22	22.04	1.24	38.56
30	1.35	22.37	1.34	31.66
65	1.43	22.63	1.43	27.82

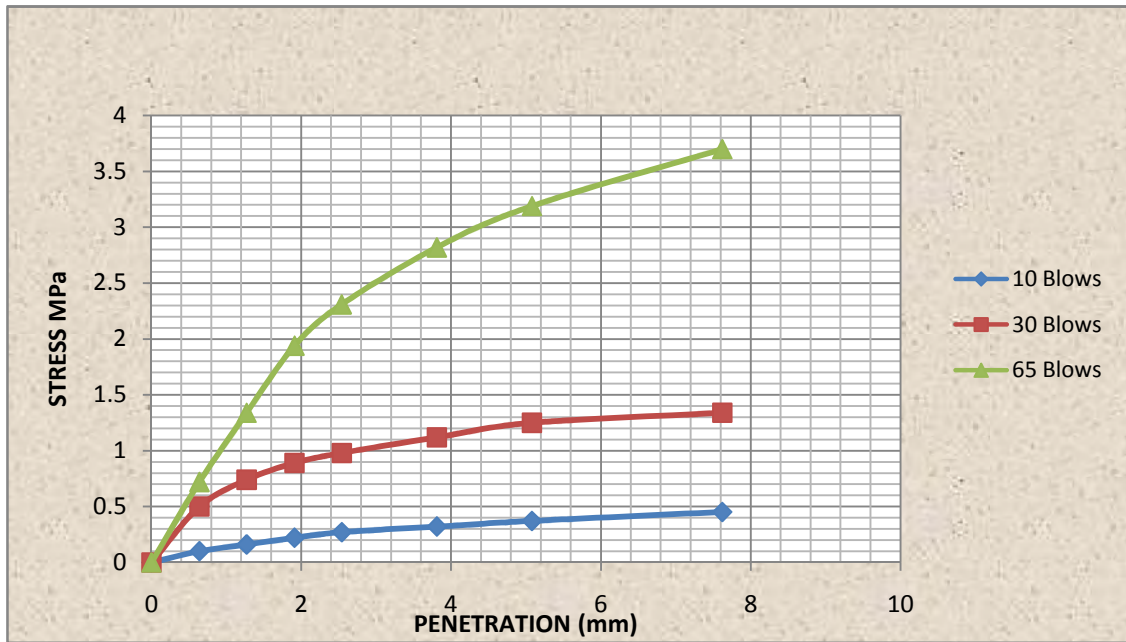


Figure A- 4: Load-penetration curves of fine-grained soil only

Table A- 8: Determination of CBR & Swell (%) of fine-grained soil only

BLOWS	STRESS (MPa)		CBR (%)		SWELL (%)
	2.54mm	5.08mm	2.54	5.08	
10	0.27	0.37	3.9	3.6	1.09
30	0.98	1.25	14.24	12.1	1.06
65	2.31	3.19	33.5	31	0.79

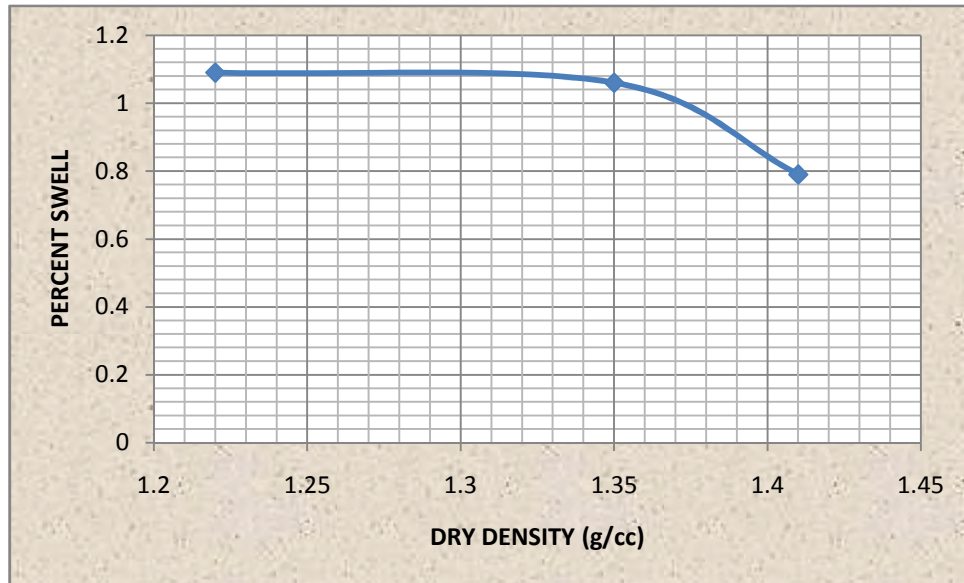


Figure A- 5: Dry Density Vs Percent Swell of fine-grained soil only

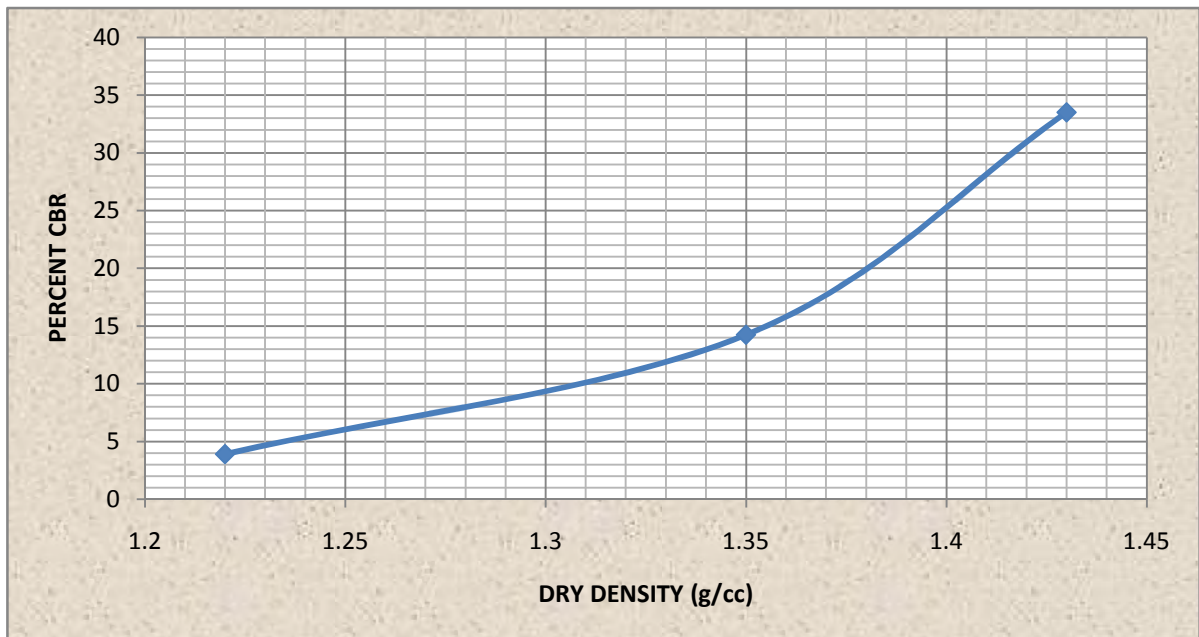


Figure A- 6: Dry Density Vs Percent CBR of fine-grained soil only

Table A- 9: Determination of CBR & Swell at 95 % of MDD of fine-grained soil only

BLOW	DRY DENSITY (g/cc)	CBR (%)	SWELL (%)
10	1.22	3.9	1.09
30	1.35	14.24	1.06
65	1.43	33.5	0.79
CBR AT 95% OF MDD		18 %	
SWELL AT 95% OF MDD		0.99 %	

Appendix-B
**Grading Limits of different proportion fine-grained soil with
cinder**

Table B- 1: Sieve analysis of Sub-base material only cinder gravel
(Wet Sieve Method)

i. Gradation Before Compaction

SIEVE SIZE(mm)	Weight retained	%retained	%Cum.%passing	Upper limit spec.	Lower limit. Spec.
50	0	0	100	100	100
25	2169	31.89	68.11	85	55
9.5	1779	26.16	41.95	70	40
4.75	1039	15.27	26.68	60	30
2	1000	14.7	11.98	51	20
0.425	520	7.64	4.34	30	10
0.075	229	3.36	0.98	15	5
pan	64	0.94	0		

Before Wash =6800 gm

After Wash = 6763 gm

Loss = 37 gm

ii. Gradation After Compaction

SIEVE SIZE(mm)	Weight retained	%retained	%Cum.%passing	Upper limit spec.	Lower limit. Spec.
50	0	0	100	100	100
25	453	10.69	89.31	85	55
9.5	1066	25.16	64.15	70	40
4.75	641	15.12	49.03	60	30
2	689	16.26	32.77	51	20
0.425	730	17.23	15.54	30	10
0.075	478	11.28	4.26	15	5
pan	180	4.24	0		

