UNIVERSITY OF NAIROBI

FACULTY OF ENGINEERING

DEPARTMENT OF CIVIL & CONSTRUCTION ENGINEERING

CHARACTERISTICS OF CINDER GRAVEL AS ROAD PAVEMENT CONSTRUCTION MATERIAL IN MERU COUNTY, KENYA

BY

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A Thesis submitted for Examination in Partial Fulfilment of the Requirements for the Award of the Degree of Master of Science in Civil Engineering of the University of Nairobi

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ABSTRACT

The availability of suitable road construction materials that meet the specification requirements are becoming scarce and therefore the use of marginal materials presents challenges during construction and performance when used in the construction of road projects. This study exploited the gap that exists by investigating the engineering properties of cinder gravel sourced from Meru County in Kenya to test the suitability of the material for road pavement construction of Low Volume Sealed Roads (LVSRs). The study established that neat cinder gravel in its natural state was non-plastic and poorly graded due to deficiency in fine particles <0.075mm (μm). Cinder gravel was blended with locally available fine material (red soil and weathered rock) to improve its engineering properties. The optimum blending ratio of 90% cinder + 10% weathered rock met the requirements for natural subbase and base materials for LVSRs.

The strength and grading properties of cinder gravel for the optimum blending ratio were evaluated at different levels of compaction. The study showed that the Maximum Dry Density (MDD) of the blended material increased with the level of compaction indicating better interlocking of the particles of the material. Similarly, the strength (unsoaked CBR) of blended cinder gravel increased with the level of compaction showing that repeated scarification and compaction of the material improved the strength. There was a gradual increase in the Plasticity Index with the number of compaction cycles of the material due to the breakage of cinder gravels and blending material into finer particles with further compaction. For soaked specimen, the CBR decreased as the cycles of compaction increased because with the ingress of water the finer particles of cinder gravel dispersed and lost the interlocking properties observed with repeated cycle compaction for the unsoaked material.

An evaluation of the shear strength of cinder gravel and particle size at different levels of compaction for the optimum blending ratio showed that the shear strength of cinder gravel decreased with compaction cycles due to the decrease in the angle of shearing resistance (φ). The study established that even though cohesive properties of the material improved with compaction cycles, it did not result to increased shear strength of cinder gravel due to the decrease in the angle of shearing resistance (φ). In conclusion cinder gravel sourced from Meru County was suitable for road pavement construction material for LVSRs with blending with locally available weathered rock. Repeated cycles of compaction of blended cinder gravel improved the mechanical properties of the material including the cohesiveness of the material as the particles became finer.

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ABBREVIATIONS

ABBREVIATION	MEANING
AADT	Average Annual Daily Traffic
ASAL	Arid and Semi-Arid Lands
AASHTO	American Association of State Highways and Transport Officials
BSI	British Standards Institution
BP	Borrow Pit
CBR	California Bearing Ratio
CESA	Cumulative Equivalent Standard Axles
ERA	Ethiopian Roads Authority
ESA	Equivalents Standard Axles
G25, G30, G50 & G80	Granular materials with minimum CBR strength of 25 after 4 days soak and so on for G30, G50 & G80
KRB	Kenya Roads Board
LL	Lower Limits
LL	Liquid Limit
LVSRs	Low Volume Sealed Roads
LS	Linear Shrinkage
iTRARR	International Transport & Road Research
MC	Moisture Content
MDD	Maximum Dry Density
mesa	Million Equivalent Standard Axles
MoTIHUD	Ministry of Transport, Infrastructure, Housing and Urban Development
MT&RD	Materials Testing & Research Division
N/A	Not Applicable
No.	Number
NP	Non-Plastic
OMC	Optimum Moisture Content
PL	Plastic Limit
PI	Plasticity Index
PM	Plastic Modulus
ReCAP	Research Community Access Program
RDM	Road Design Manual
RHS	Right Hand Side
R^2	R-squared (Coefficient of correlation)
SL	Shrinkage Limit
TRL	Transport Research Laboratory
TRRL	Transport and Road Research Laboratory
UL	Upper Limits

GLOSSARY OF ACRONYMS

ACRONYM	MEANING
cm	centimetre
C_u	Uniformity of Coefficient
c_u	Unit of cohesion
Div	Divisions
g/cc	grammes per cubic centimetres
gm	grammes
I_p	Plasticity Index
Kg	Kilogramme
Kg/m^3	Kilogramme per cubic metre
Kg/cm^2	Kilogramme per square centimetre
m	mitre
Max	Maximum
Mg/m^3	Mega grammes per cubic metre
Min	Minimum
ml	millilitre
mm	millimetre
R^2	R-squared
W_l	Liquid Limit
W_p	Plastic Limit
%	percent
µm	micrometre
τ	Shear strength
σ	Normal Stress
φ	Angle of shearing resistance
T5-0	Design Traffic Class 500,000 – 1million CESA
T5-1	Design Traffic Class 250,000 – 500,000 CESA
T5-2	Design Traffic Class 100,000 – 250,000 CESA
T5-3	Design Traffic Class 25,000 – 100,000 CESA
T5-4	Design Traffic Class <25,000 CESA
µm	Micro metre

Chapter One: Introduction

1.1Background of the Study

Naturally occurring gravels are abundant source of road construction materials but majority requires improvement with cement or lime to be suitable for high traffic pavement layers. These treatments are expensive and large financial and environmental benefits can be achieved if properties of locally available materials such as cinder gravels can be improved through mechanical stabilization (blending) to improve the engineering properties (Berhanu, 2009). Options of blending should therefore be explored before resorting to treatment for cost effectiveness in the delivery of road works.

According to the Kenyan Road Design Manual Part III (1987), many different types of gravels exist in Kenya namely lateritic gravels, quartzitic gravels, calcareous gravels, some forms of weathered rock, soft stone, coral rag and conglomerate. Various types of sand and silty or clayey sands are also found. In order to minimize construction costs, natural locally available road materials should be used as much as possible. Use of cheap locally available materials should be a priority before considering importation of distance material due to haulage costs.

A review of a number of Material Testing & Research Division (MT&RD) reports on the design or strengthening of the major Kenyan roads has shown that the soils which occur most frequently at the subgrade level can be classified in seven categories that have been designated as follows (Courteille and Serfass, 1980);

(i) Red friable clays,
(ii) Sandy clays on volcanic rocks,
(iii) Ash and pumice soils,
(iv) Sandy clays on basement rocks,
(v) Silty loams on gneiss and granite,
(vi) Coastal sands, and
(vii) Black heavy clays (Black cotton soils).

Kenyan subgrade soils for pavement design can be grouped into six bearing strength classes as given in the Kenyan Pavement Design Guidelines for LVSR of 2017. Classification of some of the subgrade materials in Kenya based on the bearing strength is as also given in the Kenyan Road Design Manual Part III of 1987.

The physiographical features of the Northern part of Eastern Province of Kenya from Geological Survey by Pulfrey and Walsh (1969) consist of Precambrian schists and gneisses and recent lava plains and volcanoes. There are scattered residual hills and mountains of the older rocks where large volcanic piles are found and include Marsabit (1680m) and Mt. Kulal (2257m). Volcanic rocks of Pleistocene or presumed Pleistocene age are found in several parts of the highlands, south-east of Nairobi and in northern Kenya. Availability of residual hills and volcanic cones in Marsabit, Mt. Kulal East of Lake Turkana, Nyambeni and Chyulu ranges calls for prospecting volcanic sands and tuffs, volcanic ash and pumice soils and cinder gravels as material for road construction. The formation of cinder materials is through volcanic activities, lava and pyroclastic (Pulfrey and Walsh (1969). According to Gareth et al (2018), in Kenya, cinder materials are found in Marsabit with about one hundred and eighty (180) volcanic cones and twenty two (22) volcanic maars (Kenya Roads Board et al (2018).

According to the guideline by Ethiopian Road Authority (2018), cinder gravel is piece of vesicular lapilli (gravel-sized pieces of solidified pyroclastic lava) ejected from a volcanic vent during an eruption, with the appearance of cinder. Vesicularity is the degree to which the material is vesicular. Higher vesicular material (such as pumice) will have lower densities, and hence lower compressive strengths, than less vesicular material. Whilst some variation can be expected in the properties of naturally occurring materials, the use of cinder gravels is further compounded by the unusually high variability, not just between different sources but also equally within the same source. These characteristics present particular challenges in identifying cinder deposits that have sufficient quality and uniformity for potential use in road construction (Ethiopian Roads Authority, 2018).

Cinder gravel being readily available and an abundant material in certain areas of volcanic occurrence offers potential for use in low-volume road construction and rehabilitation (Gareth et al., (2018). The challenge has been to mitigate or control the variability in its engineering characteristics particularly grading, plasticity, porosity and strength to meet specification requirements for pavement construction layers (Gareth et al., (2018). There is no much research that has been done in Kenya on the suitability of cinder gravel for road pavement construction material. There was need to undertake a study on the engineering characteristics of the material to offer customised solution for use in road construction. The study was limited mainly to prospecting and investigations of cinder gravels in Meru County (Nyambene Hills) where the knowledge gained could be extended to other parts where the material is readily available in the country.

1.2 Problem Statement

The socio-economic development of a country requires elaborate infrastructure development framework as an enabler to economic growth. The Kenyan Government in 2015 initiated the Low Volume Sealed Roads 10,000 Programme (LVSR) to support primary growth sectors of the economy. This programme requires huge funding while at the same time construction costs have been increasing due to scarce construction materials and use of non-sustainable construction methods.

Construction of roads in particular is becoming increasingly expensive because existing gravel sources are depleting due to population increase and changes in land use; and rates of gravel loss are becoming higher due to traffic attrition, environmental degradation and high frequency of re-gravelling. The availability of marginal materials in construction sites has forced players in the construction industry to incur huge haulage distances of good quality materials. Thus, further increasing the cost of construction and long period for the projects to be completed. This is due to limited research on locally available materials and there is need for a paradigm shift in bringing the cost of construction low through research on new materials and innovative methods of construction.

Volcanic materials such as volcanic sands and tuffs, volcanic ash and pumice and cinder gravels are readily available in some parts of the country. However, these natural materials are hardly used in road construction due to detrimental properties in meeting the specification requirements or its use may compromise the durability and performance of road pavement. Research that has been done elsewhere has shown that volcanic materials particularly cinder gravels can be improved by blending with naturally occurring materials. During the reconnaissance survey it was established that the material is extensively used in Meru without improvement in its deficiency in binding properties which results in gravel loss on carriageways.

1.3 Research Questions

Arising from the foregoing, the following formed the research questions.

(i) What are the options to improve the engineering properties of unsuitable material for use in pavement construction for Low Volume Sealed Roads considering the prohibitive initial costs?

(ii) Does the engineering characteristics of cinder gravel improve with repeated cycles of compaction and is it beneficial on construction site?

(iii) What is the relationship between shear strength and particle size of cinder gravel?

1.4 Research Objectives

The overall objective of the study was to investigate the engineering characteristics of cinder gravel as road pavement construction material in Meru County, Kenya. In order to achieve the broad objective, the study had the following specific objectives:

(i) To evaluate the engineering characteristics (Particle size distribution, Atterberg's limits, Maximum Dry Density (MDD), California Bearing Ratio (CBR)) of neat and blended cinder gravel for suitability as road pavement construction material;

(ii) To evaluate the strength and grading properties of cinder gravel at different levels of compaction;

(iii) To investigate the relationship between shear strength of cinder gravel and particle size using shear box test.

1.5 Justification

The supervision of Thanantu Bridge – Kagwata – Mulika (D118) Road, a LVSR project which was on-going in Upper Meru faced challenges of locally available quality sub base and base course material. The locally available gravel material was majorly volcanic ash which even though had higher strengths were lighter in densities, were of low plasticity and had varying properties within the same quarry. Attempts were made to blend the material with red soils to improve its properties (grading and plasticity). However, the material was not fully exploited due to variability in its engineering characteristics and it was such challenges that the research study investigated and recommended solutions to road practitioners.

The study was aimed at proposing solutions to road practitioners in areas where the material was readily available for its use. Findings from the research will also contribute to the use of cinder gravels for road construction, rehabilitation and maintenance by Road Authorities and County Governments in areas where there is abundance of the material in Kenya. This will bridge the gap for depleting good quality lateritic gravel and ultimately lower the cost of construction. The study recommended the optimum blending material proportions to improve the engineering properties of cinder gravels for use in pavement layers for road construction. In addition, the study investigated how strength of cinder gravel varies as the particles breakdown and became finer when compacted. Laboratory tests were undertaken to simulate the road pavement performance where the material will be found to be suitable for road pavement construction. As the road is trafficked repetitive loading is induced on the pavement layers and the material will be expected to withstand vertical and lateral stresses thereby delaying initiation of cracks on the pavement. As a conclusion, the study exploited the gap that existed by investigating the engineering properties of cinder gravel sourced from Meru County in Kenya to test the suitability of the material for road pavement construction of Low Volume Sealed Roads (LVSRs).

1.6 Research Scope

The study focused on laboratory tests for grading, Atterberg's limits, MDD and CBR of neat and blended cinder gravels in order to evaluate the engineering properties of the material for use in pavement layers in road construction. The study also investigated the behaviour of cinder gravel in strength when subjected to repeated cycles of moulding-compaction. Laboratory tests on the material were carried out at MT&RD and at University's Soil Laboratory. From laboratory experiments, the study developed optimum properties for use of cinder gravels from representative samples and borrow pits in Meru County. The study did not include in-situ test on site as the material was coarse and the sample to be used in the laboratory was disturbed.

Chapter Two: Literature Review

2.1 Overview

The literature review was divided into two sections: The first part covered briefly the design standards and guidelines that were referred to in evaluating the suitability of cinder gravel as road pavement construction material while the second part presents the formation, physical characteristics and engineering properties of cinder gravel from past research done elsewhere.

2.2 Design of Pavements

Roads are valuable assets as they are the primary means of communication and transportation. The major structural load-carrying elements of roads are the pavements. The road is composed of the carriageway, pavement structure, drainage structures and road furniture. The pavement structure is the structural component of the road, composed of well compacted layers of materials with different load bearing capacity (Jitareekul, 2009).

A typical pavement structure for roads with design traffic in excess of 1 million ESA is illustrated in Figure 2.1.

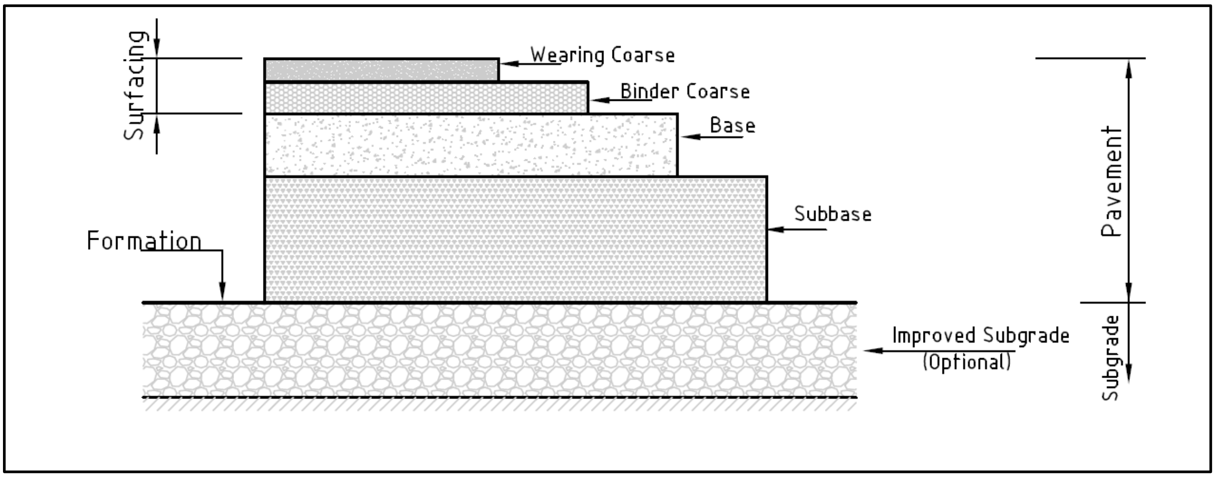

Figure 2.1: Pavement Terminology (Ministry of Transport and Communication, Roads Department, 1987)

There are two types of pavements;

(i) Flexible pavements; A flexible pavement is one with low flexural strength, thus the external load is largely transmitted to the subgrade by the lateral distribution with increasing depth. Because of the low flexural strength, the pavement deflects if the subgrade deflects. Flexible pavement is the most preferred pavement type in Kenya because of low initial construction cost.

(ii) Rigid pavements; A rigid pavement does flex under loading. Rigid pavement derives its capacity to withstand loads from flexural strength or beam strength, permitting the slab to withstand minor irregularities in the subgrade or subbase upon which it rests. In rigid pavement, the concrete slab is the main load bearing layer, therefore the performance of the pavement is more of a factor of the concrete slab rather than the subgrade.