Before Wash =4237 gm

After Wash = 4062 gm

Loss = 175 gm

Table B- 2: Sieve Analysis of Sub-base material cinder gravel with 5 % fine-grained Soil

(Wet Sieve Method)

i. Gradation Before Compaction

SIEVE SIZE(mm)	Weight retained	%retained	%Cum.%passing	Upper limit spec.	Lower limit. Spec.
50	0	0	100	100	100
25	2670	33.9	66.1	85	55
9.5	2566	32.6	33.5	70	40
4.75	828	10.5	23	60	30
2	558	7.1	15.9	51	20
0.425	536	6.8	9.1	30	10
0.075	390	4.95	4.2	15	5
pan	325	4.1	0		

Before Wash =7873 gm

After Wash = 7566 gm

Loss = 307 gm

ii. Gradation After Compaction

SIEVE SIZE(mm)	Weight retained	%retained	%Cum.%passing	Upper limit spec.	Lower limit. Spec.
50	0	0	100	100	100
25	423	10.21	89.79	85	55
9.5	949	22.91	66.88	70	40
4.75	555	13.4	53.48	60	30
2	685	16.54	36.94	51	20
0.425	728	17.58	19.36	30	10
0.075	477	11.52	7.84	15	5
pan	324	7.82	0		

Before Wash =4141 gm

After Wash = 3824 gm

Loss = 317 gm

Table B- 3: Sieve analysis of Sub-base material cinder gravel with 10 % fine-grained Soil
(Wet Sieve Method)

i. Gradation Before Compaction

SIEVE SIZE(mm)	Weight retained	%retained	%Cum.%passing	Upper limit spec.	Lower limit. Spec.
50	0	0	100	100	100
25	1415	28.2	71.8	85	55
9.5	1629	32.46	39.34	70	40
4.75	542	10.8	28.54	60	30
2	465	9.26	19.28	51	20
0.425	335	6.67	12.61	30	10
0.075	272	5.42	7.19	15	5
pan	359	7.15	0		

Before Wash =5017 gm

After Wash = 4672 gm

Loss = 345 gm

iii. Gradation After Compaction

SIEVE SIZE(mm)	Weight retained	%retained	%Cum.%passing	Upper limit spec.	Lower limit. Spec.
50	0	0	100	100	100
25	270	6.465	93.535	85	55
9.5	1147	27.46	66.075	70	40
4.75	595	14.24	51.835	60	30
2	613	14.67	37.165	51	20
0.425	613	14.67	22.495	30	10
0.075	472	11.3	11.16	15	5
pan	466	11.16	0		

Before Wash =4176 gm

After Wash = 3717 gm

Loss = 459 gm

Table B- 4: Sieve analysis of Sub-base material cinder gravel with 15 % fine-grained Soil
(Wet Sieve Method)

i. Gradation Before Compaction

SIEVE SIZE(mm)	Weight retained	%retained	%Cum.% passing	Upper limit spec.	Lower limit. Spec.
50	0	0	100	100	100
25	1305	21.08	78.9	85	55
9.5	1770	28.59	50.32	70	40
4.75	900	14.53	35.8	60	30
2	827	13.36	22.44	51	20
0.425	406	6.56	15.88	30	10
0.075	355	5.73	10.15	15	5
pan	627	10.13	0		

Before Wash =6190 gm

After Wash = 5583 gm

Loss = 607 gm

ii. Gradation After compaction

SIEVE SIZE(mm)	Weight retained	%retained	%Cum.% passing	Upper limit spec.	Lower limit. Spec.
50	0	0	100	100	100
25	381	9.1	90.9	85	55
9.5	1141	27.25	63.65	70	40
4.75	547	13.06	50.59	60	30
2	569	13.6	36.99	51	20
0.425	563	13.44	23.55	30	10
0.075	452	10.8	12.74	15	5
pan	533	12.73	0		

Before Wash =4186 gm

After Wash = 3658 gm

Loss = 528 gm

Table B- 5: Sieve analysis of Sub-base material cinder gravel with 20 % fine-grained Soil
(Wet Sieve Method)

i. Gradation Before Compaction

SIEVE SIZE(mm)	Weight retained	%retained	%Cum.%passing	Upper limit spec.	Lower limit. Spec.
50	0	0	100	100	100
25	2274	23.61	76.39	85	55
9.5	2694	27.97	48.42	70	40
4.75	1492	15.5	32.92	60	30
2	863	8.96	23.96	51	20
0.425	457	4.75	19.21	30	10
0.075	648	6.72	12.49	15	5
pan	1202	12.48	0		

Before Wash =9630 gm

After Wash = 8462 gm

Loss = 1168 gm

ii. Gradation After Compaction

SIEVE SIZE(mm)	Weight retained	%retained	%Cum.%passing	upper limit spec.	Lower limit. Spec.
50	0	0	100	100	100
25	710	17.26	82.74	85	55
9.5	1048	25.48	57.26	70	40
4.75	405	9.85	47.41	60	30
2	434	10.55	36.86	51	20
0.425	470	11.43	25.43	30	10
0.075	413	10.04	15.39	15	5
pan	633	15.39	0		

Before Wash =4113 gm

After Wash = 3484 gm

Loss = 1168 gm

Table B- 6: sieve analysis of Sub-base material cinder gravel with 25 % fine-grained Soil
(Wet Sieve Method)

i. Gradation Before Compaction

SIEVE SIZE(mm)	Weight retained	%retained	%Cum.%passing	Upper limit spec.	Lower limit. Spec.
50	0	0	100	100	100
25	2135	24.12	75.88	85	55
9.5	1878	21.22	54.66	70	40
4.75	926	10.46	44.2	60	30
2	1053	11.89	32.31	51	20
0.425	818	9.24	23.07	30	10
0.075	700	7.9	15.2	15	5
pan	1340	15.1	0		

Before Wash =8850 gm

After Wash = 7569 gm

Loss = 1281 gm

ii. Gradation After Compaction

SIEVE SIZE(mm)	Weight retained	%retained	%Cum.%passing	Upper limit spec.	Lower limit. Spec.
50	0	0	100	100	100
25	492	12.5	87.5	85	55
9.5	991	25.08	62.42	70	40
4.75	463	11.71	50.71	60	30
2	438	11.08	39.63	51	20
0.425	431	10.9	28.73	30	10
0.075	402	10.17	18.56	15	5
pan	735	18.6	0		

Before Wash =3952 gm

After Wash = 3221 gm

Loss = 731 gm

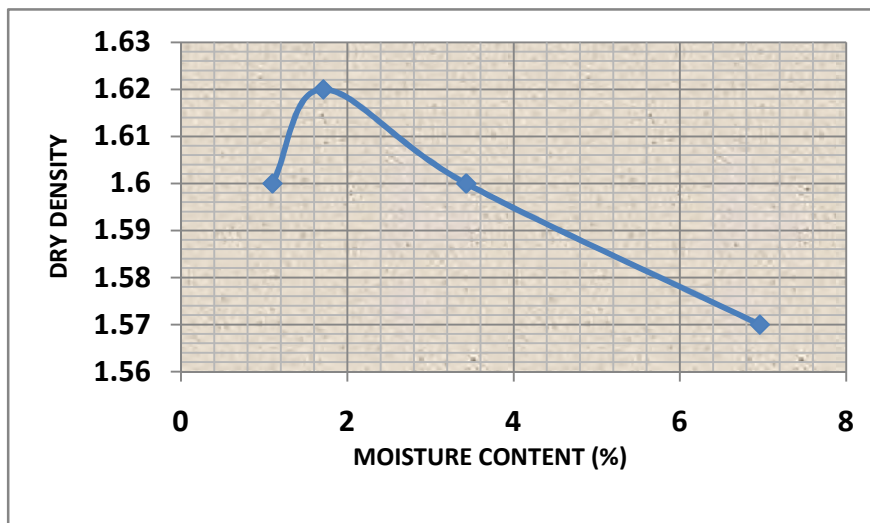
Appendix-C
Moisture-Density relation by modified proctor test

Table C- 1: Moisture-Density relations for cinder gravel only

Project: Butajira-Gubre Asphalt Road
 Location: Around Butajira Area

Sample type: Disturbed
 Test type: Modified Proctor test

sample no	1	2	3	4
Mold +wet soil (g)	10860	10900	10950	11000
Volume of Mold	2124	2124	2124	2124
Mold (g)	7418	7418	7418	7418
Wet soil (g)	3442	3482	3532	3582
Bulk density g/cm ³	1.62	1.64	1.66	1.68
Moisture content determination				
Wet soil + cont.(g)	354	361	397	435
Dry soil + cont.(g)	350	355	384	407
container (g)	5	5	5	5
Mass of moisture (g)	4	6	13	28
Dry soil (g)	345	350	379	402
Moisture cont (%)	1.1	1.71	3.43	6.96
Dry density (g/cm ³)	1.6	1.62	1.6	1.57



MDD=1.62 g/cc
OMC=1.71 %

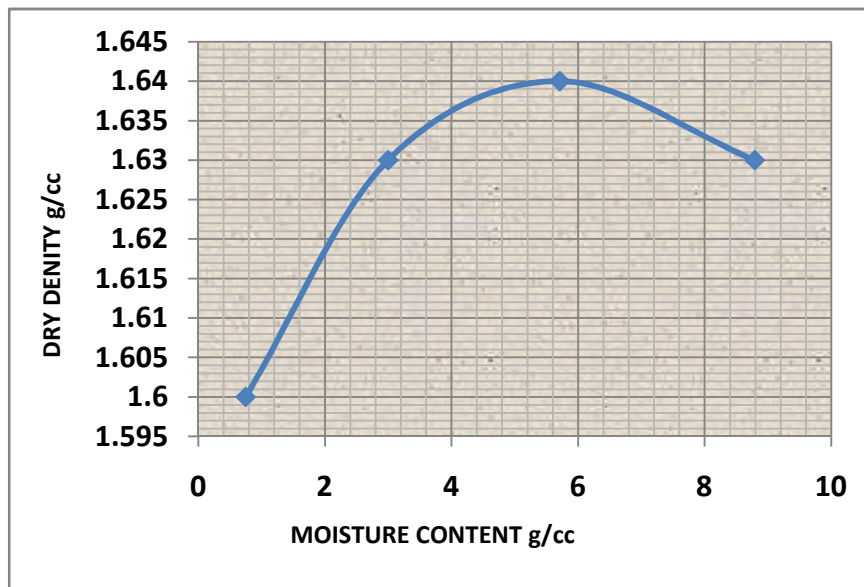
Figure C- 1: Moisture-Dry density curve for cinder gravel only

Table C- 2: Moisture-Density relations for cinder gravel with 5 % fine-grained Soil

Project: Butajira-Gubre Asphalt Road
 Location: Around Butajira Area

Sample type: Disturbed
 Test type: Modified Proctor test

sample no	1	2	3	4
Mold +wet soil (g)	10850	11000	11109	11203
Volume of Mold	2124	2124	2124	2124
Mold (g)	7418	7418	7418	7418
Wet soil (g)	3432	3582	3691	3785
Bulk density g/cm ³	1.62	1.68	1.73	1.78
Moisture content determination				
Wet soil + cont.(g)	412	277	375	376
Dry soil + cont.(g)	409	269	355	346
container (g)	5	5	5	5
Mass of moisture (g)	3	8	20	30
Dry soil (g)	404	264	350	341
Moisture cont (%)	0.75	3	5.71	8.79
Dry density (g/cm ³)	1.6	1.63	1.64	1.63



MDD=1.64 g/cc

OMC=5.71 %

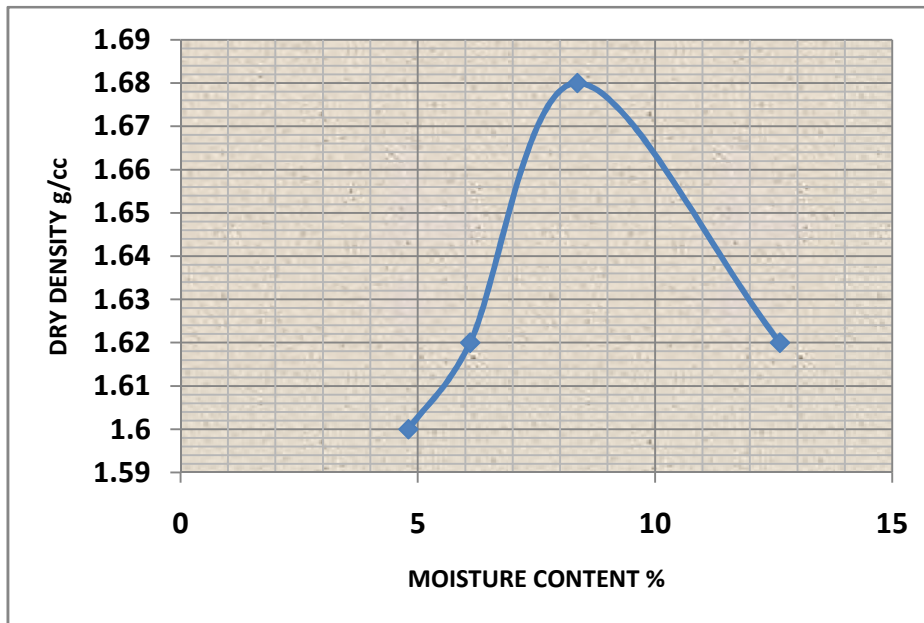
Figure C- 2: Moisture-Dry density curve for cinder gravel with 5 % fine-grained Soil

Table C- 3: Moisture-Density relations for cinder gravel with 10 % fine-grained Soil

Project: Butajira-Gubre Asphalt Road
 Location: Around Butajira Area

Sample type: Disturbed
 Test type: Modified Proctor test

sample no	1	2	3	4
Mold +wet soil (g)	10977	11077	11275	11290
Volume of Mold	2124	2124	2124	2124
Mold (g)	7418	7418	7418	7418
Wet soil (g)	3559	3659	3857	3872
Bulk density g/cm3	1.68	1.72	1.82	1.82
Moisture content determination				
Moisture can no	29	34	22	71
Wet soil + cont.(g)	334	336	342	335
Dry soil + cont.(g)	319	317	316	298
container (g)	5	5	5	5
Mass of moisture (g)	15	19	26	37
Dry soil (g)	314	312	311	293
Moisture cont (%)	4.8	6.1	8.36	12.63
Dry density (g/cm3)	1.6	1.62	1.68	1.62



MDD=1.68 g/cc
OMC=8.36 %

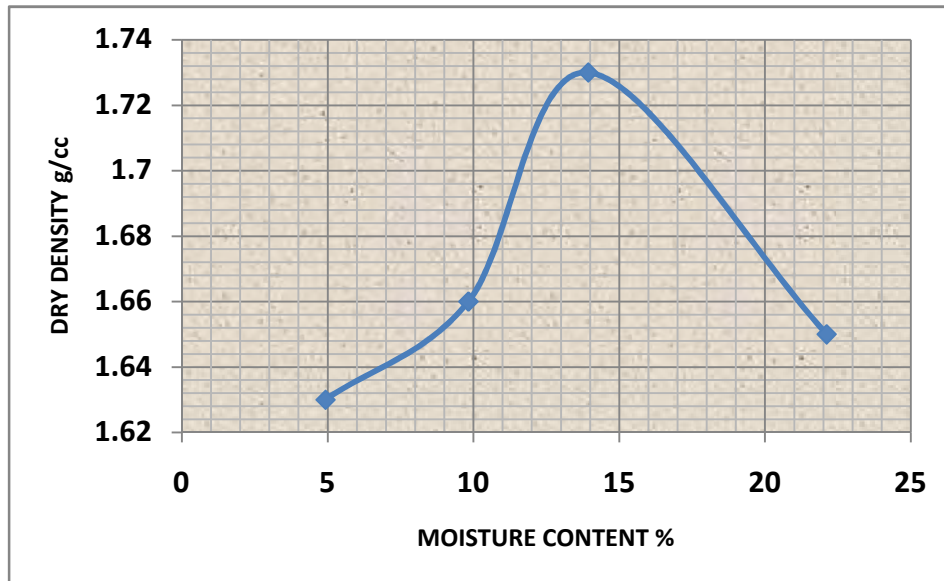
Figure C- 3: Moisture-Dry density curve for cinder gravel with 10 % fine-grained Soil

Table C- 4: Moisture-Density relations for cinder gravel with 15 % fine-grained Soil

Project: Butajira-Gubre Asphalt Road
 Location: Around Butajira Area

Sample type: Disturbed
 Test type: Modified Proctor test

sample no	1	2	3	4
Mold +wet soil (g)	11052	11310	11600	11702
Volume of Mold	2124	2124	2124	2124
Mold (g)	7418	7418	7418	7416
Wet soil (g)	3634	3892	4182	4286
Bulk density g/cm ³	1.71	1.83	1.97	2.01
Moisture content determination				
Moisture can no	19	47	22	27
Wet soil + cont.(g)	324	329	381	419
Dry soil + cont.(g)	309	300	335	344
container (g)	5	5	5	5
Mass of moisture (g)	15	29	46	75
Dry soil (g)	304	295	330	339
Moisture cont (%)	4.93	9.83	13.94	22.12
Dry density (g/cm ³)	1.63	1.66	1.73	1.65