2.2.1 Flexible Pavement Structure

The flexible pavement consists of a relatively thin wearing surface built over a base layer and subbase layer, and they rest upon the compacted subgrade. According to Gichaga and Parker (1988) the strength of a flexible pavement is derived from the composite effect of the various layers of the pavement. These layers are arranged in such a way that the layer strength increases from the subgrade upwards, with the strongest material being placed on the surface. A flexible pavement comprises of the following layers; Surfacing layer, Road base, Subbase and Subgrade. The surfacing layer consists of two layers;

(i) The wearing course which is generally impervious in order to prevent water from entering the pavement structure and into the subgrade. It also provides a skid resistant surface.

(ii) Base course or binder course which provides a good surface on which to construct the wearing course.

The base course is the main structural layer whose purpose is to distribute traffic loads so that stresses and strains developed in the subgrade and subbase are within the capacity of the materials in those layers. The subbase is also a load distributing layer but slightly weaker than the road base. It also helps in protecting the subgrade. The subgrade is the natural ground or improved with borrowed material upon which the pavement structure is constructed.

2.2.2 Design of Flexible Pavements

Flexible pavement design is basically a structural design exercise whereby one ensures that traffic loads are distributed so that stresses and strains developed at all levels in the pavement structure and the subgrade are within the capabilities of the materials at each level. In simple terms the stresses created by the wheel loads should not exceed the capacity of subgrade. The design process consists of two phases;

(i) Determination of thicknesses of pavement layers having certain mechanical properties which can be obtained from catalogues and;
(ii) Determination of the composition of the materials that will provide those properties.

The overall thickness of the pavement structure will depend on the traffic loading to be carried, quality of subgrade, the mechanical properties of the construction materials and the prevailing climate. The loading due to traffic is considered in terms of magnitude and repetitions of traffic loads. Environmental elements such as rainfall and temperatures must be addressed during design and construction stages. Consideration of temperatures is important because some road materials are affected by changes in temperature and in the case of flexible pavements, the performance of bituminous layers will be a function of the pavement temperature as strength properties such as stiffness will decrease with increase in temperatures.

Road surface drainage must be addressed at design stage for which the designer will consider the rainfall pattern and the expected run-off and hence design the required drainage facility.

Flexible pavement design methods can be divided broadly into empirical and analytical methods. Due to the accumulated experience in the use of empirical and semi-empirical methods these methods continue to be used more than analytical methods (Gichaga and Parker (1988).

2.2.3 Empirical and Semi-Empirical Flexible Pavement Design Methods

Empirical and semi-empirical design methods have been developed on the basis of long-term pavement performance for specific traffic loading and environmental conditions. This means that for as long as conditions for which these methods were developed prevail, the performance of the pavement should be satisfactory.

2.2.4 Analytical Design Methods

In this method, the idea is to design a pavement structure using theories mainly elastic theory where given the materials available for construction the thicknesses of the various layers of the pavement structure are designed in such a way that critical factors such as stresses, strains and deformations caused by the traffic loading are within permissible limits. If the results are not within the permissible stresses, then it is repeated as the design must be economically viable. The essential features of analytical design of flexible pavements are;

(i) Selection of a suitable elastic or visco-elastic model to be used,
(ii) Solutions to the equations for stresses, strains and deformations in the model,
(iii) Characteristics and mechanical properties of the construction materials and
(iv) The definition of design criteria in terms of stress, strain and deformation.

Analytical design methods have not yet gained wide acceptance by highway designers mainly because of the complexity of the mathematical models involved. Other reasons for the lack of acceptance relate to inadequate material characterisation which prevents a designer from carrying out a theoretical analysis with confidence.

2.2.5 Rigid Pavements

Rigid pavements are constructed using either mass concrete or reinforced concrete slabs. They bear traffic loads by beam action. The structural capacity of the pavement adopts the stiffness modulus of concrete, enabling the base to carry much load and minimal transmission to the road subgrade layer. Rigid pavements are suitable for heavy traffic and typical pavement structure is illustrated in Figure 2.2.

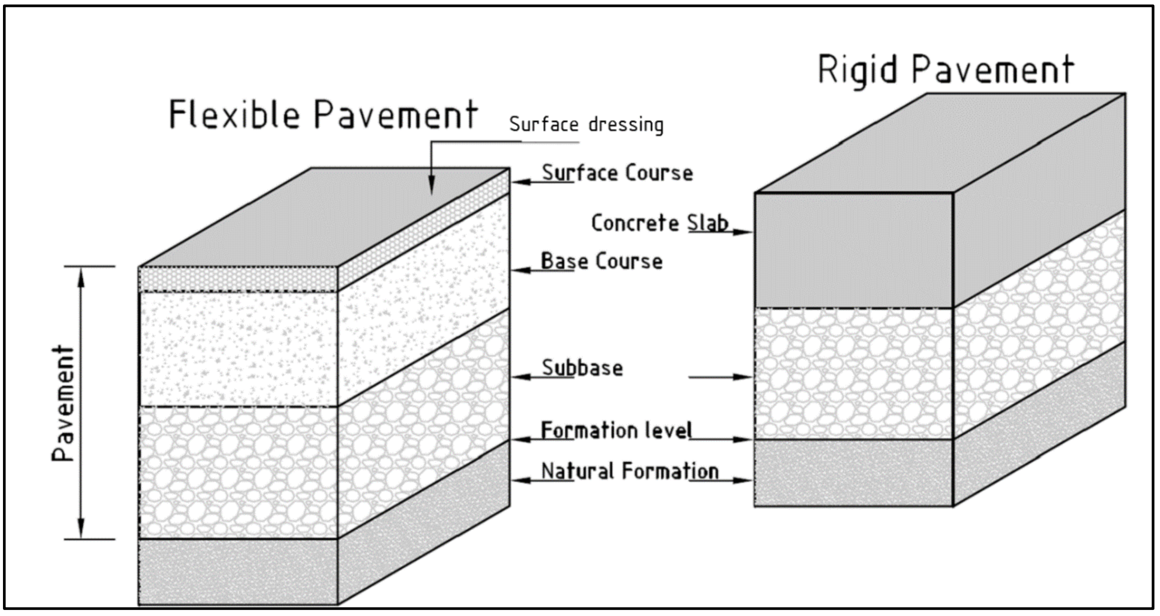

Figure 2.2: Flexible and Rigid pavements (Britannica Encyclopaedia, 1999)

2.3 Low Volume Sealed Roads (LVSRs)

Low volume roads are considered to have a design traffic loading of less than 1 million ESA as implied by the following definitions from different countries;

(i) A *very low-volume local road* is a road that is functionally classified as a local road and has a design average daily traffic volume of 400 vehicles per day or less (American Association of State Highway and Transportation Officials (AASHTO), 2001);

(ii) Low volume roads are defined as those roads carrying; Up to about 300 vehicles per day and less than about 1 million equivalent standard axles (Mesa) over its design life (Ethiopian Roads Authority, 2018);

(iii) Low volume roads are considered to have a design traffic loading of less than 1 million equivalent standard axles over its design life (Ministry of Transport, Infrastructure, Housing and Urban Development (MoTIHUD), 2017). The design period usually 15 years is the time the pavement will require strengthening in order to carry traffic for a further period.

The Kenyan Road Design Manual Part III of 1987, does not provide pavement structures for traffic loading below 250,000 cumulative equivalent standard axles (CESA). Furthermore, the pavement structures provided for traffic loading between 250,000 and 1,000,000 CESA (T5) are based on permissible subgrade strains for 1,000,000 CESA and are consequently mostly overdesigned.

Based on RDM Part III, roads with traffic below 250,000 CESA have to be designed for improvement to gravel standards. Construction of gravel roads is however becoming increasingly expensive because existing gravel sources are depleting due to population increase and changes in land use, plus rates of gravel loss are becoming higher due to traffic attrition, environmental degradation and high frequency of re-gravelling. For sustainability, gravels roads require to be sealed under stage construction method. The pavement terminology for LVSRs is illustrated in Figure 2.3. The capping layer is equivalent to the term “improved subgrade” in RDM Part III.

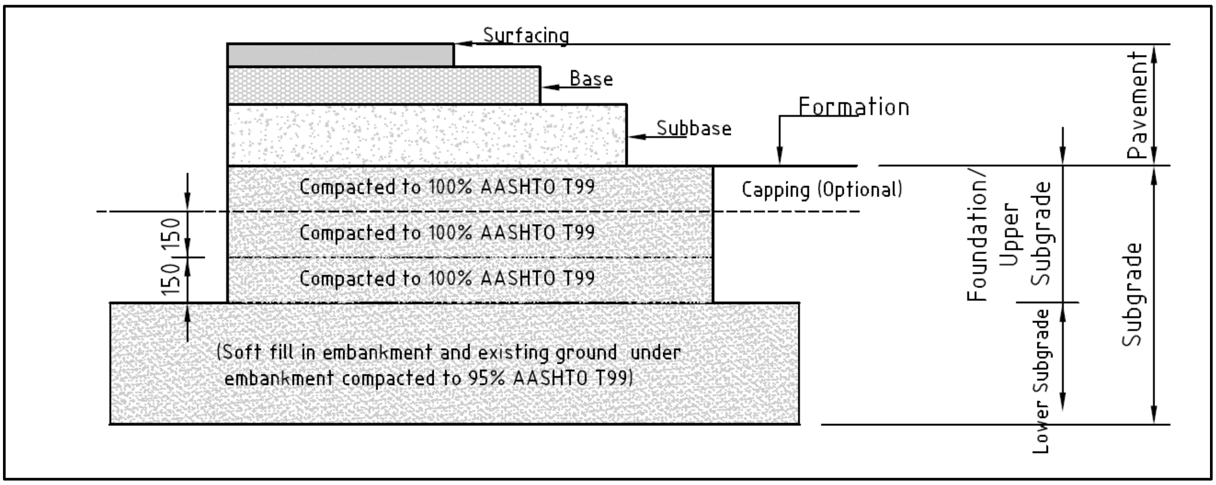

Figure 2.3: Pavement Terminology for Low Volume Roads (MOTIHUD, 2017)

2.4 Pavement Design Standards and Guidelines

The design of flexible pavements consists of two phases;

(i) Determination of thicknesses of pavement layers having certain mechanical properties which can be obtained from catalogues and

(ii) Determination of the composition of the materials that will provide those properties.

The overall thickness of the pavement structure will depend on the traffic loading to be carried, quality of subgrade, the mechanical properties of the construction materials and the prevailing climate. The loading due to traffic is considered in terms of magnitude and repetitions of traffic loads.

The Ministry of Transport and Infrastructure in Kenya has developed manuals and guidelines to guide the design of flexible pavements; the Kenyan Road Design Manual Part III (1987) and the Kenyan Pavement Design Guideline for Low Volume Sealed Roads (LVSRs) (2017). The two standards were the benchmark in checking design requirements for suitability of cinder gravel as road pavement construction material during the study.

2.4.1 Traffic Loading for Pavement Design

The designer determines the Average Annual Daily Traffic (AADT) using the road and the growth in traffic over the design period. Axle loading for design is computed from the cumulative equivalent standard axles (CESA) for each vehicle type expected to use the road in the design period.

The Kenyan Road Design Manual Part III, 1987, does not provide pavement structures for traffic loading below 250,000 cumulative equivalent standard axles (CESA). The manual recommends that such roads be improved to gravel standards construction. This is unsustainable due to requirement for re-gravelling after every 2-3 years. There is need therefore to seal the low trafficked roads using the Low Volume Sealed Roads (LVSRs) concept in order to minimise life cycle costs of rural roads.

2.4.2 Subgrade Bearing Strength Class for Pavement Design

Classification of some of the subgrade materials in Kenya based on the bearing strength is given in the manuals and the Guidelines for Pavement Design. According to the Kenyan Road Design Manual Part III, some of the ash and pumice soils have a very low maximum dry density and a lower Young's Modulus than might be expected from the measured CBR values. Such soils (standard compaction MDD less than 1.4 Mg/m^3) cannot be classified for pavement design purposes on the basis of CBR only.

2.4.3 Pavement Foundation / Subgrade

The Pavement foundation is the platform on which the pavement rests and is known as the subgrade. The foundation (subgrade) comprises 300mm below formation in embankments or cuttings compacted to 100% MDD (AASHTO T99) in two layers of 150mm thickness. Where the subgrade material is not complying with the strength requirements of the selected foundation class a capping layer shall be introduced to improve the foundation class (MoTIHUD, 2017).

The pavement foundation classes are categorised in terms of the surface stiffness modulus at optimum moisture content and are given in the Pavement Design Guidelines for LVSR as well as the corresponding equivalent subgrade class. According to the Guideline, the use of lightweight materials such as cinder and fly ash minimises settlement by reducing the weight of embankment.

2.4.4 Standard Pavement Structures

The standard pavement structures for higher traffic roads and LVSRs are as given in the Kenyan Road Design Manual Part III and the Kenyan Pavement Design Guideline respectively. There are fourteen (14) pavement combinations and the applicable traffic classes in the catalogues of the guideline which a designer can select. However, it is at a discretion of the designer to adopt a suitable pavement structure that is economical, technically sound and fit for local conditions (pavement materials, drainage, maintenance requirements, environmental consideration and road safety). Noting that base quality natural gravels are scarce in Kenya, mechanical stabilisation is necessary to achieve a material complying with the specification requirements (MoTIHUD, 2017).

2.5 Cinder Gravels

2.5.1 Definition of volcanic cinder

Volcanic cinders are deposits of granular materials from past volcanic activity. The material may be found in cone-shaped residual hills or large concave depressions on tops of volcanic mountains. Cinder vary in color even within the same source and may be brownish, reddish, greyish or blackish gravel sized particles. The particles also vary in sizes and shapes and may be large size of up to 500mm in diameter to much smaller particles as sand and silt of less than 2mm in diameter. However, some cones may exhibit uniform particles with the largest size being 30mm in diameter. Other physical features of cinder gravels are low density or light weight, rough vesicular surface and highly porous material (Ermias, 2019).

The lapilli ejected during basaltic eruptions varies considerably in its vesicularity as shown in Plate 2.1, but densities are much greater than those of pumice materials, and usually between 1.2 and 2 g/cc. The more vesicular the material the more it is lighter in density. These fragments are black or dark grey in colour when initially ejected but oxidise to dark red and red-brown following contact with the atmosphere. Geologically, this material is referred to as scoria, though it is also known as cinder due to it having a similar appearance to charcoal or slag (Ethiopian Roads Authority, 2018). Ermias (2019) in his thesis observed that an advantage of cinders for suitability as road pavement construction material is the relative ease with which it can be extracted from the borrow pit by mechanical shovel, pick hoes and other hand tools. However, for harder material a bulldozer or ripper is required to open up a working face.

 

Low-vesicularity cinder gravel	Moderate-high vesicularity cinder gravel

Plate 2.1: Cinder gravel of varying vesicularity (Ethiopian Roads Authority, 2018)

2.5.2 Locating cinder gravel deposits

Most cinder gravel deposits are found within volcanic cones, which give the clearest surface evidence of the location of these materials. However, predicting the type (quality) of material that lies within them either from their location or shape is difficult due to the extreme variability of the material contained within the cones (Ethiopian Roads Authority, 2018).

Scoria (cinder gravel) is found in regions where volcanic activity has occurred. It is a porous material ranging from black to dark red in colour and is formed from volcanic eruption as gases escapes from the residual rock (Geology Science, 2021). The material is found around the vents of volcanoes inform of cone-shaped hills known as volcanic ash cones. Regions of volcanic activity on the Earth surface are dotted with many cones known as volcanoes. Scoria-producing volcanoes usually have short eruptions, and the residual material is often used as lightweight aggregates for landscaping and drainage works or for casting low strength non-structural concrete (Geology Science, 2021).

2.5.3 Physical characteristics of cinder (scoria) and pumice

Table 2.1 gives physical characteristics that distinguishes cinder gravel from pumice which are both volcanic materials and often found within the same locality. The main difference is in the color, density, vesicularity, roughness and presence of mineral crystals in the surface of the material.

Table 2.1: Physical characteristics of cinder (scoria) and pumice

Scoria (cinder)	**Pumice**
Scoria are dark coloured ferromagnesian minerals / volcanic glass. It is vesicular (contains gas cavities) basaltic lava with small (<1mm) vesicles	Pumice are light coloured minerals including quartz and feldspar volcanic glass. It is in rock foam with a lot of air in its structure and often floats on water
It is usually black or dark grey to reddish brown	Is white to light grey or light tan
Thick walls of scoria make it heavy enough to sink in water	Has higher concentration of trapped bubbles enabling it to float on water
It is relatively low in density due to its vesicles	Pumice is much lighter than scoria
It has larger vesicles with thicker walls	Weathered pumice loses its glassy appearance when exposed to water but is still light weight
Using a hand lens, very tiny mineral crystals can be observed in scoria	Feels abrasive or rough against the human skin
It is commercially used as a high-temperature insulating material. It also has applications in landscaping and drainage.	
It is used in landscaping, drainage, rip-rap or low-quality road metal.	

(Geology Science, 2021).

2.5.4 Past Research Studies on Volcanic Cinder Gravel

There are examples in Ethiopia to demonstrate the successful use of cinder gravel as material for subgrade, capping, and in sub-base layers on both high volume and low volume roads. However, Newill and Kassaye (1987) showed that these materials can also be used in the base course layer in low volume roads. Cinder gravels thus have the potential for use in the construction of all the pavement layers of low volume roads (Ethiopian Roads Authority, 2018).