MDD=1.73 g/cc

OMC=13.94 %

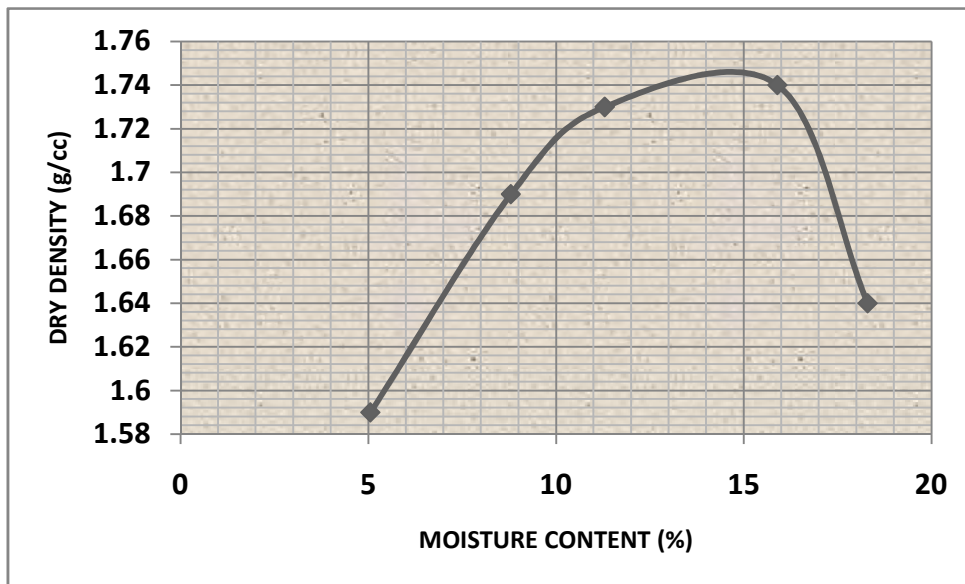
Figure C- 4: Moisture-Dry density curve for cinder gravel with 15 % fine-grained Soil

Table C- 5: Moisture-Density relations for cinder gravel with 20 % fine-grained Soil

Project: Butajira-Gubre Asphalt Road
 Location: Around Butajira Area

Sample type: Disturbed
 Test type: Modified Proctor test

sample no	1	2	3	4	5
Mold +wet soil (g)	10978	11315	11515	11710	11535
Volume of Mold	2124	2124	2124	2124	2124
Mold (g)	7415	7415	7415	7415	7415
Wet soil (g)	3563	3900	4100	4295	4120
Bulk density g/cm3	1.68	1.84	1.93	2.02	1.94
Moisture content determination					
Moisture can no	95	28	47	42	31
Wet soil + cont.(g)	358	319.6	281.2	334.4	405.7
Dry soil + cont.(g)	341	294.2	253.2	289.2	343.7
container (g)	5	5	5	5	5
Mass of moisture (g)	17	25.4	28	45.2	62
Dry soil (g)	336	289.2	248.2	284.2	338.7
Moisture cont (%)	5.06	8.8	11.3	15.9	18.3
Dry density (g/cm3)	1.59	1.69	1.73	1.74	1.64



MDD=1.75 g/cc

OMC=14.6 %

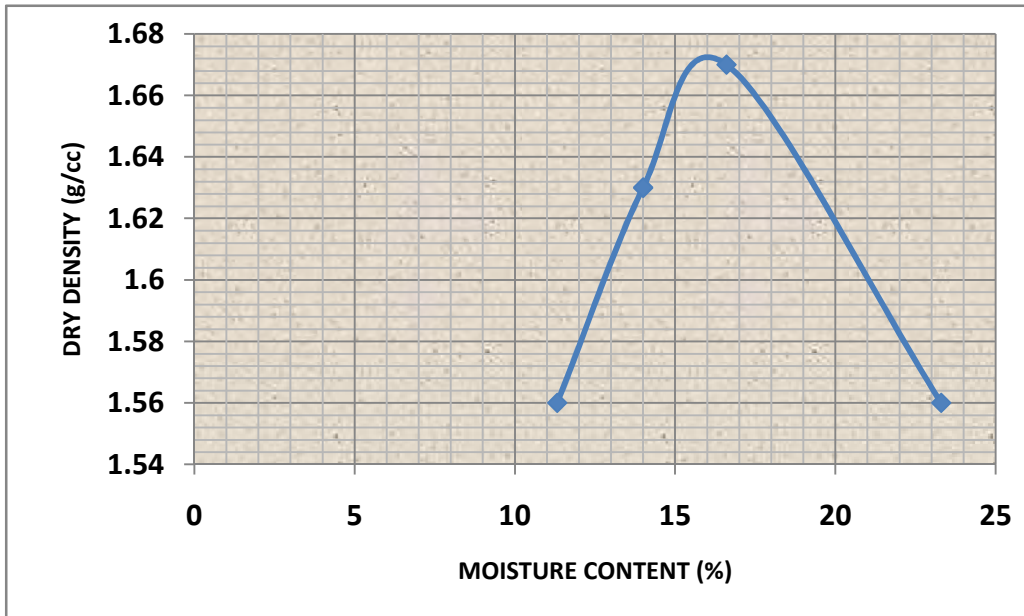
Figure C- 5: Moisture-Dry density curve for cinder gravel with 20 % fine-grained Soil

Table C- 6: Moisture-Density relations for cinder gravel with 25 % fine-grained Soil

Project: Butajira-Gubre Asphalt Road
 Location: Around Butajira Area

Sample type: Disturbed
 Test type: Modified Proctor test

sample no	1	2	3	4
Mold +wet soil (g)	11095	11353	11545	11503
Volume of Mold	2124	2124	2124	2124
Mold (g)	7414	7414	7414	7414
Wet soil (g)	3681	3939	4131	4089
Bulk density g/cm3	1.73	1.85	1.94	1.93
Moisture content determination				
Moisture can no	CMC-3	36	22	38
Wet soil + cont.(g)	272.4	281.3	258.7	243.4
Dry soil + cont.(g)	245.2	247.4	222.5	198.4
container (g)	5	5	5	5
Mass of moisture (g)	27.2	33.9	36.2	45
Dry soil (g)	240.2	242.4	217.5	193.4
Moisture cont (%)	11.32	14	16.6	23.3
Dry density (g/cm3)	1.56	1.63	1.67	1.56



MDD=1.67 g/cc
 OMC=16.1 %

Figure C- 6: Moisture-Dry density curve for cinder gravel with 25 % fine-grained Soil

Appendix-D
California Bearing Ratio (CBR) test results

All the CBR tests were conducted on the following values

Ring Calibration Factor	= 46.945 N/div
Plunger Area	= 1935 mm ²
Rate of strain	= 1.27 mm/min
Rammer Wt	= 4.54 kg
Diameter of Mold	= 152.4 mm
Volume of Mold	= 2124 cm ³

Table D- 1: Determination of dry density before & after soaking of cinder with 5 % fine-grained soil

BLOWS	BEFORE SOAKING		AFTER SOAKING	
	DD (g/cc)	MOISTURE (%)	DD (g/cc)	MOISTURE (%)
10	1.39	5.37	1.38	22.82
30	1.49	4.83	1.5	19.66
65	1.65	4.58	1.66	16.37

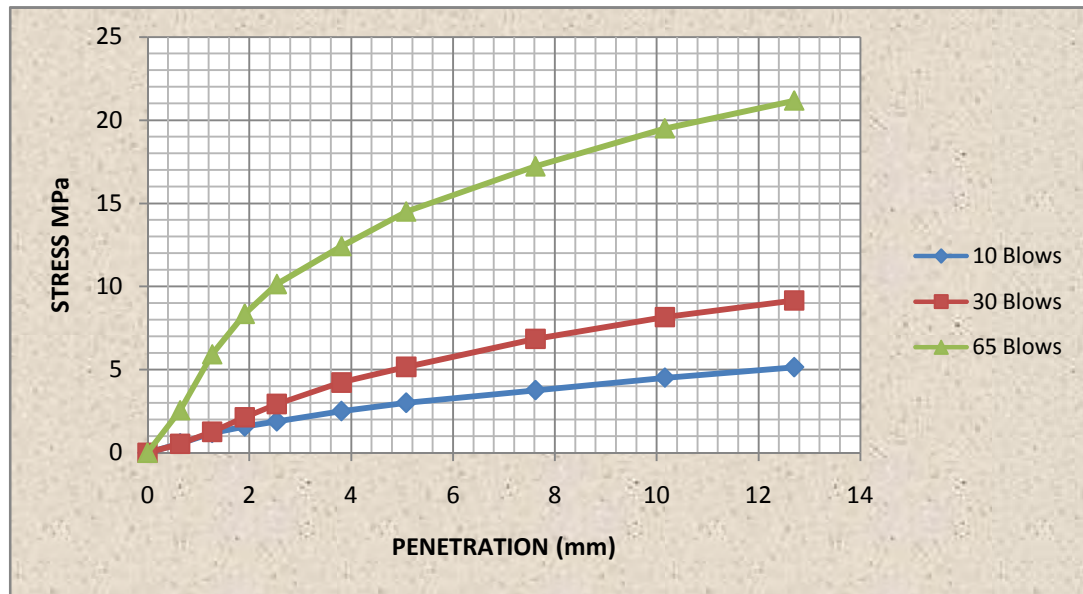


Figure D- 1: Load-penetration curves of cinder gravel with 5 % fine-grained soil

Table D- 2: Determination of CBR & Swell (%) of cinder with 5% fine-grained soil

BLOWS	STRESS (MPa)		CBR (%)		SWELL (%)
	2.54mm	5.08mm	2.54	5.08	
10	1.89	3.0	29.12	27.39	0.08
30	2.93	5.16	50.17	42.46	0.05
65	10.14	14.5	146.97	140.85	0.017

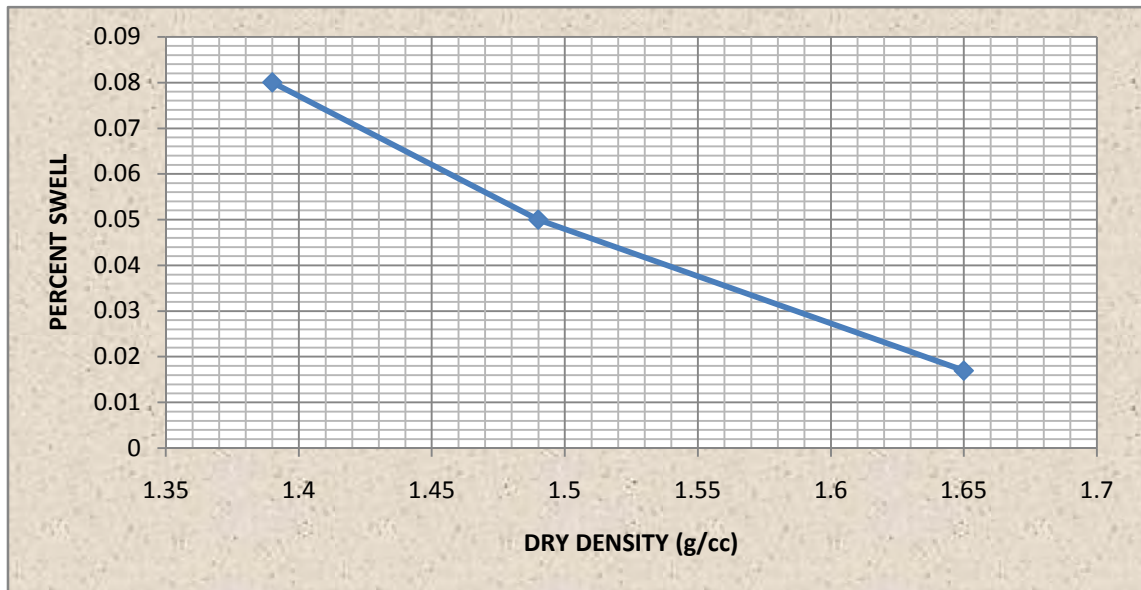


Figure D- 2: Dry Density Vs percent swell of cinder with 5% fine-grained soil

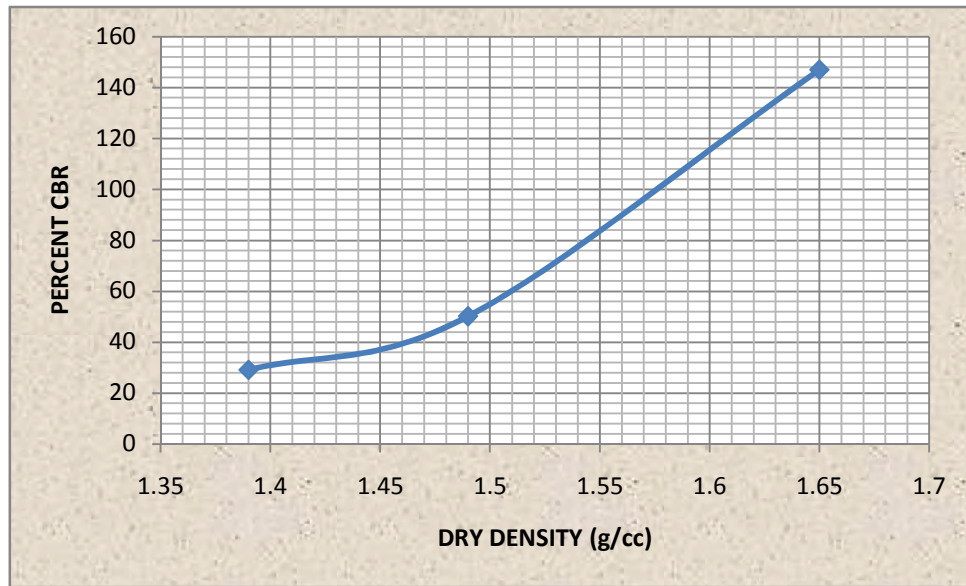


Figure D- 3 Dry Density Vs percent CBR of cinder with 5% fine-grained soil

Table D- 3: Determination of CBR & Swell at 95 % of MDD of cinder with 5 % fine-grained soil

BLOW	DRY DENSITY (g/cc)	CBR (%)	SWELL (%)
10	1.39	29.12	0.08
30	1.49	50.17	0.05
65	1.65	146.97	0.017
CBR AT 95% OF MDD		88 %	
SWELL AT 95% OF MDD		0.035 %	

Table D- 4: Determination of dry density before & after soaking of cinder with 10 % fine-grained soil

BLOWS	BEFORE SOAKING		AFTER SOAKING	
	DD (g/cc)	MOISTURE (%)	DD (g/cc)	MOISTURE (%)
10	1.41	6.47	1.43	19.9
30	1.63	5.9	1.6	18.6
65	1.76	7.03	1.78	15.32

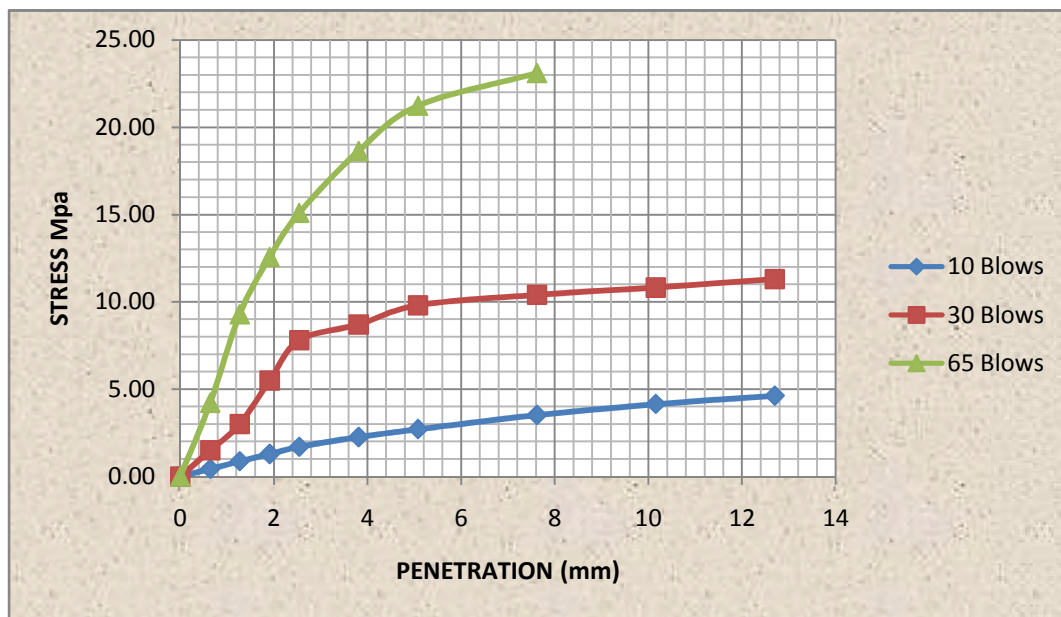


Figure D- 4: Load-penetration curves of cinder gravel with 10 % fine-grained soil

Table D- 5: Determination of CBR & Swell (%) of cinder with 10% fine-grained soil

BLOWS	STRESS (MPa)		CBR (%)		SWELL (%)
	2.54mm	5.08mm	2.54	5.08	
10	1.7	2.71	26.31	24.63	0.07
30	5.53	7.81	80.17	75.85	0.05
65	15.09	21.23	218.69	206.17	0.025