Newill and Kassaye (1980) undertook preliminary investigation of cinder gravels from approximately 70 cones in Ethiopia. The investigations covered field survey, laboratory investigation and performance monitoring of a cinder gravel road. The main conclusion from the study was that blending the material with locally available finer material improved the engineering properties particularly grading and plasticity. The researcher observed that this could be the treatment required for improving suitability of the material for road pavement construction. Performance monitoring of gravel road under the study confirmed that cinder gravel may improve in its engineering properties of grading and strength even under traffic compaction. The material may be suitable for dry compaction in arid and semi-arid areas where availability of water for construction is scarce. The results from the preliminary investigations indicated that cinders could provide useful road construction materials especially for gravel roads. However, further research needs to be done on the suitability of cinder gravels under different traffic and climatic conditions for full-scale exploitation as road pavement construction material (Newill and Kassaye, 1980).

2.5.5 Engineering properties of cinder gravels

(a) Particle Size Distribution

The particle size distribution of a soil is presented as a curve on a semilogarithmic plot, the ordinates being the percentage passing the sieve size on the abscissa (Craig R.F, 2004). For a granular material the shape of the particle size distribution curve or grading curve is an indicator of the distribution of particles in the soil sample (Smith and Smith, 1988). The grading curve could either be well graded or poorly graded depending on the shape of the grading curve. For a poorly graded soil, if the grading curve is steep then the material has particles sizes of a limited range or of similar sizes and is said to be closely graded or uniformly graded. On the contrary, if the material has large percentages of larger and smaller particles and a less percentage of intermediate particles exhibiting significant flat section or plateau grading curve then the soil is gap graded (Smith and Smith, 1988).

According to research by Maniyazawal F.W. (2020) and (Seyfe and Geremew, (2019) it was found out that the gradation of cinder gravel doesn’t satisfy the specification bound limits but when blended at 30% and 40% respectively the sieve analysis curve lies within the intended specification limits. The blended cinder gravel showed improved engineering properties when subjected to laboratory test analysis. Cinder gravels are generally coarse material and lacks finer particles (less than 4.75 mm). Upon compaction, the larger particles disintegrate to smaller size particles thus contributing to the fines content and improved grading, therefore particle size distribution tests should be done after compaction, even for blended cinder gravels (Ethiopian Roads Authority, 2018).

(b) Atterberg’s Limits

For cohesionless soils such as gravel or sand, the strength and compressibility of a soil are only slightly affected by a change in water content whereas a cohesive soil, silt or clay, tends to become considerably stronger and less compressible, less easy to mould, as it dries out

(Smith and Smith, 1988). The water content at which the soil stops acting as a liquid and starts acting as a plastic solid is known as the Liquid Limit (W_L). As the soil dries up or further moisture is driven out of the soil it becomes possible for the soil to resist large shearing stresses and its engineering properties improves. Eventually the soils exhibit no permanent deformation and simply fractures with no plastic deformation, it acts as a brittle solid. The limit at which plastic failure changes to brittle failure is known as the Plastic Limit (W_P). The Plasticity Index (I_P) is the range of water content within which a soil is plastic; the finer the soil the greater its plasticity index (Smith and Smith, 1988).

$I_P = W_L - W_P$……………………………………………………………**Equation 2.1**

Shrinkage limit is the state where the drying process of the soil is prolonged after the plastic limit has been reached where the soil continues to decrease in volume until a certain value of moisture content is reached. This value is known as the shrinkage limit and at values of moisture content below this level the soil is partially saturated (Smith and Smith, 1988).

The neat cinder has no plasticity index (Ermias, 2019) as the ratio of blending increases the plasticity index increases. This is a result of contribution of fines from the blending materials contributing to plastic properties to the cinder gravels. Most cinder materials are non-plastic; however, occasionally the gravels can be found within a matrix of plastic fines (weathered material) (Ethiopian Roads Authority, 2018).

(c) Maximum Dry Density

By using the AASHTO T180 test method, it is not always accurate or even possible to determine the maximum dry density and OMC of cinder gravels. It is for this reason that the TRRL/ERA 1975 study recommended the method of re-using a single moulded specimen to obtain all five points of the compaction curve (Ethiopian Roads Authority, 2018). For cinder gravels that naturally occur with plastic fines, there is no need to use the method of re-using the specimen. The same applies for non-plastic cinder material blended with plastic fines (Ethiopian Roads Authority, 2018).

Berhanu (2009) on his research study found out that the maximum dry density of blended cinder increases with increases in the blending proportions. The study concluded that a 12% blending ratio showed optimum engineering properties. At higher blending proportions beyond the 12%, the MDD dropped significantly indicating that it was the optimum percentage proportion to cover up the deficiency of fines lacking in the cinder gravel.

(d) California Bearing Ratio

Neat cinder gravels exhibit a wide range of CBR values. However, since in most cases neat cinder gravels will be blended with other materials to improve the particle size distribution and to aid compaction, the CBR should be treated as a method of selecting suitable material for blending with other finer materials (Ethiopian Roads Authority, 2018).

Maniyazawal (2020) and Berhanu (2009) found out that the CBR of neat cinder is lower than that of cinder blended with fine soils. However, cinder gravel blend replacement with 30% conventional base course material and 12% replacement with fines showed increases in CBR. This shows that the mechanical characteristics of blended cinder has higher interlocking mechanisms thus they are able to resist the loads which guarantees the durability of the pavement.

2.5.6 Mechanical Stabilization (Blending)

Mechanical stabilisation (also known as blending) refers to the process of combining two or more granular materials with the aim of obtaining a material of better engineering properties (usually bearing strength, and sometimes, improved plasticity index). Blending of natural gravels is usually done either to improve the bearing strength of the natural gravel through improving its particle size distribution or improving its plasticity index (Ethiopian Roads Authority, 2018). Mechanical stabilisation has a vital role to play in the use of cinder gravels in all pavement layers. This is because cinder gravels often lack fine particles, and where present they are, in general, non-plastic (Ethiopian Roads Authority, 2018).

For base course layers, the addition of fine particles may result in a decrease of the required CBR. The CBR would then have to be increased by the addition of stronger material such as crushed rock. It is also possible in certain circumstances to use crushed stone to improve both the particle size distribution and the strength properties of the material (CBR). This may require much more crushed rock than the alternative of using the combination of cinder gravel, fine gravel/clay and crushed rock (Ethiopian Roads Authority, 2018).

Material for blending (mechanical stabilisation) is usually located at the same cinder cones as the material requiring mechanical stabilisation. These materials often possess plastic properties although their availability cannot be guaranteed. The Atterberg Limits of this material should always be tested before further blending tests (Ethiopian Roads Authority, 2018). Suitable blending material should be plastic for particles passing 425 μm sieve and retained on the 75 μm sieve size.

2.5.7 Cement and Lime Stabilisation

In the design and construction of low volume roads, the use of chemical stabilisation (lime, pozzolans, cement and others) may present a prohibitive initial cost. However, there may be circumstances where it is not possible to find materials available in their natural state or by modifying them by mechanical stabilisation, and meet the required specifications. Under such circumstances and especially for the design of roads carrying higher traffic levels, lime or cement stabilisation should be considered. However, mechanical stabilisation of cinder gravels with other natural materials should be tried before the use of chemical stabilisation to reduce construction costs. (Ethiopian Roads Authority, 2018).

2.5.8 Stabilization of cinder gravels for High Trafficked Roads

According to Newill and Kassaye (1980), a joint research project undertaken between the Ethiopian Roads Authority and Transport and Road Research Laboratory (UK) on characteristics of cinder gravels showed that the two most important factors that impact on the engineering characteristics of cinder gravels are grading and the strength of the gravel particles (Berhanu, 2009). The study recommended further research on the potential use of the abundant natural cinder gravels in pavement layers for high traffic roads by improving their engineering properties (Berhanu, 2009).

The results of the gravel surfaced road sections monitored showed that improved performance can be obtained by mechanically stabilizing cinder gravels with plastic fines (Berhanu, 2009). The results of the joint research work should be further taken up to investigate the potentials use of these abundantly available natural gravels in base course for heavily trafficked roads by improving their engineering properties (Berhanu, 2009). The study of cinder gravel sourced from two sites showed MDD and CBR tests conducted on mechanically stabilized cinder increase in fine soils up to 12% after which the strength of the blended material reduced and 7% when stabilized with cement, which attained the minimum specified strength in Ethiopian Roads Authority Pavement Design Manual for a base course layer for heavily trafficked pavement structure (Berhanu, 2009).

The results from the investigation ascertained that properties of cinder gravel can be improved by stabilisation for use for high traffic base course layers. However, a road performance monitoring is required to study the performance of stabilised cinder gravels against the possible detrimental effects of cracking due to stresses induced by thermal, shrinkage, and traffic effects. The viability of cement stabilised cinder gravels should be assessed for every project versus expenses related to getting quality aggregate and high haulage distance (Berhanu, 2009). The stabilization itself may be prohibitive for low volume sealed roads where cost should be kept at a minimum for the project to be viable.

According to the Kenyan Road Design Part III, 1987, it may be advantageous to mechanically stabilize unsuitable natural gravels, by mixing in sand to reduce the plasticity or stone (crushed or not) to provide hard coarse particles. An addition of 30% of sand or stone is regarded, practically and economically, as a maximum.

2.5.9 Effect of re-using the blended specimen

For some cinder gravels, the strength (as represented by CBR) increases as the particles breakdown during compaction to smaller fractions. This breakdown is achieved through repeated cycles of moulding-compaction. It is important to investigate the effect of re-using the specimen in this manner. For some cinder gravels, the strength keeps increasing with further compaction, for others the strength increases to a maximum value then decreases sharply. If the cinder gravel shows this tendency, then additional benefit is obtained by re-using the specimen during compaction on the project site. Therefore, optimum compaction cycles should be determined (Ethiopian Roads Authority, 2018).

2.5.10 Other engineering application of cinder gravel

The use of locally available, but frequently non-standard, pavement construction materials plays a significant role in the design concept of Low Volume Sealed Roads. Therefore, it is vitally important to use materials appropriate to their role in the road, that is, to ensure that they are neither substandard or wastefully above the standards demanded by their engineering task (J.R. Cook et al., 2002). The availability of marginal material within the vicinity of the construction has also been a great concern in the construction industry (Austroads report, 2018). Cinder gravel is a coarse material and has a wide range of particle size which can be used for various application in the road construction, including drainage of pavement structures, road pavement seats and lightweight embankments (Lemougna et al., 2018). According to the Kenyan Pavement Design Guidelines for LVSRs, the use of lightweight materials such as cinder and fly ash minimises settlement by reducing the weight of embankment (MoTIHUD, 2017).

Cinder gravels offer significant potential for utilization in railway construction due to local abundance and low environmental pollution risk (Qiang Luo et al., 2020). The material was used as railway earth structure during construction of Addis Ababa–Djibouti standard gauge railway on portions / sections entirely in the Rift Valley where the cinder gravel is abundant. The laboratory test results confirmed that the engineering properties of the cinder gravel can be improved by mechanical stabilization techniques before using as fill material for railway earth structures. They concluded that stabilized cinder generally outperforms the baseline fill due to better dynamic stability of the constructed facilities investigated during the study (Qiang Luo et al., 2020).

Cinder gravel being a marginal material may not meet the standards for pavement layers. However, it may provide solution for other applications of induced failures in pavement as a result of drainage challenges inform of drainage blankets. The material may be applicable in subsurface drainage such as arresting water rise through capillary action therefore promoting free draining shoulder. Cinder gravel may be suitable for dry compaction and offer significant reduction in construction costs as well as reduced construction time.

2.5.11 Literature Review Summary and Knowledge Gap

The desk study from the literature review and observation made during the reconnaissance survey in Meru County identified the following gaps in the use of cinder gravel as a road pavement construction material:

(i) The material was used by Road agencies in Kenya to carry out maintenance works without improvement which resulted in gravel loss on the carriageway due to deficiency in binding properties of the material;

(ii) In the three borrow pits prospected the material varied in colour, quality, texture and density and was established that road practitioners in the area abandoned sites before full exploitation of the material which eventually may lead to environmental hazards and safety risks;

(iii)Since research that has been done elsewhere has shown that the material breakdown during compaction, thus generating fines, it was necessary to determine an appropriate blending ratio with locally available material (red soil and weathered rock) to improve the engineering properties;

(iv) The literature review has identified that no research on the material has been done in Kenya yet the material is readily available in areas of volcanic occurrence as well as arid and semi-arid areas (ASAL) of Marsabit and Mt. Kulal East of Lake Turkana where water for compaction and road construction is scarce.

Arising from the research gaps thereof, there was need to undertake further research for the use of cinder gravel as a road pavement construction material. Laboratory tests were required to determine the engineering characteristics of the material which will guide prospecting and exploitation of cinder gravels in the area.

Chapter Three: Methodology

3.1 Overview

The methodology section is divided into three parts; the first part covered the proposed sites for sampling the material for the research study as obtained from reconnaissance survey carried out, the second part describes material collection and sampling procedures while the last part describes the detailed methodology for the tests carried out on the material to achieve the objectives of the study.

The methodology involved both fieldwork and laboratory tests in accredited laboratories Material Testing and Research Division (MT&RD) of the Ministry and University's Soil Mechanics Laboratory. Tests were carried out progressively from grading, Atterberg's limits, compaction tests, strength (CBR), repetitive sample compaction tests and shear box tests. Grading and Atterberg's limits tests of cinder gravel blended with locally available fine material were carried out after compaction and CBR tests because the properties of the material after compaction was markedly different since cinder gravel breaks down to finer particles upon compaction. The results were analysed as soon as the tests were completed so that erroneous results were detected early, and any outliers corrected with a repeat test. The summary of research design flow chart adopted for determination of engineering properties of neat cinder gravel and blended material samples is given in Figure 3.1.

3.2 Outline of Research Activities

The main objective of the research was to investigate the engineering characteristics of cinder gravel as road pavement construction material. Therefore, to achieve the objective of this study, laboratory tests were done. Neat and blended cinder gravel were investigated to assess their physical and mechanical properties whether it conformed to specifications for pavement layers for road construction in the design manuals. The samples collected from various borrow pits identified during the reconnaissance survey were subjected to sieve analysis to check conformance to the requirements in grading, Atterberg's limits to check for their plasticity, compaction tests (maximum dry density) and strength tests (California Bearing ratio). Since plasticity was an important property for binding particles in the material, non-plastic cinder was blended with the fine soil in appropriate proportions to improve grading and plasticity as illustrated in the research flow chart in Figure 3.1.

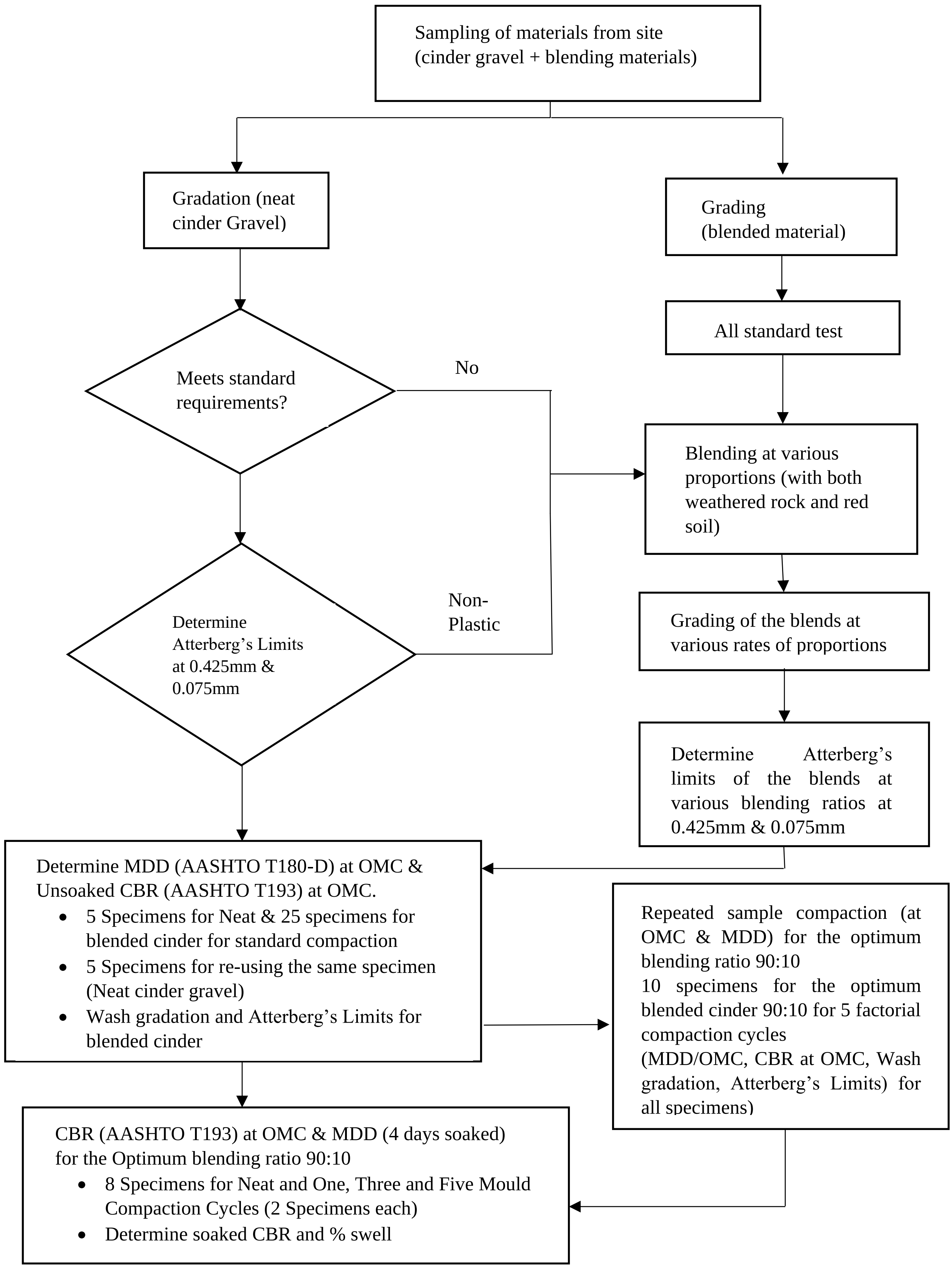

Figure 3.1: Research design flow chart for neat cinder gravel and blended material.