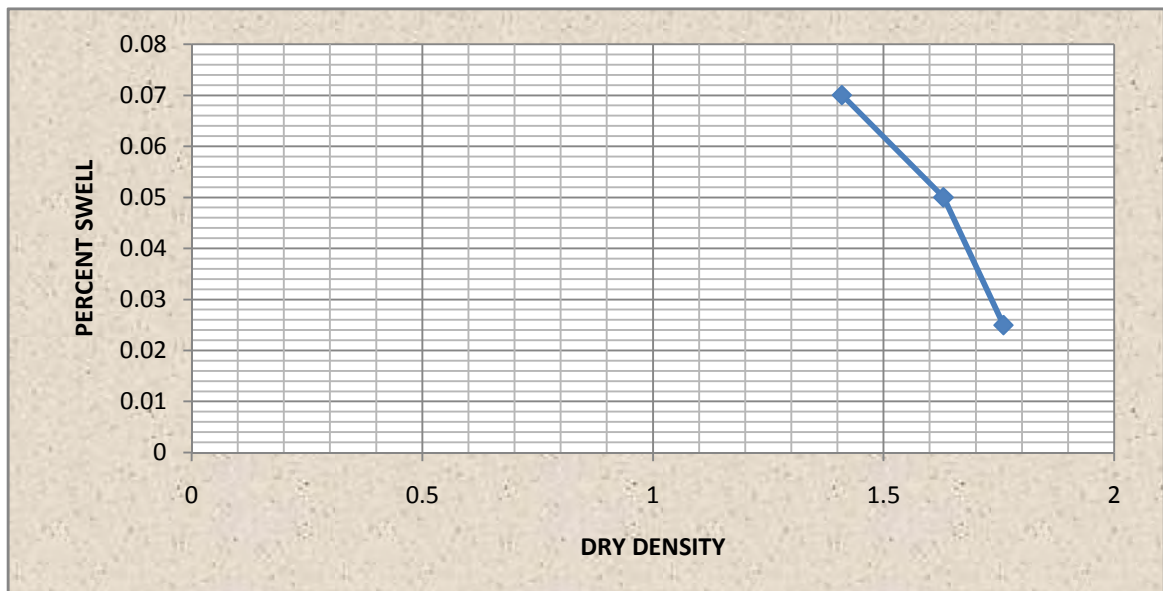


Figure D- 5: Dry Density Vs percent swell of cinder with 10% fine-grained soil

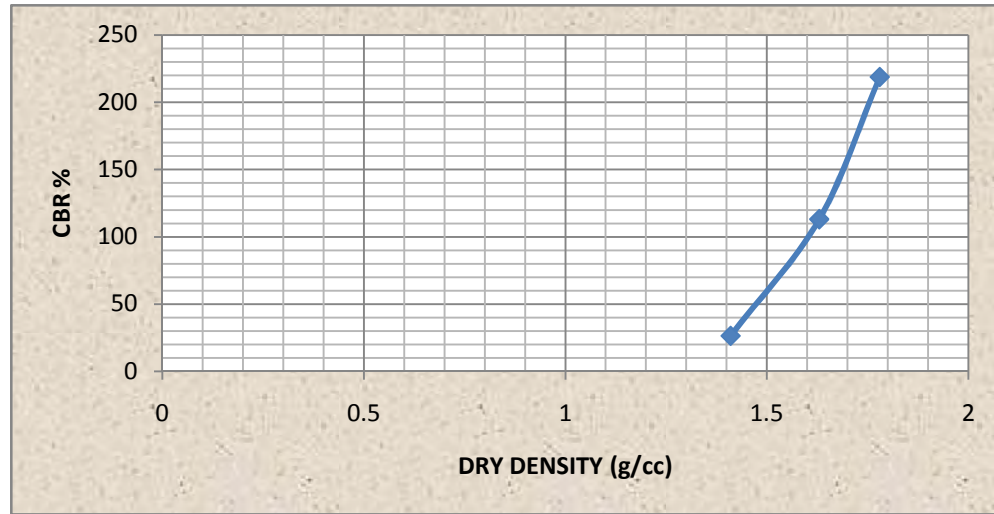


Figure D- 6: Dry Density Vs percent CBR of cinder with 10% fine-grained soil

Table D- 6: Determination of CBR & Swell at 95 % of MDD of cinder with 10 % fine-grained soil

BLOW	DRY DENSITY (g/cc)	CBR (%)	SWELL (%)
10	1.41	26.31	0.07
30	1.63	80.17	0.05
65	1.76	218.69	0.025
CBR AT 95% OF MDD		95 %	
SWELL AT 95% OF MDD		0.05 %	

Table D- 7: Determination of dry density before & after soaking of cinder with 15 % fine-grained soil

BLOWS	BEFORE SOAKING		AFTER SOAKING	
	DD (g/cc)	MOISTURE (%)	DD (g/cc)	MOISTURE (%)
10	1.52	14.0	1.51	18.31
30	1.68	13.84	1.69	16.04
65	1.79	13.91	1.78	15.82

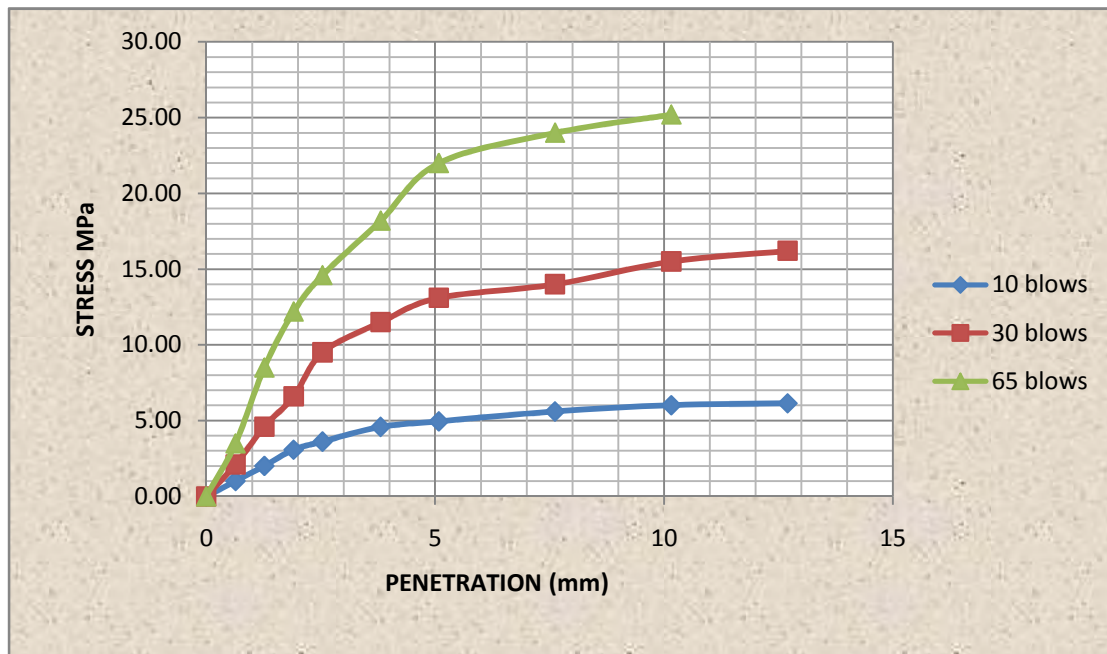


Figure D- 7: Load-penetration curves of cinder gravel with 15 % fine-grained soil

Table D- 8: Determination of CBR & Swell (%) of cinder with 15% fine-grained soil

BLOWS	STRESS (MPa)		CBR (%)		SWELL (%)
	2.54mm	5.08mm	2.54	5.08	
10	3.61	4.95	52.32	48	0.12
30	5.6	9.52	138	127	0.06
65	15.28	21.83	221.44	212	0.034



Figure D- 8: Dry Density Vs percent swell of cinder with 15% fine-grained soil

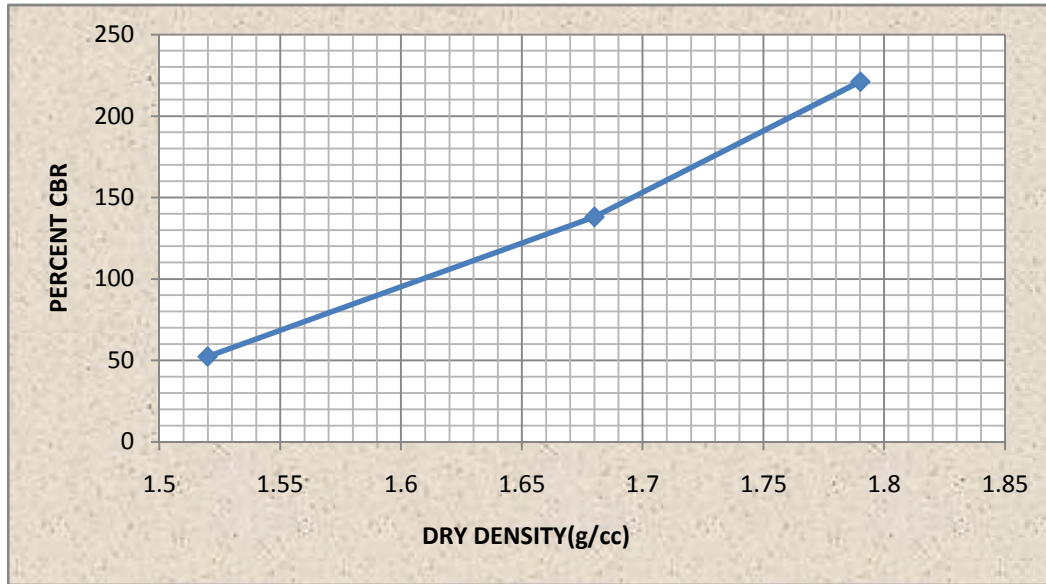


Figure D- 9: Dry Density Vs percent CBR of cinder with 15% fine-grained soil

Table D- 9: Determination of CBR & Swell at 95 % of MDD of cinder with 15 % fine-grained soil

BLOW	DRY DENSITY (g/cc)	CBR (%)	SWELL (%)
10	1.52	52.32	0.12
30	1.68	138	0.06
65	1.79	221.44	0.034
CBR AT 95% OF MDD		110 %	
SWELL AT 95% OF MDD		0.075 %	