Cinder gravel was subjected to standard compaction tests and for neat non-plastic cinder a single moulded specimen was reused to obtain all the five points of the compaction curve in determining the MDD and OMC. By re-using the specimen, a more definitive curve was obtained as presented in chapter 4. Cinder gravel breaks down to finer material during compaction which might increase or decrease the strength. Therefore, for the blend ratio that met the specification requirements for pavement layers the specimen was subjected to repeated cycles of moulding-compaction at OMC. To replicate the worst-case scenario on site conditions in the field, moulds were soaked for 4 days at maximum dry density and OMC for neat cinder, blended cinder for the optimum blending ratio. The effect of soaking on strength and swelling properties of the material for repeated cycles of moulding-compaction was carried out as illustrated in the research flow chart in Figure 3.1. Data analysis of test results were done using graphical methods.

The study also investigated how the shear strength of cinder gravel behaved when the material was compacted and densification occurred using a shear box test. Such properties can be mirrored on site where optimum level of compaction is controlled to achieve higher compaction of pavement layers. The summary of research design flow chart for determination of engineering properties of neat cinder gravel and blended material samples is given in the research flow chart in Figure 3.1. The Laboratory tests were carried out to the standards given in Table 3.1;

Table 3.1: Types of tests for neat and blended cinder gravel and standards used

Type of Test	**Standards**
Particle size distribution	BS 1377: part 2: 1990, Reference to part 1 for sample preparation
Atterberg's Limits	BS 1377 part 2 1990, Reference to part 1 for sample preparation
Moisture-density relations	AASHTO T 180-D: 2004
California Bearing Ratio	AASHTO T 193-2003
Shear Strength	BS 1377 part 7: 1990, Reference to part 1 for sample preparation

The standard tests were carried out on all the samples to determine the preliminary engineering characteristics of the material in order to inform further tests and investigations to be carried out.

3.2 Prospecting and sampling of cinder gravels in Meru County

During reconnaissance survey it was established that cinder gravels was extensively used in Meru albeit in its natural state without improvement of its properties. The material was used in parking lots in hotels due to its non-plastic nature and low risks of soaking and or pumping of plastic fines into the driveways. The material was also used by road agencies to undertake maintenance works but without improvement and compaction which resulted in gravel loss on the carriageways as was observed in Plates 3.1 and 3.2.

	
Note cinder gravel loss on the c/way when used as GWC without improvement due to deficiency in binding properties	The study of characteristics of cinder gravel was aimed at proposing solutions to the Road practitioners

Plate 3.1:Cinder gravel loss on carriageways when used as GWC without improvement

	
Coarse particles of cinder gravels not easily disintegrable nor affected by changes in moisture and ideal for use in parking lots	Cinder gravel spread in parking lots of Hotels in Meru County

Plate 3.2 Cinder gravels was used in parking lots in hotels due to its low plasticity properties.

During reconnaissance survey the following three existing borrow pits were identified where cinder gravel as well as blending material was sourced;

i. Marega / Thanantu Borrow pit off Meru – Mikinduri Road,

ii. Kitheo Borrow pit, and

iii. Nkinyanga Borrow pit off C91 Meru – Maua road RHS

These borrow pits had been exploited before and the material (cinder gravel) were not fully utilized because of its detrimental properties that made it unsuitable for use as shown in Plate 3.3.

		
Thanantu B/P – Unexploited cinder gravel	Kitheo B/P – Abandoned material site showing crumbling nature of cinder gravel during excavation	Nkinyanga B/P - Material was of variable engineering characteristics

Plate 3.3: Material sites prospected for cinder gravel in Meru County

The prospected material sites depicted abandoned quarries and an uneconomical use of the material due to varying engineering properties. The study leverage on these research gaps to propose solutions for appropriate use of cinder gravel.

Sampling of cinder gravel began with the first borrow pit and progressively on to the other borrow pits once the results of the three objectives were evaluated. Cinder gravel and red soil blending material were first sampled at Thanantu borrow pit off Meru – Mikinduri Road. The red soil was sampled as blending material about 100m from the borrow pit where cinder gravel was sourced while the weathered rock was sampled from Ntoombo material site about 10kms North of Thanantu borrow pit. The site plates taken during sampling is shown in Plate 3.4 while the location map of the borrow pits sampled is given in Figure 3.2.

		
Sampling cinder gravel at Thanantu Borrow pit.	Sampling red soil blending material adjacent to Thanantu BP	Sampling weathered rock blending material from Ndoomo Borrow pit

Plate 3.4: Sampling of cinder gravel and blending material from borrow pits

The borrow pit contained various types of cinder from brown greyish, brown reddish to brown, black in colour. The variability in colour and density was observed from the centre of the borrow pit and the material varied in colour towards the edges. The predominant type; brown greyish at the centre of the borrow pit was sampled for laboratory investigations. It was further observed from the borrow pit that deposit of cinder gravel within the same cone or source varied in physical properties and characteristics from soft to hard material. Arising from this variability some cinder gravel material could meet standard specifications for subgrade / capping layer, subbase or base course pavement layers.

Initially six bags of cinder gravel and two bags each of red soil and weathered rock were sampled in 50kgs bags and transported to Nairobi for testing and further analysis. Due care was taken not to contaminate the samples while transporting to Nairobi and were covered in a tarpaulin against dust and rain. The material was replenished as testing progressed in order to cover all the three objectives of the study.

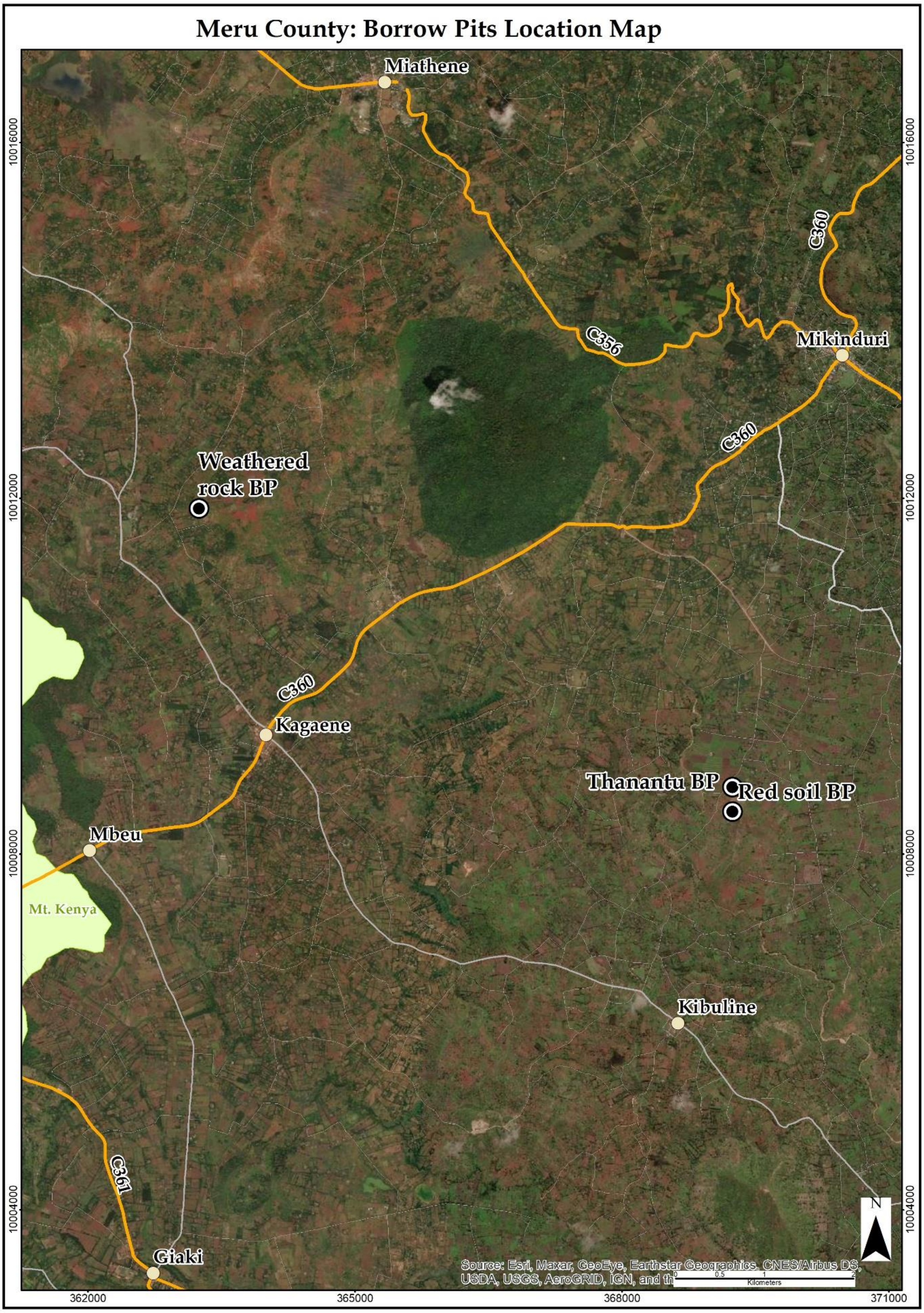

Figure 3.2: Location Map of borrow pits sampled for cinder gravel and blending material in Meru County.

3.4 Sample preparation for testing

Samples of cinder gravel and red soil blending material sourced from Thanantu borrow pit and weathered rock blending material from Ntoombo borrow pit were air dried for 3-4 days in the laboratory in preparation for standard tests as shown in the Plate 3.5. This was in preparation for particle size distribution, Atterberg's limits, compaction and strength tests.

		
Air drying cinder material sampled from Thanantu BP in the laboratory in preparation for standard tests	Air drying red soil blending material in preparation for standard tests	Air drying weathered rock blending material in preparation for standard tests

Plate 3.5: Sampling preparation of cinder gravel and blending material for testing

3.5 Methodology for Particle size distribution determination

Three samples weighing 1500g each for neat cinder gravel and blending material were subjected to particle size distribution to check the limits of gradation. All standard sieve sizes; 50mm, 40mm, 28mm, 20mm, 14mm, 10mm, 6.3mm, 5mm, 2mm, 1mm, 600µm, 425µm, 300µm, 212µm, 150µm and 75µm were used and a stopper to capture finer particles passing 75mm sieve size. The mass retained in each sieve was weighed in a digital weighing balance and the percentage weight retained in each sieve determined and thereafter percentage weight passing each sieve calculated. The percentage passing in each sample was plotted in a logarithmic scale and compared with the minimum and upper limits for natural gravel requirements for subbase and base material in the Kenyan Road Design Manual Part III and the Kenyan Pavement Design Guidelines for LVSRs. The RDM Part III provides the grading limitation that qualifies the use of natural granular materials like cinder gravel in road construction. It meets the grading requirement when certain percentage passes the required sieve sizes. The LVSRs guideline provides various categories in which cinder gravel has to meet in order to be used in road construction. For this study, natural/blended granular materials (G30) and (G80) grading envelopes specification limitation was used as given in Fig. 3.3(a) and (b). The results are discussed in Chapter 4 of the report and the grading curves for neat cinder gravel and blending material are given in the same chapter.

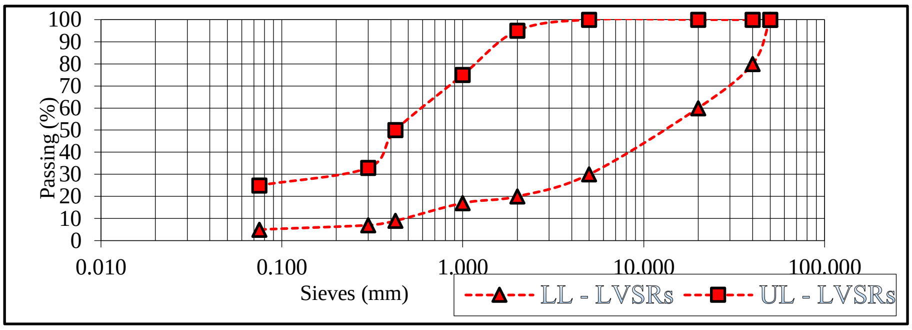

Figure 3.3(a) Specifications requirements for subbase materials for LVSRs

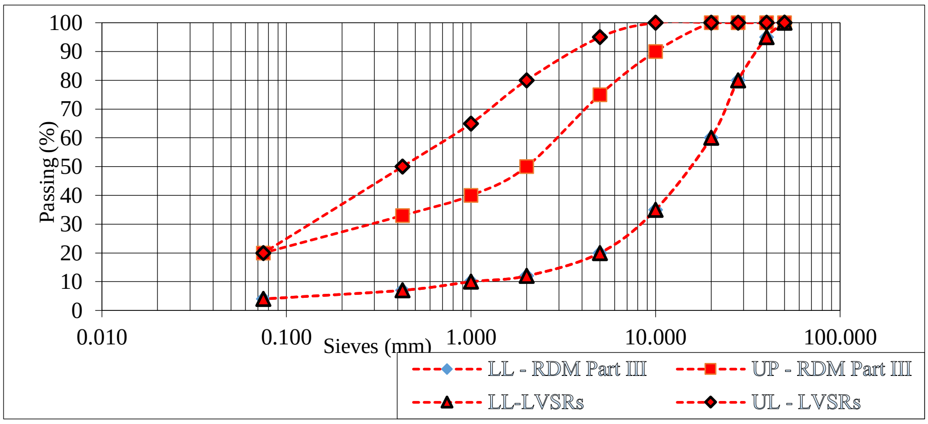

Figure 3.3 (b) Specifications requirements for base materials for LVSRs

Since natural material are compacted on site during road construction and that cinder gravels breaks into finer material when compacted it was necessary to determine grading of cinder after compaction using the wet sieve analysis BS 1377 Part 2 Standard procedure. Wet sieve analysis was carried out for compacted material after MDD and CBR determination and after soaking for 24 hours. To separate silt and clay-sized particles, 2gms of Sodium Hexameta Phosphate was added to 1 litre of water for each sample and mixed uniformly. Thereafter the material was oven dried for 24 hours at 105ºC before carrying out dry sieve analysis of material retained on 2.36mm, 600µm and 75µm sieves.

3.6 Methodology for Atterberg's Limits

Atterberg limits for neat cinder gravel, blended cinder with weathered rock and red soil and blending materials (weathered rock and red soil) was carried out in accordance with BS 1377 Part 2. The Liquid Limits (LL), Plastic Limits (PL) and Linear Shrinkage (LS) was determined for at least two samples for the finer particles passing 425mm sieve as illustrated in the Plate 3.6. From the results the Plasticity Index (PI), Plasticity Modulus (PM) and uniformity of coefficient (C_u) were computed and discussed in chapter 4.

		
Sample penetration for Liquid limit determination using Dynamic Cone Penetrometer method	Sample preparation for moisture content determination of Liquid Limits and Plastic Limits	Preparation of sample for linear shrinkage determination

Plate 3.6: Atterberg's Limits determination of cinder gravel blended with weathered rock

The cone was allowed to penetrate the soil sample for 5 seconds and the penetration (mm) at various water content (%) of at least 4 values was determined. The penetration verses water content graph was plotted and the liquid limit determined as the water content at which the cone penetration was 20mm.

3.7 Methodology for compaction tests (AASHTO T180-D)

Compaction tests were carried out to determine the relationship between moisture content and density (MDD / OMC) of neat cinder gravel, blending material (weathered rock and red soil) and blended cinder with weathered rock and red soil. The material for compaction tests were sieved and from the material passing the 20mm IS sieve, a representative sample of 5000gm was used for each sample A & B. A 152.4mm diameter by 116.43mm height cylindrical mould (with collar attached) was used as shown in Plate 3.7.

The specimen was formed by compacting the prepared soil in the mould in five approximately equal layers to give a total compacted depth of about 127mm, each layer was compacted by 56 uniformly distributed blows using a 4.54-kg rammer and a 457-mm drop in accordance with AASHTO T180 D standard procedure. An increment of 100ml of water was added to the specimen before each compaction cycle was carried out. For each sample a range of between 200ml to 700ml of water was used to give the 5 points of the dry density / moisture content results. After adding water to the specimen, it was covered with aluminium foil, and allowed to rest for at least an hour for the material to absorb the moisture uniformly.