Table D- 10: Determination of dry density before & after soaking of cinder with 20% fine-grained soil

BLOWS	BEFORE SOAKING		AFTER SOAKING	
	DD (g/cc)	MOISTURE (%)	DD (g/cc)	MOISTURE (%)
10	1.51	13.56	1.52	20.0
30	1.58	14.17	1.59	18.63
65	1.76	13.75	1.77	15.51

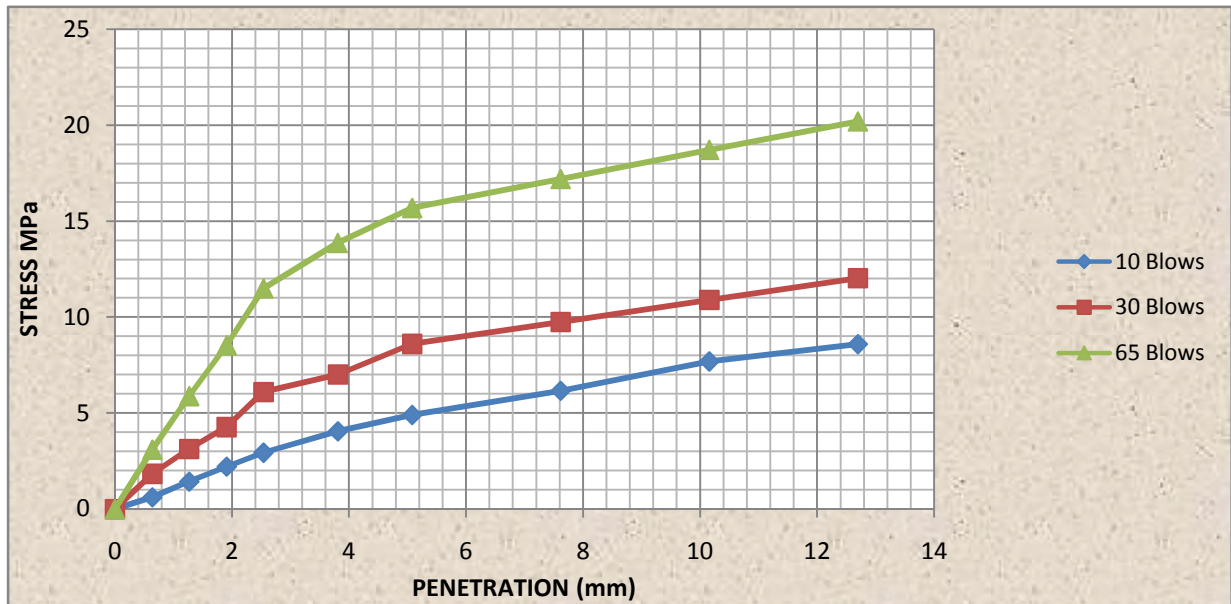


Figure D- 10: Load-penetration curves of cinder gravel with 20 % fine-grained soil

Table D- 11: Determination of CBR & Swell (%) of cinder with 20% fine-grained soil

BLOWS	STRESS (MPa)		CBR (%)		SWELL (%)
	2.54mm	5.08mm	2.54	5.08	
10	2.94	4.9	47.6	42.6	0.15
30	6.1	8.61	88.4	83.6	0.13
65	11.5	15.69	167	152.33	0.11

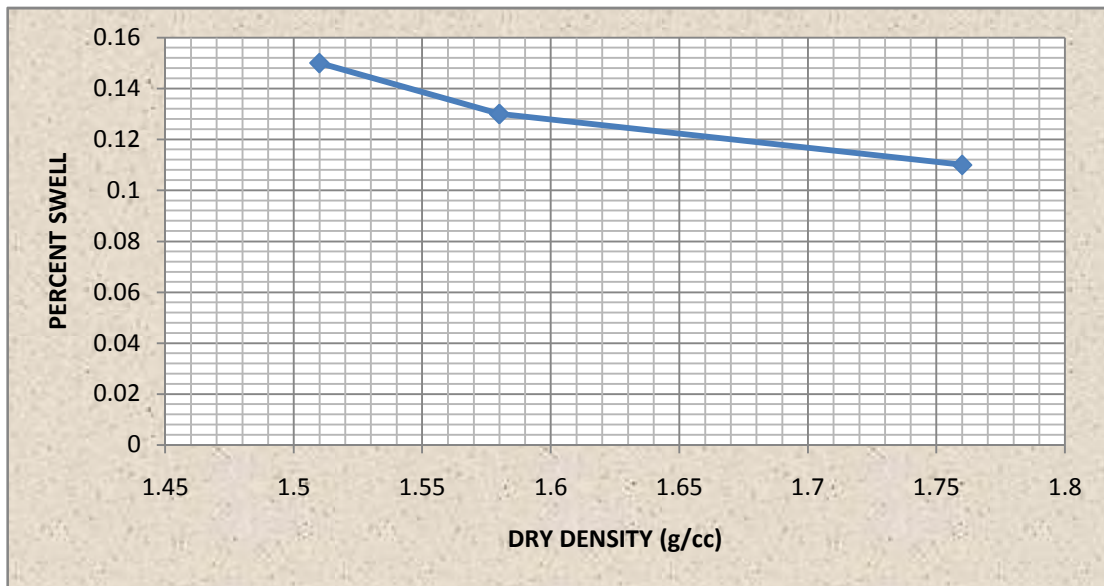


Figure D- 11: Dry Density Vs percent swell of cinder with 20% fine-grained soil

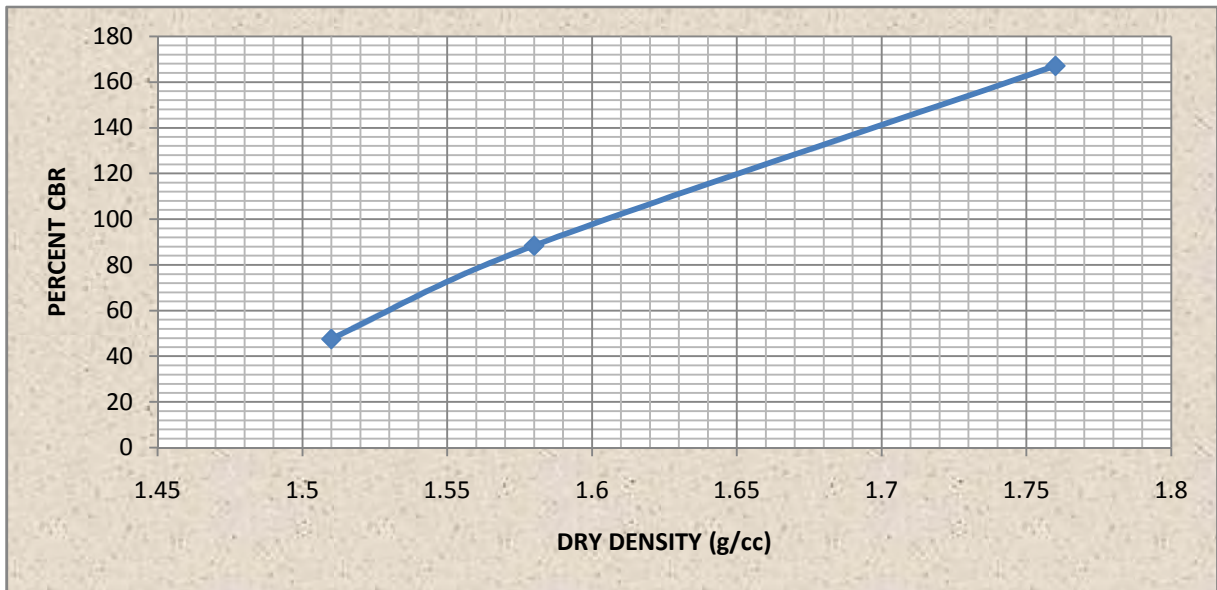


Figure D- 12: Dry Density Vs percent CBR of cinder with 20% fine-grained soil

Table D- 12: Determination of CBR & Swell at 95 % of MDD of cinder with 20 % fine-grained soil

BLOW	DRY DENSITY (g/cc)	CBR (%)	SWELL (%)
10	1.51	47.6	0.15
30	1.58	88.4	0.13
65	1.76	167	0.11
CBR AT 95% OF MDD		120 %	
SWELL AT 95% OF MDD		0.12 %	

Table D- 13: Determination of dry density before & after soaking of cinder with 25% fine-grained soil

BLOWS	BEFORE SOAKING		AFTER SOAKING	
	DD (g/cc)	MOISTURE (%)	DD (g/cc)	MOISTURE (%)
10	1.51	15.68	1.52	20.23
30	1.63	15.45	1.65	17.27
65	1.66	15.52	1.67	16.49