		
Sample preparation for compaction tests of a representative sample passing 20mm IS Sieve	A 152.4mm diameter by 116.43mm height cylindrical mould for AASHTO T180-D standard compaction test	Specimens covered with aluminium foil to allow moisture to dissipate evenly through the specimen

Plate 3.7: Preparation of specimen for compaction tests

Before compaction, lubrication oil was applied to the surface of the mould to prevent the stickiness of the soil to the mould. Following compaction, the extension collar was removed and the compacted soil carefully trimmed even with the top of the mould by means of a straightedge. The moisture content and the dry mass of compacted soil was calculated for each of the compacted samples. The oven-dry densities of the soil were plotted as ordinates and corresponding moisture contents as abscissae to obtain the MDD and OMC of the material. The optimum moisture content and maximum dry density for each sample of neat and blended material was determined and the results are as discussed in chapter 4 of the report. Since the neat cinder was non-plastic a method of re-using a single moulded specimen was used to obtain the five points of the compaction curve as expounded in the subsequent section 3.8 of the methodology.

3.8 Methodology of re-using the specimen to determine the MDD and OMC of neat cinder gravel

According to the guideline for the use of cinder gravels in pavement layers for Low Volume Roads by Ethiopian Roads Authority (2018), research by TRRL/ERA 1975 study found out that by using the AASHTO T180-D test method, it is not always accurate or even possible to determine the maximum dry density (and OMC) of cinder gravels. This is because the dry density/ moisture content curve does not exhibit a definite curve. This behaviour was found with neat cinder gravel for this study. The method of re-using a single moulded specimen to obtain all five points of the compaction curve was adopted during the study. After each compaction test of the specimen, the unsoaked CBR at the top and bottom of the specimen was measured. The compacted material was removed from the mould and placed in a tray where a representative portion of the soil was taken for moisture content determination. Thereafter a suitable increment of water was added to the specimen and mixed thoroughly into the soil and re-compacted by 56 uniformly distributed blows in accordance with AASHTO T180-D standard procedure. The procedure was repeated on the same mould with increment of water content in each cycle until the density dropped depicting a concave density/moisture relationship curve. Analysis of results are discussed and presented in chapter 4.

3.9 Methodology for CBR Strength tests (AASHTO T193)

California Bearing Ratio (CBR) is a ratio expressed in percentage of force per unit area required to penetrate a soil mass with a circular plunger of 50 mm diameter at the rate of 1.25 mm/min to that required for corresponding penetration in a standard material. The ratio is usually determined at 2.5 and 5mm penetration, where the ratio at 5 mm is higher than that at 2.5 mm, the ratio at 5 mm is used for the design purpose (Sudhir et al., 2014). The specimen is usually prepared at MDD and OMC and usually tested soaked or unsoaked. CBR is usually used to determine the strengths of the material to be used in the construction of pavement and usually influenced the thickness of the pavement to be used. After each compaction test of neat and blended cinder, the specimen was subjected to strength test using Multispeed CBR Tester for penetration at the top and bottom as shown in Plate 3.8.

		
Mounting specimen for CBR Testing	Setting up Multispeed CBR Tester	Taking penetration reading from Multispeed CBR Tester

Plate 3.8: Testing for CBR of the mould using Multispeed Tester

The Multispeed CBR Tester was used because the material was un-stabilized. The standard load factor for 2.5mm and 5.0mm penetration plunger is 13.24 and 19.96 respectively. The larger CBR value obtained from 2.5mm and 5.0mm penetration was adopted as the unsoaked CBR value of the specimen. CBR of the specimen was calculated from the equation below;

$$\text{CBR (\%)} = \left[\frac{\text{Load sustained by the specimen at 2.5mm or 5mm penetration}}{\text{Load sustained by the standard aggregate at the corresponding penetration}}\right]*100$$

…………..Equation 3.1

The CBR at the bottom of the specimen was measured since it is the higher compacted surface of the sample. At least two specimens were penetrated at similar moisture content and the average unsoaked CBR obtained for the various blending ratios. The highest CBR obtained at a particular moisture content was determined as un-soaked CBR of the blending ratio and the results are discussed in Chapter 4.

3.10 Methodology for soaked CBR strength tests (AASHTO T193)

CBR test was carried out in accordance with AASHTO T193 standard test procedure. Neat cinder gravel and the optimum blended cinder (90:10 ratio) was sieved through 20mm IS sieve, for a representative sample of 5000gm for each sample A & B. A 152mm internal diameter by 178mm height mould provided with an extension collar approximately 50mm in height and a perforated base plate that could be fitted to either end of the mould was used. Five specimens at optimum MDD and OMC as determined in accordance with AASHTO T180-D compaction method detailed in the preceding section 3.7 was prepared for neat cinder gravel, one, three and five compaction cycles for the 90:10 optimum blending ratio.

The mould was clamped to the base plate, extension collar attached and weighed. A spacer disk was inserted into the mould and a filter paper inserted to prevent material from sticking into the spacer disk. The specimen was compacted in 62 uniformly distributed blows in approximately five equal layers to mould the CBR specimen to 100% MDD determined by AASHTO T180-D method. After compaction to a total depth of 127mm, the extension collar was removed and excess compacted specimen trimmed off by a straight edge. The weight of mould and compacted specimen was determined and recorded. A coarse filter paper was placed on the perforated base plate and the mould containing compacted specimen inverted and clamped to the base plate. Surcharge weights weighing 4kgs together with a perforated swell plate was placed on the surface of compacted specimen. The swell gauge was placed in contact with swell stem and the initial swell reading taken. The specimen was then soaked in a water tank for 4 days after which the final swell reading was taken and recorded. The swell as a percentage (%) of the initial sample length was calculated as below;

$$\text{Percent swell} = \frac{\{\text{change in length in mm during soaking}\} \times 100}{\text{Initial sample length in mm}} \quad \textbf{……………… Equation 3.2}$$

The sample was allowed to drain off water in a vertical position for about 15 minutes. The sample was weighed again to calculate the percentage of water absorbed and then tested for CBR penetration.

3.11 Specifications requirements for suitability of granular materials for road construction

The requirements in the Kenyan Road Design Manual Part III and the Kenyan Pavement Design Guidelines for LVSRs were referenced in determination of suitability of cinder gravel for road pavement construction. The optimum blended cinder gravel was checked for suitability as subbase and base materials upon meeting the requirements in Table 3.2;

Table 3.2: Natural granular material specifications for pavement for LVSRs

Material Type	**Type of test**	**Material requirements**	**Application**
G25 material (Natural Gravels)	CBR at 95% MDD (AASHTO T180) and 4 days soak (%)	Min 25	Sub-base material for: • T5-1 traffic (250,000-500,000 CESA) and • T5-2 traffic (100,000-250,000 CESA)
	Swell (%)	Max. 1.0	
	Uniformity Coefficient (C_u)	Min. 5	
	Plasticity Index (%) (in wet areas)	Wet areas: Max 15 Dry areas: Max 20	
	Plasticity modulus	Max 250	
G30 material (Natural Gravels)	Grading after Compaction	Compliance with desirable limits for G30 in LVSRs Guidelines	Sub-base material for T5-0 (500,000 – 1Million CESA) and base material for: • T5-4 (< 25,000 CESA) • T5-3 (25,000 – 100,000 CESA)
	CBR at 95% MDD (AASHTO T180) and 4 days soak (%)	Min 30	
	Plasticity Index (%)	Max 12	
	Plasticity Modulus	Max 250	
G80 material (Natural Gravels)	Grading after compaction	Compliance with desirable limits for G80 in LVSRs Guidelines	Base material for T5-0 (500,000 – 1Million CESA)
	CBR at 95% MDD (AASHTO T180) and 4 days soak (%)	Min 80	
	Plasticity Index (%)	Max 10	
	Plasticity modulus	Max 250	

Legend:

G25, G30 and G80 denotes natural or blended (mechanically stabilised) granular materials with the minimum CBR strength of 25, 30 and 80 respectively measured after 4 days soak.

3.12 Methodology of repeated compaction tests

Some cinder gravels show increase in strength as the particles breakdown during compaction to finer particles while others do decrease (Ethiopian Roads Authority, 2018). To determine this variation, repeated cycle compaction on the same specimen was done for moulded sample at MDD and OMC. The first specimen was compacted in accordance with AASHTO T180-D standard procedure and MDD, CBR, Moisture content, Atterberg's limits determined. For the second specimen, the compacted material was extracted from the mould, mixed thoroughly and re-compacted (without addition of water) to represent two mould-compaction cycles. A similar procedure was carried out for three, four and five mould compaction cycles. For every compaction cycle, the MDD and CBR test at OMC was determined, then followed by grading (wet sieve analysis) and Atterberg's limit determination. The CBR at OMC, CBR for repeated compaction cycles and that of standard compaction was compared to evaluate whether there was variation in strength as a result of repeated compaction and if beneficial or detrimental for use on site. The procedure for repeated sample compaction is shown in Table 3.3.

Table 3.3: Repeated cycles compaction

Specimen was moulded at MDD & OMC	**Level of compaction**	**Tests done**
First	One mould-compaction cycle	MDD/OMC, CBR, Grading and Atterberg's limits
Second sample	Two mould-compaction cycles	MDD/OMC, CBR, Grading and Atterberg's limits
Third sample	Three mould-compaction cycles	MDD/OMC, CBR, Grading and Atterberg's limits
Fourth sample	Four mould- compaction cycles	MDD/OMC, CBR, Grading and Atterberg's limits
Fifth sample	Five mould- compaction cycles	MDD/OMC, CBR, Grading and Atterberg's limits

For the number of mould-compact cycles that gave the highest CBR, particle size distribution and the plasticity modulus was checked if they were within the design requirements / specifications for suitability of the material for pavement layers given in the design manuals and guidelines. If the two parameters were outside the requirements, then the number of cycles that best met the requirements was selected. CBR at 4 days soaked at optimum number of repeated compaction was determined for design as shown in the design flow chart in Figure 3.1. Analysis of the results for repeated cycle compaction are discussed in Chapter 4 of the report.

3.13 Methodology for Shear strength tests

Shear box test machine was used to determine the maximum resistance that the soil could be mobilized against shear stress. The shear resistance of soil is a result of friction and interlocking of particles, and possibly cementation or bonding at particle contacts. The test determines the angle of shearing resistance and cohesion of the soil. The two parameters were varied depending on the state of soil. Due to interlocking, particulate material may expand or contract in volume as it is subject to shear strains. If soil expands its volume, the density of particles decreases and the strength decreases; in this case, the peak strength is followed by a reduction of shear stress (Onyelowe K.C., 2012).

BS 1377- 7:1990, limits that the maximum size of the aggregates to be used in the shear box shall not exceed one tenth of the specimen mould height. However, this procedure was not representative of the actual process in the field, due to large particles size of cinder gravel and BS limitation of the size of the specimen to be used. Consequently, repetitive sample compaction at MDD and OMC was employed to capture the sample representation. The methodology comprised compacting five specimens of the material at different compaction cycles of five factorial and thereafter extracting a specimen in a shear box mould for shear value determination. Remixing samples after each compaction cycle ensured uniformity of the material before remoulding. The Shear strength test procedure adopted for the optimum blended cinder gravel material is given in Figure 3.4.

To enable shear box test to be carried out a 6cm x 6cm x 4cm mould was fabricated as shown in Figure 3.3. Samples were molded at maximum MDD and OMC of optimum blending 90:10 ratio determined in accordance with AASHTO T180-D standard test procedure. A split mould of 152mm diameter and 178mm height was used for the ease of extrusion of the sample when the cutter had been driven in the sample. A spacer block was placed on top of compacted sample and the split mould mounted to the hydraulic jack and turned upside down, the spacer block restrained the sample in position so that when driving in the cutter for shear box mould, it restrained the sample from displacement. The specimen for shear box test was extracted from the bottom surface of compacted sample because it was the surface of higher compaction.

The cutter was placed in position on the compacted sample and a flat hollow metal surface placed on the cutter top surface to aid in uniform driving of the cutter. The hydraulic jack was lowered in position and pumped gently to drive the cutter into the sample as shown in Plate 3.9. When it was fully driven in, the hydraulic jack was disengaged, and the split mould opened to remove the sample and gently cut to remove the cutter from the sample. The sample in the cutter was trimmed using a sharp straight edge and the mould sides unscrewed to remove the specimen. After extrusion from the mould, the specimen 6cm square by 4cm height was covered with a cling film foil paper to prevent moisture lose from the sample and also allow excess water to dissipate into the sample for 24hours in readiness for shear box test. Three specimen per test for each cycle of compaction were molded.

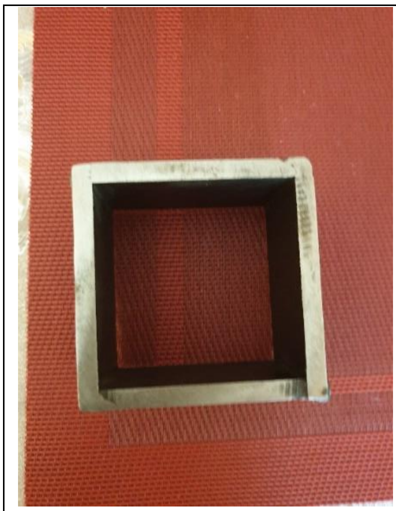		
Fabricated 6cm x 6cm x 4cm mould. The screws aided in extraction of specimen from the mould	Driving the mould into the compacted sample using hydraulic jack	Trimming and extracting specimen from compacted sample extruded from the split mould

Plate 3.9: Extraction of Shear Box Test specimen

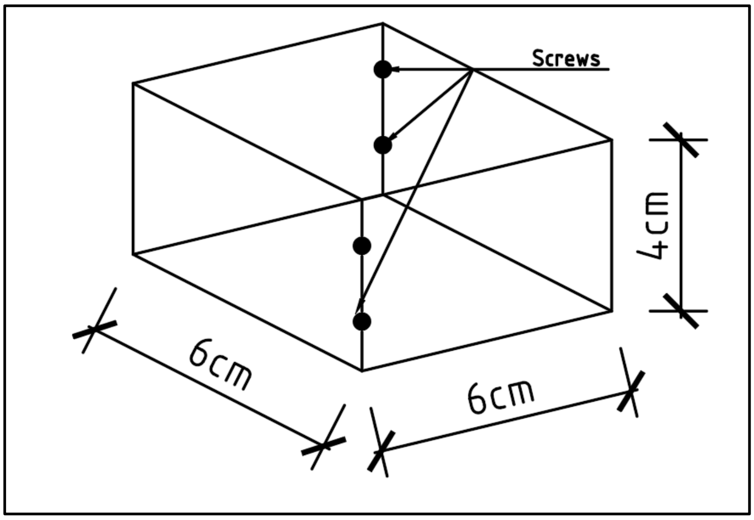

Figure 3.3: Detachable prefabricated shear box mould

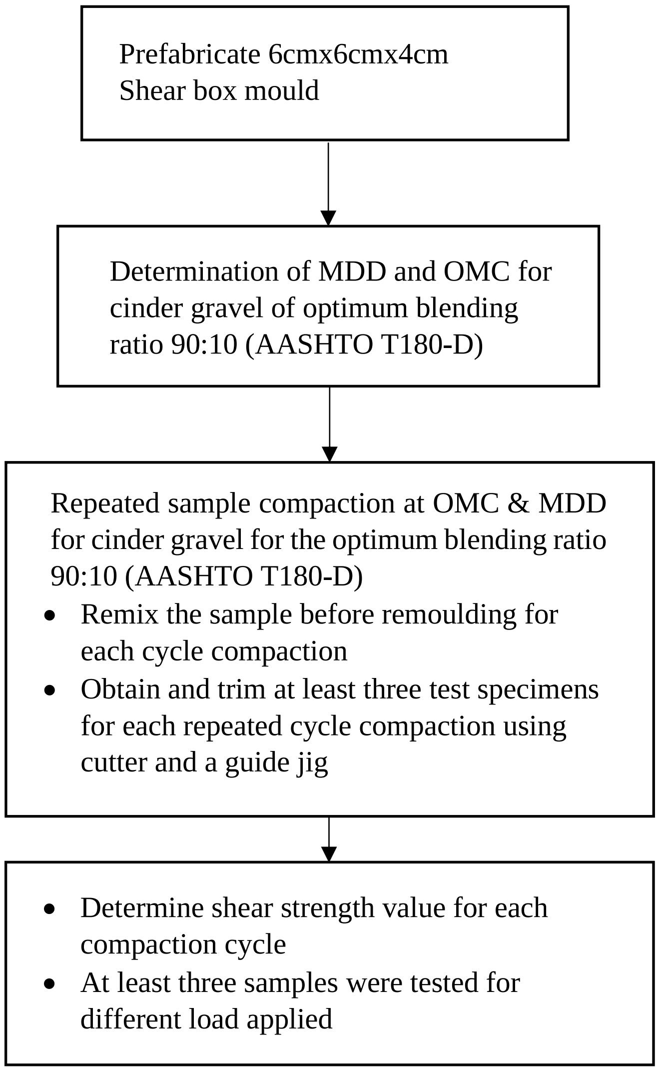

Figure 3.4: Shear strength test process

During the shear box testing three loads were applied; 32.2 Kg, 68.9 Kg and 105 Kg and the weight of hanger of 4.5 Kg was added to the load since it affects the normal stress. The specimen was soaked for a few minutes before testing as shown in Plate 3.10.