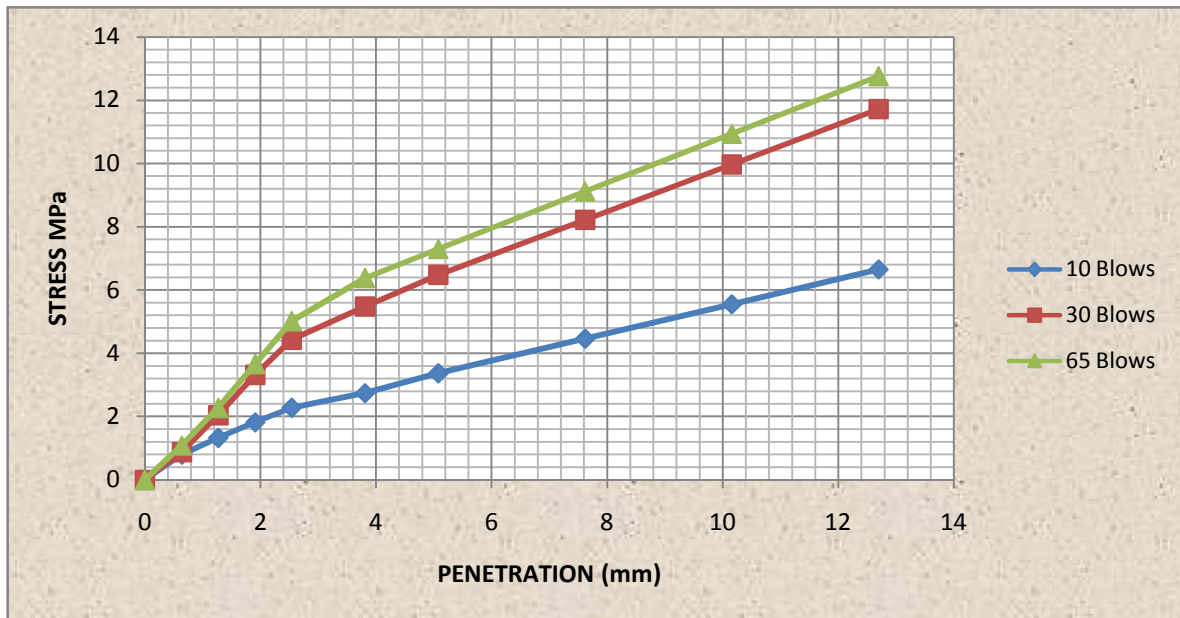


Figure D- 13: Load-penetration curves of cinder gravel with 25 % fine-grained soil

Table D- 14: Determination of CBR & Swell (%) of cinder with 25% fine-grained soil

BLOWS	STRESS (MPa)		CBR (%)		SWELL (%)
	2.54mm	5.08mm	2.54	5.08	
10	2.28	3.37	33	32.7	0.37
30	4.43	6.48	64.2	62.9	0.25
65	5.02	7.3	72.8	70.9	0.17

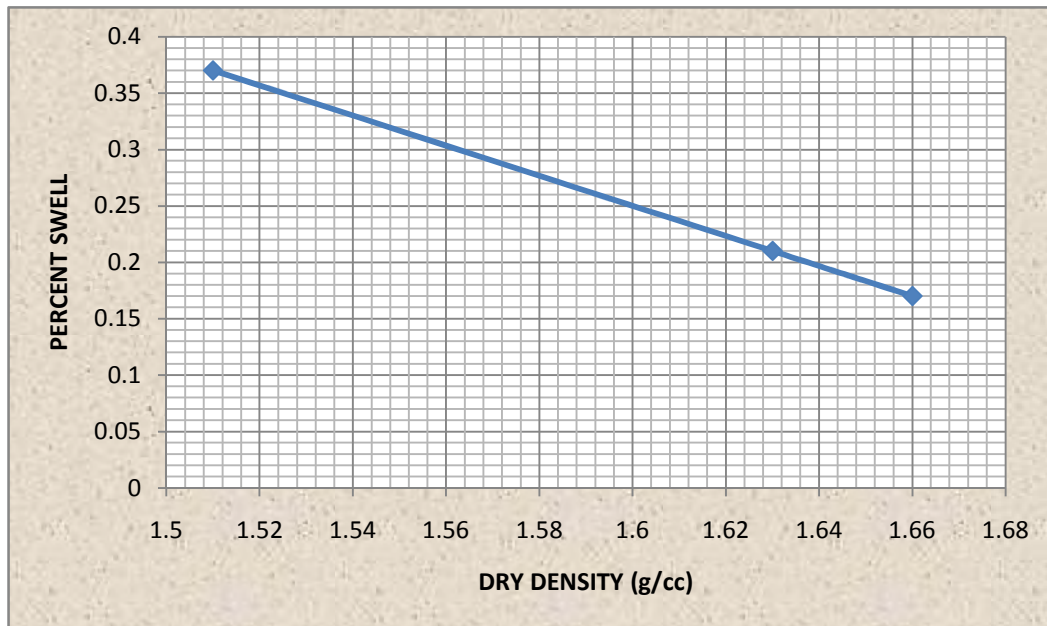


Figure D- 14: Dry Density Vs percent swell of cinder with 25% fine-grained soil

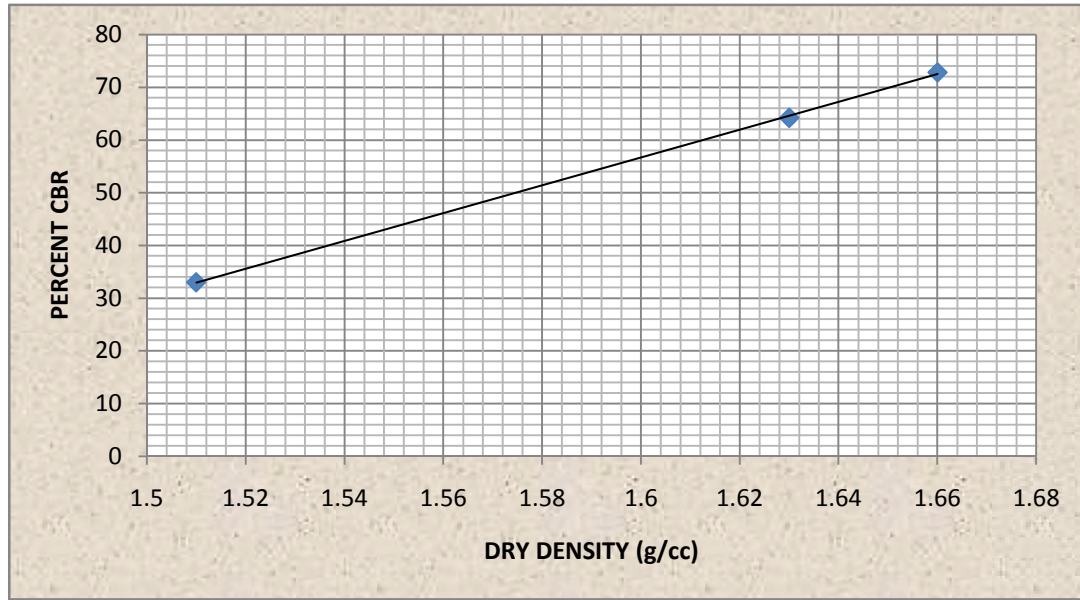


Figure D- 15: Dry Density Vs percent CBR of cinder with 25% fine-grained soil

Table D- 15: Determination of CBR & Swell at 95 % of MDD of cinder with 25 % fine grained soil

BLOW	DRY DENSITY (g/cc)	CBR (%)	SWELL (%)
10	1.51	33	0.37
30	1.63	64.2	0.25
65	1.66	72.8	0.17
CBR AT 95% OF MDD		55 %	
SWELL AT 95% OF MDD		0.26 %	

Appendix-E

Additional Test Results

Table E- 1: Absorption and Specific gravity of natural cinder gravel (materials < 4.75mm)

Trail no	#1	#2
pycnometer no	P1	P2
Mass of saturated-surface-dry test sample in air, g (A)	500	500
Mass of Pycnometer + Soil +water (M ₂)(g)	1561.9	1571.4
Mass of pycnometer + water (M ₁)(g)	1269.9	1280
Mass of oven-dry test sample in air (B) (g)	449	449
Absorption (%) = (A-B)/B	11.35	11.35
Average Absorption (%)	11.35	
specific gravity of Cinder at 23 °C = A/(M ₁ + A - M ₂)	2.4	2.397
Average specific gravity of cinder at 23 °C	2.4	

Table E- 2: Determination of Linear shrinkage values

% fine	L (mm)	Lo(mm)	LS %
0	139.8	140	0.14
5	139.5	140	0.36
10	139.2	140	0.57
15	138.8	140	0.86
20	138.4	140	1.14
25	138	140	1.43

DECLARATION

I hereby declare that the work which is being presented in this thesis entitled “**BLENDING OF CINDER WITH FINE-GRAINED SOIL TO BE USED AS SUB-BASE MATERIALS (The Case of Butajira-Gubre Road)**” is original work of my own, has not been presented for a degree in any other university and that all sources of material used for the thesis have been duly acknowledged.

Yitayou Eshete

(Candidate)

Date

This is to certify that the above declaration made by the candidate is correct to the best of my knowledge.

Dr. Hadush Seged

(Thesis Advisor)

Date