		
Extracted specimen fitted in the Shear Box Mould in readiness for shear testing	Soaking the specimen for a few minutes prior to testing	Note the displacement of upper half of box at shear failure

Plate 3. 10: Assembling specimen for shear box test

A graph of Normal Stress (σ) on the abscissae and shear stress (τ) on the ordinate was plotted to determine the unit of cohesion (c_u) and the angle of shearing resistance (φ) of cinder gravel at different cycles of compaction. Shear strength was calculated from Coulomb's law theory equation 3.3 (Smith and Smith, 1988).

$$\tau = c_u + \sigma \tan \varphi \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \textbf{Equation 3.3}$$

where;

τ = shear stress at failure (shear strength)

c_u = unit of cohesion

σ = total normal stress on failure plane

φ = angle of shearing resistance

The strength parameters of cinder gravel were compared for different cycle compactions.

Chapter Four: Results and Discussions

4.1 Neat cinder gravel and blending material

4.1.1 Overview

Test results were analyzed in tabular form, graphs were plotted, trend lines /curves and equations were developed and inferences to the test results made to draw conclusions from the study. At least two samples (A&B) were carried out for each parameter to have representative test results and comparison made to weed out any outliers and where necessary repeat tests were carried out. The results were averaged in spreadsheets in preparation for further analysis and plotting of graphs. The results were analyzed in three broad categories and deductions made in line with the objectives of the study.

The results plotted in graphs were tested for statistical significance of the observed values to the fitted regression line by coefficient of correlation R-squared. The higher the R-squared, the better the model fitted the analyzed data. Analyzed data on grading, MDD/OMC, strength and plasticity are presented and discussed in this chapter while unprocessed data for shear stress verses normal stress of blended cinder gravel are presented as Appendix A.

4.1.2 Grading of neat cinder and blending material

The tests result for grading of neat cinder and blending materials are presented in grading graphs in Figures 4.1(a) and (b) and summarised in Table 4.1. From the table, neat cinder gravel samples from Thanantu borrow pit (B/P) did not meet the requirements for use as pavement construction material in grading due to deficiency in fine particles < 0.075mm (µm), thus required blending with weathered rock or red soil to improve grading for suitability as road pavement material.

Table 4.1: Sieve Analysis of sample material

Material	Tests carried out	Preliminary Analysis of results	Remarks
Neat cinder from Thanantu borrow pit	Grading on all standard sieve sizes (three	• The three samples of cinder gravel did not meet specification requirements for Natural Gravels for subbase material in	• Cinder gravel from Thanantu B/P required blending with

Material	Tests carried out	Preliminary Analysis of results	Remarks
	samples per test)	the LVSRs guideline for sieve size 1mm and below • The three samples of cinder did not meet specification requirements for Natural Gravels for Base material in the LVSRs guideline for sieve size 1.5mm and below. • The three samples of cinder did not meet the specification requirements for Natural Gravels for Base material in the RDM Part III for sieve size 1mm and below.	finer material to improve on grading properties of the material
Weathered rock blending material from Ntoombo borrow pit	Grading on all standard sieve sizes (three samples per test)	• The weathered rock conformed to grading specification for Natural gravels for base material in the LVSRs guideline and RDM Part III • However, the weathered rock marginally met the specification requirements for Natural Gravels for subbase material in the LVSRs guidelines for sieve size 0.075mm.	• Weathered rock from Ntoombo B/P met the minimum requirement as blending material
Red soil blending material adjacent to Thanantu B/P	Grading on all standard sieve sizes (three samples per test)	• The three samples of red soil did not meet specification requirements for Natural Gravels for subbase material in the LVSRs guidelines for sieve size 1mm and below • The three samples of red soil did not meet specification requirements for Natural Gravels for base material in RDM Part III and LVSRs guidelines for sieve size 1.0mm and below.	• Red soil did not meet the minimum requirement for natural gravels for subbase and base materials for LVSRs in grading

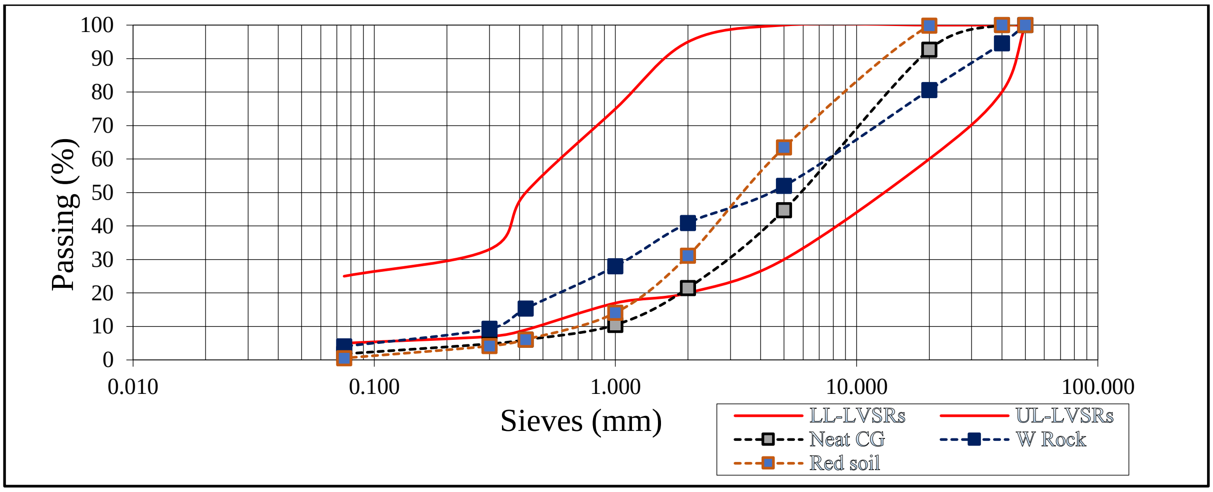

Figure 4.1(a): Grading curve of neat cinder and blending materials against grading requirements for subbase materials (G30) for LVSRs

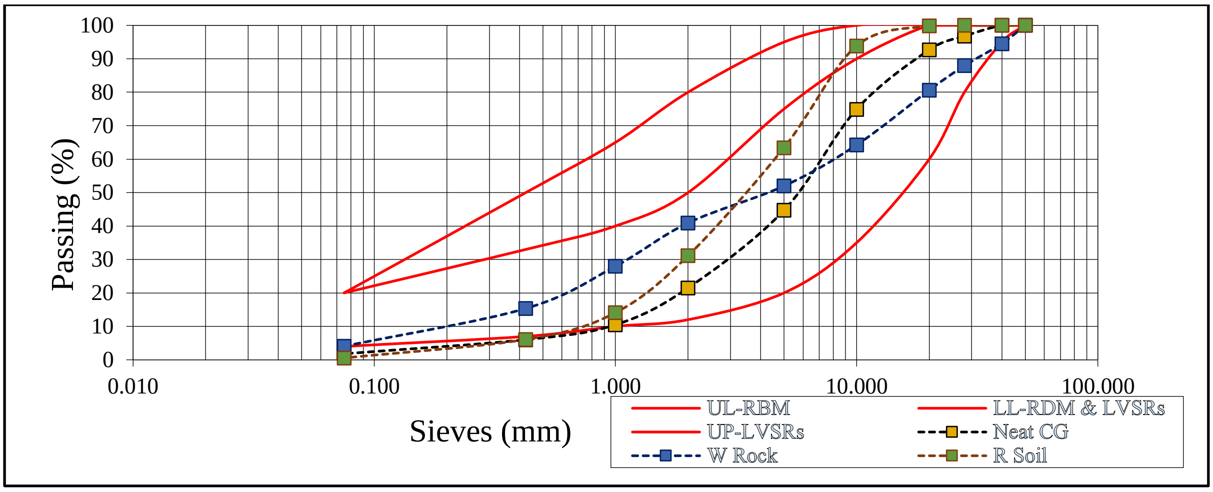

Figure 4.1(b): Grading curve of neat cinder and blending materials against grading requirements for base materials in RDM-III and (G80) for LVSRs

From Figures 4.1(a) and (b), weathered rock blending material met the grading requirements for subbase materials (G30) and base materials in RDM Part III and (G80) for LVSRs. Neat cinder gravels and red soil did not meet the specification requirements for the lower sieve sizes (< 1mm). The material required blending with weathered rock to improve on the properties.

4.1.3 Atterberg's Limits of neat cinder gravel and blending material

Atterberg's Limits tests on sampled cinder gravel and blending materials were carried out and the analysis of test results have been given in Table 4.2. The results showed that neat cinder gravel samples from Thanantu borrow pit (B/P) were non-plastic and required blending with either red soil or weathered rock to improve plasticity of the material for use as road pavement construction material. The red soil exhibited high plasticity while the weathered rock was of medium plasticity. Therefore, blending with weathered rock was considered a better option if the blended cinder is to meet the specification requirements for pavement material in plasticity characteristics.

Table 4.2: Atterberg limits of neat cinder gravel and blending material

Material	**Sample No.**	**Atterberg Limits**							**Remarks**
		LL (%)	**PL (%)**	**PI (%)**	**LS (%)**	**Average PI (%)**	**Average LS (%)**	**Average PM***	
Neat cinder from Thanantu B/P	A	N/A	NP	N/A	2.9	N/A	2.9	NP	Neat cinder was non-plastic and blending was required
	B	N/A	NP	N/A	2.9				
Red soil blending material adjacent to Thanantu B/P	A	76.9	40.0	36.9	17.9	36.1	18.25	220	Red soil was plastic and was used to improve properties of neat cinder gravel
	B	76	40.7	35.3	18.6				
Weathered rock blending material from Ntoombo B/P	A	53.6	30.8	22.8	10.7	21.8	10.7	336	Weathered rock was plastic and was used to improve properties of neat cinder gravel
	B	54.2	33.4	20.8	10.7				

* - Plasticity Modulus (PM) = PI x (% < 425μm) where % < 425μm = percentage of particle sizes less than 0.425mm sieve.

4.1.4 Moisture content / dry density of neat cinder gravel

Compaction tests for neat cinder gravel were carried out in accordance with AASHTO T180-D standard test procedures. The following observations were made of neat cinder gravel during compaction;

(i) The material did not stick on the rammer during compaction showing that the material was non-plastic. This characteristic was also observed during Atterberg's Limits determination where the material crumbled during rolling an indicator of non-plastic nature of the material,

(ii) At high moisture content the mass of the sample increased continually, and bleeding of the material was observed during compaction which is not the norm for the standard density/MC concave curve for natural gravels. This is because the volume of air voids was completely filled with water and at the same time the vesicles on the surface of cinder gravel absorbed more water thus the resulting increase in mass of the sample.

It was noted that the dry density/ moisture content curve for neat cinder gravel did not exhibit a definite curve as shown in Figure 4.2, and therefore the method of re-using a single moulded specimen for all the compaction cycles was adopted during the study.

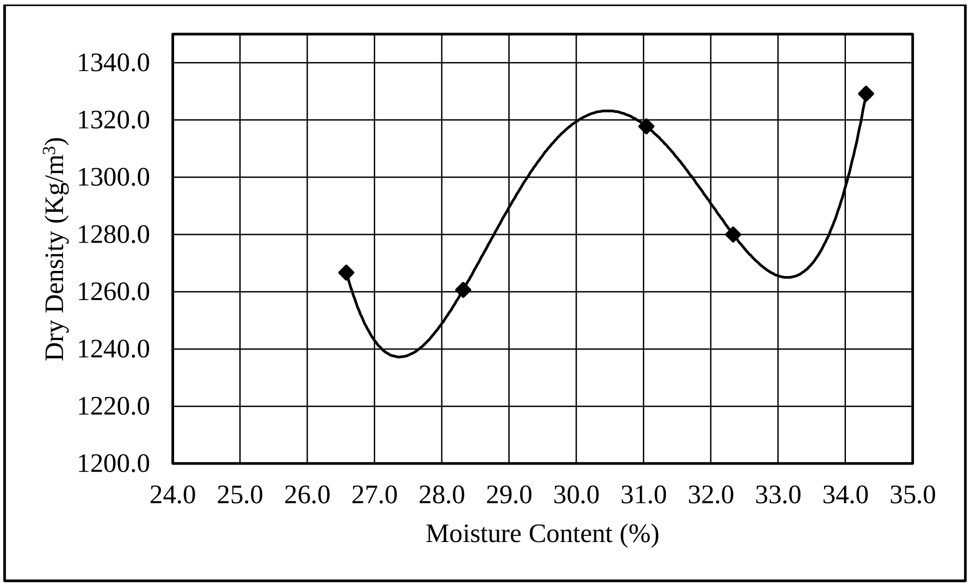

Figure 4.2: Dry density/ Moisture content relationship for neat cinder Gravel.

4.1.5 Moisture content / dry density relationship of cinder gravel by re-using the specimen

The procedure is a variation of AASHTO T180-D standard in that the same specimen is re-used in all the five compaction cycles necessary to determine the material dry density. By re-using the specimens, a more definitive curve is obtained as shown in Figure 4.3. All other aspects of AASHTO T180-D remained unchanged.

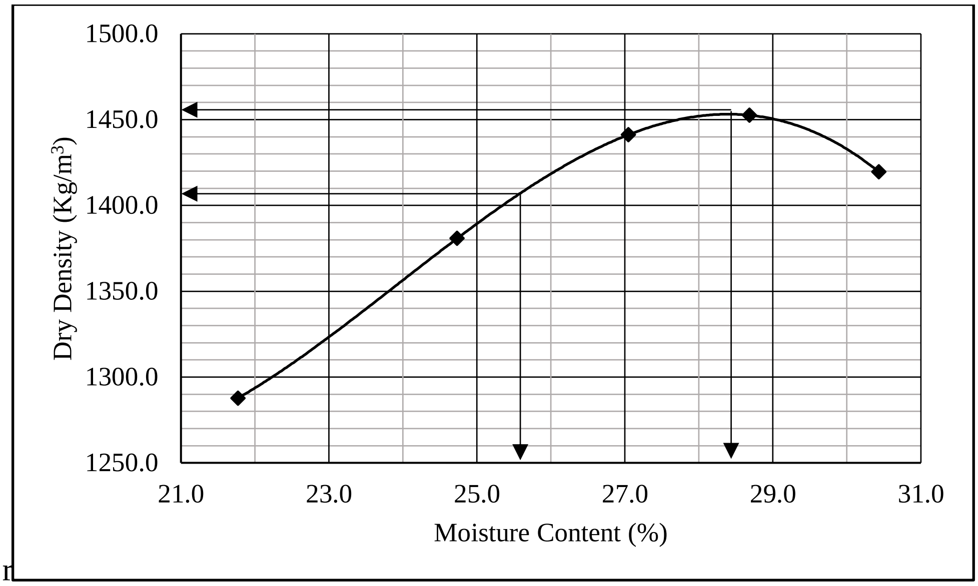

Figure 4.3: Dry density/ moisture content relationship of neat cinder gravel by re-using specimen.

It was noted that by re-using the specimen, the compaction curve of cinder gravel was well defined. The maximum dry density of the material increased significantly brought about by the breakage of material to finer particles (densification). The MDD and OMC of neat cinder gravel using the method of re-using specimen was 1455 kg/m^3 and 28.3% respectively. The high MDD of cinder gravel met requirement for the suitability of the material for pavement design. For each repeated compacted specimen unsoaked CBR of the specimen was measured and the CBR/ Moisture density relationship curve plotted as shown in Figure 4.4.

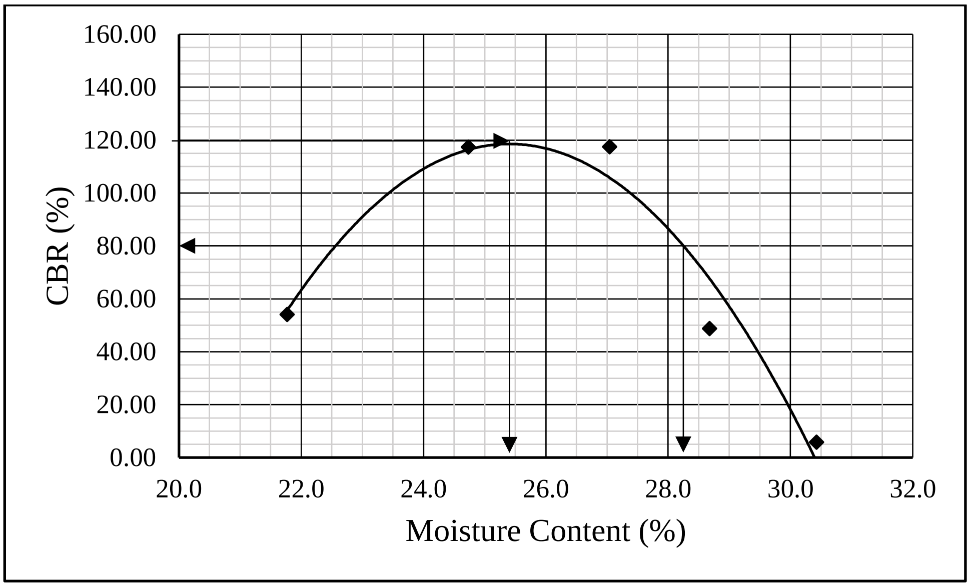

Figure 4.4: CBR/ Moisture content relationship of neat cinder gravel by re-using specimen.

Figure 4.4 shows that the strength of re-used cinder gravel specimen increased with further compaction to a maximum value before it decreased sharply. This indicates that the properties of the material improved with compaction as the material break up and becomes finer. The illustration further shows that the highest CBR of neat cinder Gravel (120%) occurs at 25.5% moisture content which was not at the maximum dry density determined in Figure 4.3. The strength as measured by CBR of the material at optimum moisture content was 80% as shown in Figure 4.4. This observation calls for further investigation of the strength and densification properties of cinder gravels before specifying requirements of the material for pavement construction.

4.1.6 CBR / dry density relationship of neat cinder gravel by re-using the specimen

From CBR/dry density curve in Figure 4.5, the maximum dry density of neat cinder was 1455 kg/m^3 with a CBR of 60% which was not at the highest strength of the material of 120% obtained from CBR/moisture curve in Figure 4.4. The observation indicates that cinder gravel requires further investigation on the effects of re-using specimens on the strength as measured by CBR and the maximum dry density of the material. The explanation for this finding is given in section 4.2.4 of this chapter.

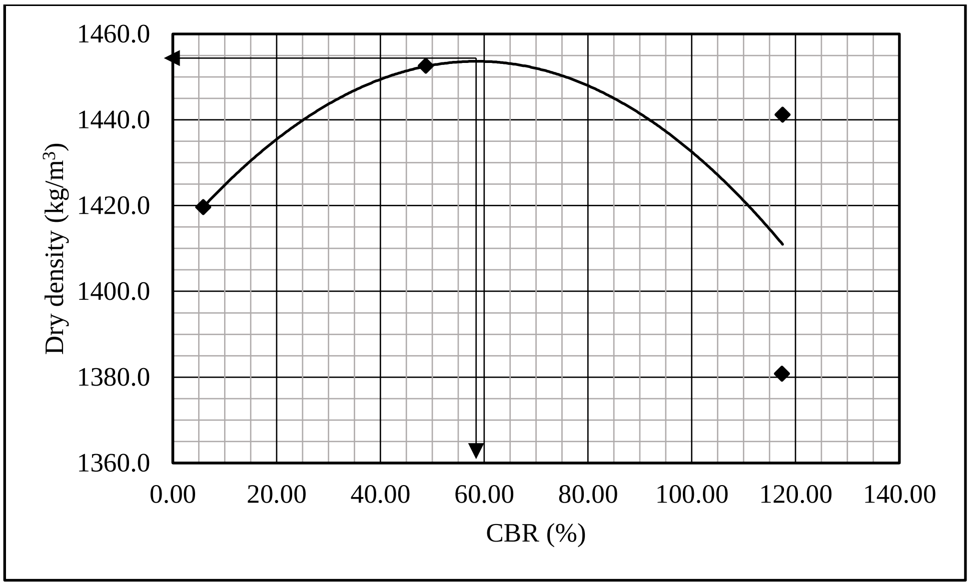

Figure 4.5: CBR / dry density relationship of neat cinder gravel by re-using the specimen

4.1.7 Dry density / moisture content relationship of weathered rock as blending material

A determination was made of the maximum dry density and OMC of the weathered rock sourced from Ntoombo B/P as blending material. The fine material was plastic and results in Table 4.1 and Figures 4.1(a) and (b) showed that the blending material fit the grading requirement for natural gravels for subbase and base material for LVSRs in the pavement design guidelines and RDM Part III. The MDD and OMC of weathered rock was determined using AASHTO T180-D standard testing procedure and results presented as shown in Table 4:3;

Table 4.3: MDD/OMC determination of weathered rock as blending material

Sample	MDD (Kg/m^3)	OMC (%)	Remarks
Sample A	1500	24.2	Suitable as blending material due to high dry density
Sample B	1510	23.0	
Average	1505	23.6	

The material had high dry density and was suitable as a blending material to improve the properties of cinder gravel.

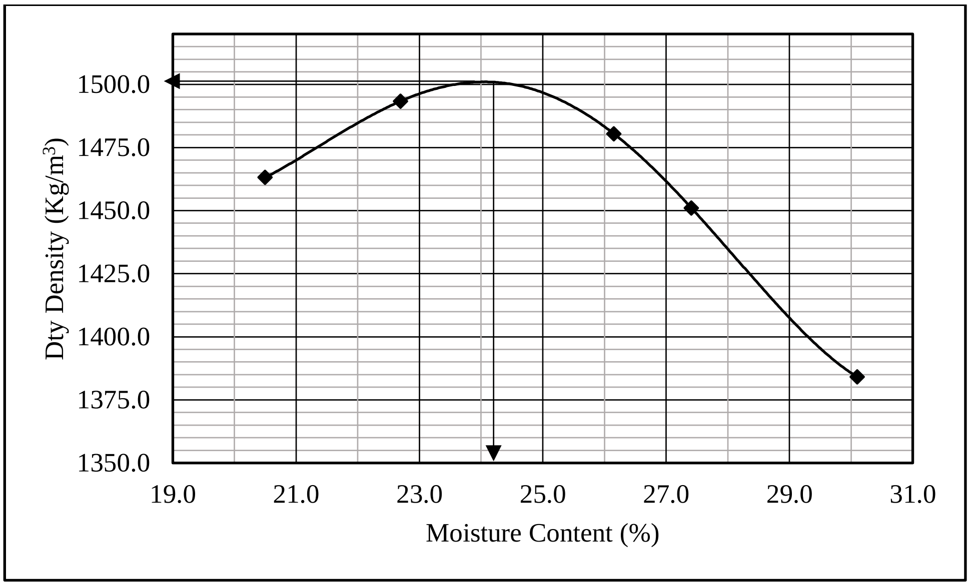

Figure 4.6: Dry density / Moisture content relationship of Weathered Rock sample A

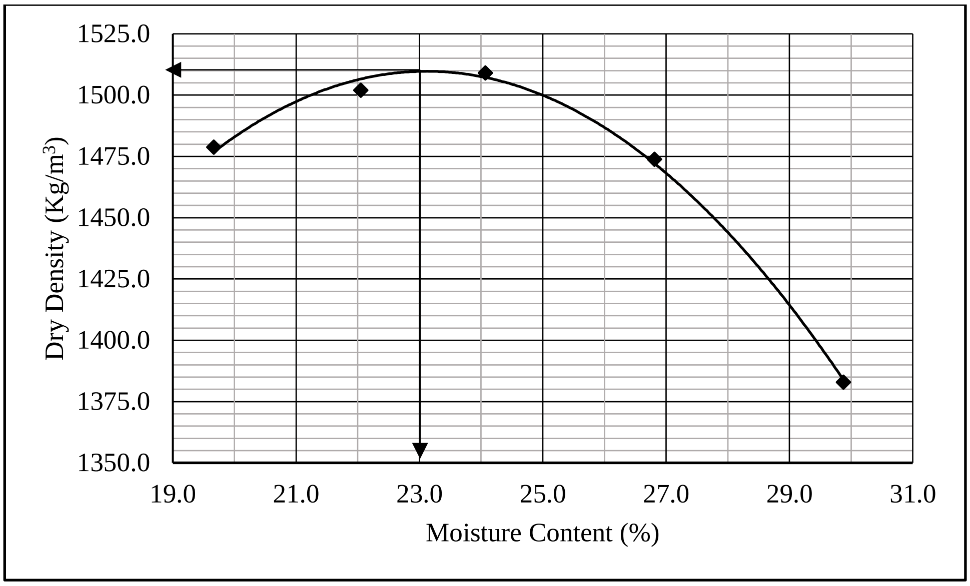

Figure 4.7: Dry density / Moisture content relationship of Weathered Rock sample B

As deduced from Figures 4.6 and 4.7 the average MDD and OMC of weathered rock was 1505 kg/m^3 and 23.6% respectively. The high dry density obtained made the weathered rock suitable as blending material since addition of a small quantity of fines to coarse-grained cinder gravel, improved the density and the engineering properties of the material for a given compaction effort.

4.1.8 Dry density / moisture content relationship of red soil as blending material

A determination was made of the maximum dry density and OMC of red soil sourced from a borrow pit adjacent to Thanantu B/P as blending material. The fine material was plastic and from Table 4.1 did not meet specification requirements for natural gravel for subbase and base materials for LVSRs in grading. The MDD and OMC of the red soil was determined using AASHTO T180-D standard test procedure and results presented as shown in Table 4:4;

Table 4.4: MDD/OMC determination of red soil as blending material

Sample	**MDD (Kg/m^3)**	**OMC (%)**	**Remarks**
Sample A	1368	34.5	According to the RDM Part III, a material with maximum dry density of less than 1400 Kg/m^3 is not recommended for pavement design purpose
Sample B	1395	35.2	
Average	1382	34.9	

The MDD of the red soil was determined from the dry density/moisture content curve as shown in Figure 4.8 and 4.9. The red soil depicted low dry density and was not suitable as a blending material to improve the properties of cinder gravel.

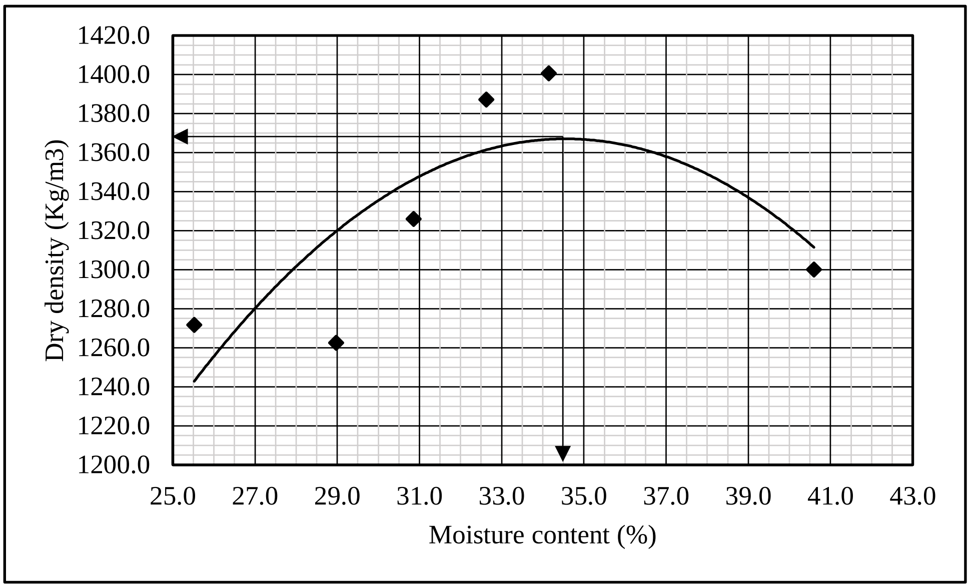

Figure 4.8: Dry density / Moisture content relationship of red soil sample A

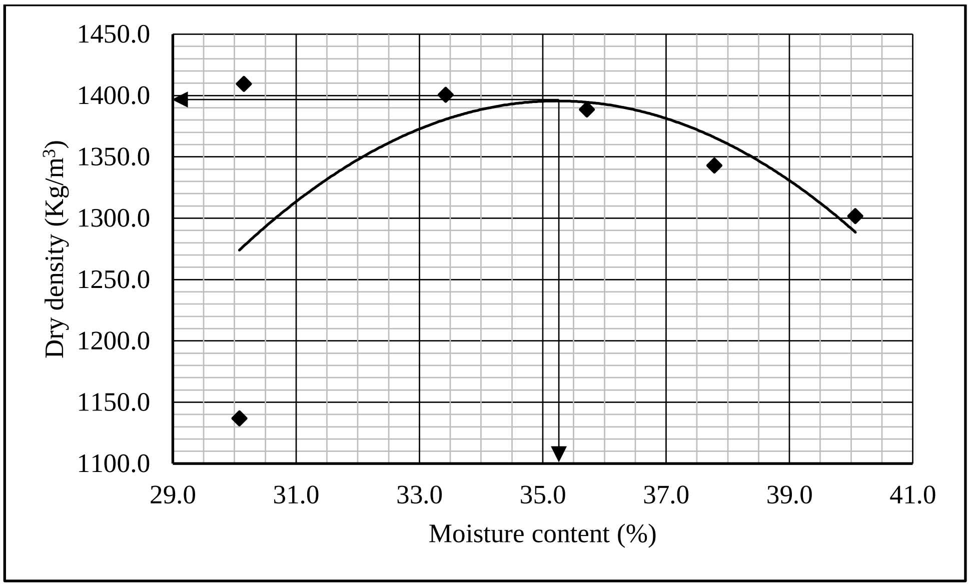

Figure 4.9: Dry density / Moisture content relationship of red soil sample B

As deduced from Figures 4.8 and 4.9 the average MDD and OMC of red soil was 1382 kg/m^3 and 34.9% respectively. The fine material was further explored for suitability as blending material for cinder gravel in Chapter 4.3.

4.2 Blending cinder Gravel with weathered rock

4.2.1 Overview

Cinder gravel was blended with weathered rock at various proportions and laboratory tests carried out to check on properties of the improved material. The proportions ranged from 90:10 to 70:30 by weight at an increment of 5% and test results are presented as below.

4.2.2 Dry density / moisture content relationship of cinder gravel blended with weathered rock

The Atterberg limits tests of neat cinder gravel determined in Table 4.2 indicated that the material was non-plastic and therefore cinder gravel required blending with a plastic blending material to improve on its engineering properties. Five specimens of five blending ratios 90:10, 85:15, 80:20, 75:25 and 70:30 of cinder gravel material blended with weathered rock were prepared to determine the MDD (AASHTO T180-D) and OMC and unsoaked CBR. The dry density/ moisture content results are summarised in Table 4.4. It is deduced from Table 4.4 that MDD increases with blending ration while OMC decreases and this matches with the explanation given in section 4.2.4 that the highest CBR of blended cinder material occurs at lower moisture content. This is because increase in finer particles in the mixture provides a large surface area for more moisture absorption in bonding the particles together.

Table 4.4: Maximum dry density/OMC relationship of blending ratios with weathered rock

Blending ratio (BR)	**Samples**						**Remarks**
	A		**B**		**Average**		
	MDD	**OMC**	**MDD**	**OMC**	**MDD**	**OMC**	
70:30	1512	22.8	1512	23.0	1512	22.9	The dry density of cinder gravel blended with weathered rock improved significantly with blending ratio
75:25	1465	25.3	1460	25.7	1463	25.5	
80:20	1487	24.8	1487	25.3	1487	25.1	
85:15	1455	25.7	1447	26	1451	25.9	
90:10	1432	25.7	1385	28	1409	26.9	

The maximum dry density of blended cinder gravel increased with increment in the blending ratio of material indicating improved compaction and binding properties of the material. This explains that the highest value of MDD occurs at a point when the finer particles (that fill all the tiny voids) in the blend is highest at 70:30 blending ratio. This means that there is more material per unit volume, hence a higher dry density. Thus, it indicates that the mechanical properties of the material had been improved by blending as shown in Figure 4.10.

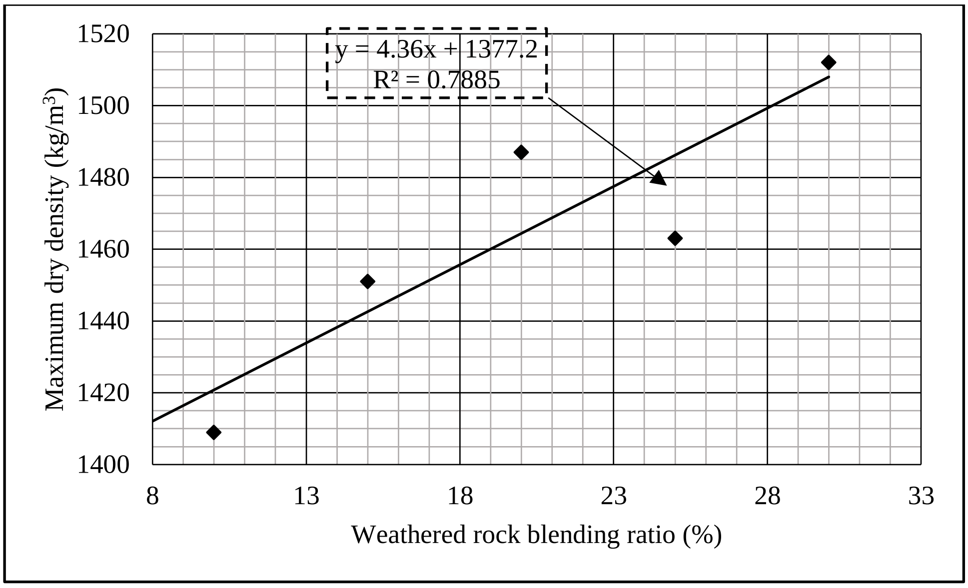

Figure 4.10: Correlation between blending ratio and maximum dry density of blended cinder gravel

The correlation between the blending ratio (%) of weathered rock with cinder gravel and the maximum dry density of the blended material is expressed by the linear trend line equation:

MDD = 4.36BR + 1377.2…………………………………………**Equation 4.1**

R^2 = 0.7885

where;

MDD = maximum dry density of blended cinder gravel in Kg/m^3

BR = Blending ratio (% weight of weathered rock blended with cinder gravel)

R^2 = Coefficient of correlation (variation of observed values to the regression line)

In the equation, the constant (1377.2) is the y-intercept, which is the MDD of a neat cinder material. The equation can be used to predict the MDD of cinder gravel at various blending ratio without carrying out individual MDD as an initial check and save on time.

4.2.3 CBR / moisture content relationship of cinder gravel blended with weathered rock

After compaction of each mould, unsoaked CBR for the blended cinder was determined using Multispeed CBR Tester equipment for neat (unstabilized) material. The readings at 2.5mm and 5.0mm penetration were recorded and the CBR values were obtained by dividing the penetration with the Standard Load Factor of 13.24 and 19.96 respectively. The highest CBR at various blending ratios is presented in table 4.5.

Table 4.5: CBR/MC relationship of blending ratios with weathered rock

Blending ratio	**Samples (Unsoaked)**						**Remarks**
	A		**B**		**Average**		
	Max CBR (%)	**MC (%)**	**Max CBR (%)**	**MC (%)**	**Max CBR (%)**	**MC (%)**	
70:30	134	20.9	162	21.4	148	21.2	The average highest CBR is 148% at 21.2% MC which did not occur at MDD of 1512 Kg/m^3 at OMC of 22.9% in Table 4.4
75:25	151	25.6	168	22.8	160	24.2	The average highest CBR is 160% at 24.2% MC which did not occur at MDD of 1463 Kg/m^3 at OMC of 25.5% in Table 4.4
80:20	105	23.1	193	22.2	149	22.7	The average highest CBR is 149% at 22.7% MC which did not occur at MDD of 1487 Kg/m^3 at OMC of 25.1% in Table 4.4.
85:15	148	23.7	155	22.7	152	23.2	The average highest CBR is 152% at 23.2% MC which did not occur at MDD of 1451 Kg/m^3 at OMC of 25.9% in Table 4.4.
90:10	108	23.9	139	24.4	123.5	24.2	The average highest CBR is 123.5% at 24.2% MC which did not occur at MDD of 1409 Kg/m^3 at OMC of 26.9% OMC in Table 4.4.

It is deduced from the results that;

(i) The average highest CBR for 70:30, 75:25, 80:20, 85:15 and 90:10 was 148%, 160%, 149%, 152% and 123.5% respectively. This shows the strength of the material increased with blending ratio up to a maximum value before it started to drop with further increase of blending material as is illustrated in Figure 4.11.

(ii) The average highest CBR of blended ratio 70:30, 75:25, 80:20, 85:15 and 90:10 occurred at moisture content of 21.2%, 24.2%, 22.7%, 23.2% and 24.2% which was lower than the OMC at 22.9%, 25.5%, 25.1%, 25.9% and 26.9% respectively. This indicates that the highest CBR value does not occur at the MDD/OMC of the material due to development of pore pressures at higher moisture contents which negates the pressure / loads impacted on the material with further compaction.

(iii) The strength (CBR) of blended cinder increases slightly with increment in the blending ratio of weathered rock indicating improved interlocking of particles. However, with further increase in the blending ratio, the high quantity of fines reduces the contact area between large to large particle of cinder gravel. The strength of the blend is significantly derived from the contact between large particles to transfer the load. Therefore, if this contact is reduced by the presence of a large enough quantity of fines, then the strength of the blend is consequently reduced as shown in Figure 4.8.

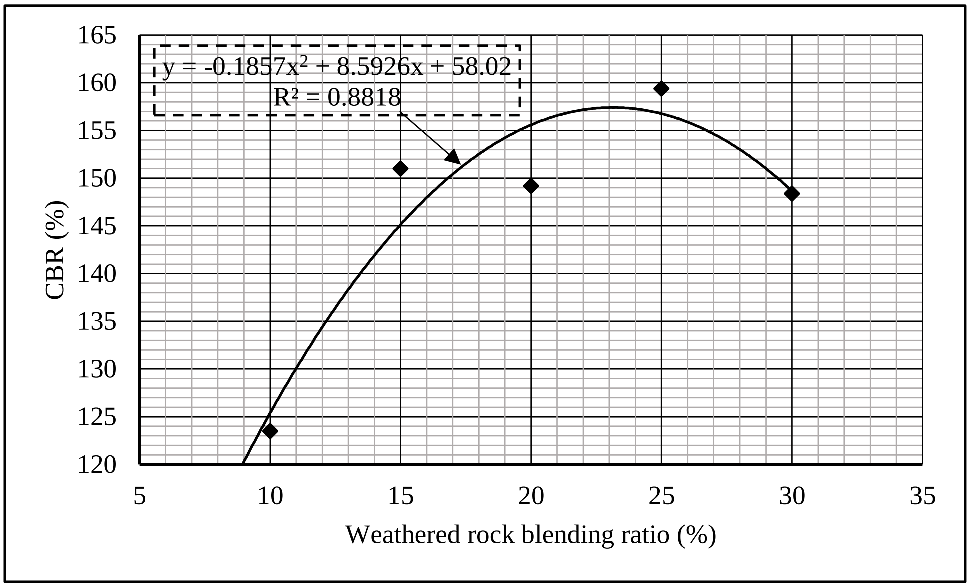

Figure 4.11: Correlation between blending ratio and CBR of blended cinder gravel

The relationship between the blending ratio (%) of weathered rock with cinder gravel and the strength of the blended material represented by CBR is expressed by the trend line curve equation:

$CBR = -0.1857BR^2 + 8.5926BR + 58.02$……………………………… **Equation 4.2**

$R^2 = 0.8818$

where;

CBR = Unsoaked strength of the blended cinder gravel (%)

BR = Blending ratio (% weight of weathered rock blended with cinder gravel)

R^2 = Coefficient of correlation (variation of observed values to the regression line curve)

4.2.4 CBR / dry density relationship of blended cinder gravel of various ratios

Plotted graphs of CBR/dry density of cinder gravel at various blending ratios showed that the maximum dry density of the blended material did not coincide with the maximum occurrence of strength (CBR) of the material as shown in Table 4.6. This was observed for both blended cinder gravel with weathered rock and red soil.

Table 4.6: CBR/dry density relationship of blended cinder gravel with weathered rock

Blending ratio	**MDD/CBR relationship**	**Max unsoaked CBR of the blending ratio (from Table 4.5)**
70:30	1528 Kg/m^3 at CBR of 86%	148%
75:25	1490 Kg/m3 at CBR of 85%	160%
80:20	1511 Kg/m3 at CBR of 95%	149%
85:15	1462 Kg/m^3 at CBR of 84%	152%
90:10	1400 Kg/m3 at CBR of 105%	123.5%

This is explained in that as the soil is compacted, an increase in the amount of water is directly proportional to the increase of the MDD and CBR. This is because the water forms a thin film around the soil particles thus helping the soil particles of cinder gravel and the weathered rock to bind together to form a strong bond. Thus, result in densification resulting into load resistance when applied. A point is reached where further addition of water result to increase in the MDD but a drop in CBR. When zero air voids occur, pore pressures develop and negates the pressure / loads impacted on the material thus significantly reduces the strength. The development of pore pressures during compaction of the material explains why maximum CBR does not coincide with MDD/OMC of cinder gravel. Thus, the highest CBR does occur before pore pressures starts acting as was observed in the study. Further addition of water beyond the optimum moisture content, the cinder gravel becomes saturated thus the water occupies all the air void spaces, and the MDD and CBR reduces. This phenomenon was observed during further compaction of the material at higher moisture content as shown in Plate 4.1.

		
Mixed specimen covered with aluminum plastic foil for about an hour before compaction for homogeneity of moisture	Saturated blended cinder gravel heaving with further compaction	Weighing compacted sample and mould for bulk density determination

Plate 4.1: Blended cinder gravel heaving during compaction at high moistures

4.2.5 Plasticity properties of cinder gravel blended with weathered rock

As observed in Table 4.2, neat cinder from Thanantu B/P was non-plastic and was therefore blended with weathered rock to improve on its plasticity. After the MDD/OMC determination and CBR test of various blending ratios, the fines (% < 425 μm) from these specimens were used to determine the Atterberg's limits (plasticity modulus) of blended cinder gravel. Since cinder gravels breakdown during compaction, thus generating fine particles, the plasticity index of the specimens after compaction and unsoaked CBR testing was measured. Plasticity characteristics of the five cinder blending ratios are given in table 4.7;

Table 4.7: Atterberg's Limits of cinder gravel blended with weathered rock at various blending ratios

Sample	**LL (%)**	**PL (%)**	**PI (%)**	**LS (%)**	**% Passing 425μm Sieve**	**PM**
Blending ratio 70:30						
A	46.8	32.34	14.46	8.6	37.0	535
B	48.8	24.81	23.99	7.1	42.3	1015
Average	47.8	28.6	19.2	7.9	39.65	775
Blending ratio 75:25						
A	46.6	29.2	17.4	7.9	9.2	160
B	46.7	32.64	14.1	7.9	10.2	144
Average	46.7	30.9	15.8	7.9	9.7	153
Blending ratio 80:20						
A	44.7	30.64	14.06	7.9	37.4	525.8
B	45.5	29.7	15.8	7.1	36.3	573.5
Average	45.1	30.2	**14.9**	7.5	36.9	550
Blending ratio 85:15						
A	48.6	34.2	14.4	7.1	10.2	147
B	47.3	30.7	16.6	7.1	7.9	131
Average	48	32.5	15.5	7.1	9.1	141
Blending ratio 90:10						
A	49.8	33.5	16.3	3.6	32.7	533
B	49.8	39.9	9.9	3.6	31.1	308
Average	49.8	36.7	**13.1**	3.6	31.9	418

From the Atterberg limits tests above it is deduced that;

(i) Blending ratio 80:20 and 90:10 met the maximum Plasticity Index (PI) of 15 for Natural Gravels for Subbase in RDM Part III, Granular Materials (G25) for subbase for Design Traffic Classes T5-1 and T5-2 in LVSRs and Natural Gravels for Base in RDM Part III. G25 denotes natural or blended (mechanically stabilised) granular materials of minimum CBR strength of 25% measured after 4 days soak.

(ii) All the blending ratios met the maximum requirements of plasticity for pavement materials in dry areas as provided for in RDM Part III and Pavement Design Guideline for LVSRs.

Both the Kenyan Road Design Manual Part III (1987) and the Kenya Pavement Design Guidelines for LVSRs (2017) gives a provision that plasticity indices of up to 25% and 20% respectively may be tolerated in dry areas and cinder gravels being predominantly found in dry areas (less than 500mm annual rainfall) the tolerances in plasticity indices may be accommodated.

4.3 Blending cinder gravel with red soil

4.3.1 Overview

Particle size distribution and Atterberg's Limits given in Tables 4.1 and 4.2 showed that red soil had high plasticity and depicted poor grading/gap grading as shown in Figures 4.1(a) and (b). Despite the finding, cinder gravel was blended with red soil sourced from Thanantu borrow pit at various proportions and laboratory tests were carried out to check if there was improvement in properties of the blended material. The proportions ranged from 90:10 to 70:30 with increments of 5% of red soil being added to cinder gravel and the test results are presented as below.

4.3.2 Dry density/moisture content relationship of cinder gravel blended with red soil

The Atterberg limits tests of neat cinder as determined in Section 4.2 showed that the material was non-plastic and therefore cinder material required blending with a plastic material to improve on its engineering properties especially plasticity. Five specimens of the five blending ratios 70:30 to 90:10 of cinder material and red soil were prepared to determine the MDD and OMC (AASHTO T180-D) and CBR. The dry density/ moisture content results are summarised in Table 4.8.

Table 4.8: Maximum dry density/OMC relationship of blending ratios with red soil

Blending ratio	**Samples**						**Remarks**
	A		**B**		**Average**		
	MDD	**OMC**	**MDD**	**OMC**	**MDD**	**OMC**	
70:30	1480	27.7	1495	26.8	1488	27.3	The dry density of cinder gravel blended with red soil improved with increment in the blending ratio
75:25	1445	28.2	1445	28.2	1445	28.2	
80:20	1432	28.2	1446	26.7	1432	26.4	
85:15	1439	28.2	1460	27.8	1449	28.0	
90:10	1500	24.8	1483	25.4	1492	25.1	

The maximum dry density of blended cinder increased with increment in the blending ratio of the material indicating improved compaction and binding properties. Thus, it indicates that the mechanical properties of the material had been improved by blending. Compared with blending with weathered rock, cinder gravel had a lower maximum dry density when blended with red soil.

4.3.3 CBR/moisture content relationship of cinder gravel blended with red soil

After compaction of each mould, unsoaked CBR for the blended cinder was determined using Multispeed CBR Tester equipment for neat material. As for cinder gravel blended with weathered rock in section 4.2.4, blending with red soil depicted similar characteristics in that the maximum CBR did not coincide with MDD. The explanation for this characteristic is similar as explained for the weathered rock. The highest CBR at various blending ratios is presented in Table 4.9.

Table 4.9: CBR/MC relationship of blending ratios with red soil

Blending ratio	**Samples**						**Remarks**
	A		**B**		**Average**		
	CBR (%)	**MC (%)**	**CBR (%)**	**MC (%)**	**CBR (%)**	**MC (%)**	
70:30	99	23.2	148	24.1	123.5	23.65	The average highest CBR is 123.5% at 23.65% MC which did not occur at MDD of 1488 Kg/m^3 at OMC of 27.3% in Table 4.8.
75:25	130	24.75	121	25	125.5	25	The average highest CBR is 125.5% at 25% MC which did not occur at MDD of 1445 Kg/m^3 at OMC of 28.2% in Table 4.8.
80:20	140	25.3	148	24.0	144	24.7	The average highest CBR is 144% at 24.7% MC which did not occur at MDD of 1432 kg/m^3 at OMC of 26.4% in Table 4.8.
85:15	150	25	-	-	150	25	The highest CBR is 150% at 25% MC which did not occur at MDD of 1449 kg/m^3 at OMC of 28% in Table 4.8.
90:10	130.5	19.5	119	19.8	125	19.7	The highest CBR is 125% at 19.7% MC which did not occur at MDD of 1492 kg/m^3 at OMC of 25.1% in Table 4.8.

It is deduced from the results that;

(i) The average highest CBR of cinder gravel blended with red soil was lower than for blended with weathered rock for similar ratios. This indicates that blending with red soil results in lower strength.

(ii) Just like cinder gravel blended with weathered rock, the highest CBR value did not occur at MDD/OMC due to the development of pore pressures which negates the pressure / loads impacted on the material with further compaction. The finding should be considered in the specification of the material for construction.

4.3.4 Atterberg's Limits of cinder gravel blended with red soil

As observed in Table 4.2, neat cinder from Thanantu B/P was non-plastic and was blended with red soil to improve on its plasticity. After the MDD/OMC determination and CBR test of various blending ratios, the fines (% < 425 μm) from these specimens were used to determine the Atterberg limits (plasticity modulus) of blended cinder. Plasticity characteristics of the five cinder blending ratios are given in Table 4.10;

Table 4.10: Atterberg's Limits of cinder gravel blended with red soil at various blending ratios

Sample	LL (%)	PL (%)	PI (%)	LS (%)	% Passing 425μm Sieve	PM
Blending ratio 70:30						
A	63	35.9	27.1	10	46.7	1261
B	64.3	36.3	28	10.7	48.6	1361
Average	63.7	36.1	27.6	10.4	47.65	1311
Blending ratio 75:25						
A	63.4	37	26.4	11.4	35.4	935
B	63.8	43.5	20.3	10.7	35.5	721
Average	63.6	40.1	23.5	11.1	35.5	834
Blending ratio 80:20						
A	64.3	35.3	29	12.9	38.4	1114
B	64	31.6	32.4	12.1	39.6	1283
Average	64.2	33.5	24.1	12.5	39	1199
Blending ratio 85:15						
A	61.9	37.9	24	10	30.9	742
B	58.5	37.4	21.1	8.6	41.8	882
Average	60.2	37.65	22.6	9.3	36.35	822
Blending ratio 90:10						
A	59.4	36.7	22.7	7.9	33.3	756
B	58.5	45	13.5	7.1	36.1	487
Average	58.95	40.85	18.1	7.5	34.7	628

From the Atterberg limits tests it was deduced that cinder gravel blended with red soil ratios 70:30, 75:25, 80:20, 85:15 and 90;10 did not meet the minimum plasticity requirements of 15 for pavement layers in RDM Part III and the Pavement Design Guidelines for LVSRs. The red soil binding material sourced from within Thanantu B/P had high plasticity for improvement by mechanical stabilization only of cinder gravel. Thus, chemical stabilization with lime or cement would be necessary to improve the material for suitability for road pavement construction.

4.3.5 Grading of cinder gravel blended with red soil

Grading of cinder gravel blended with red soil for various ratios were carried out to check if it conformd with the requirements for subbase and base materials for LVSRs. Grading was carried out after compaction in accordance with the procedure described in the methodology and the results are presented in Figures 4.12(a) and (b).

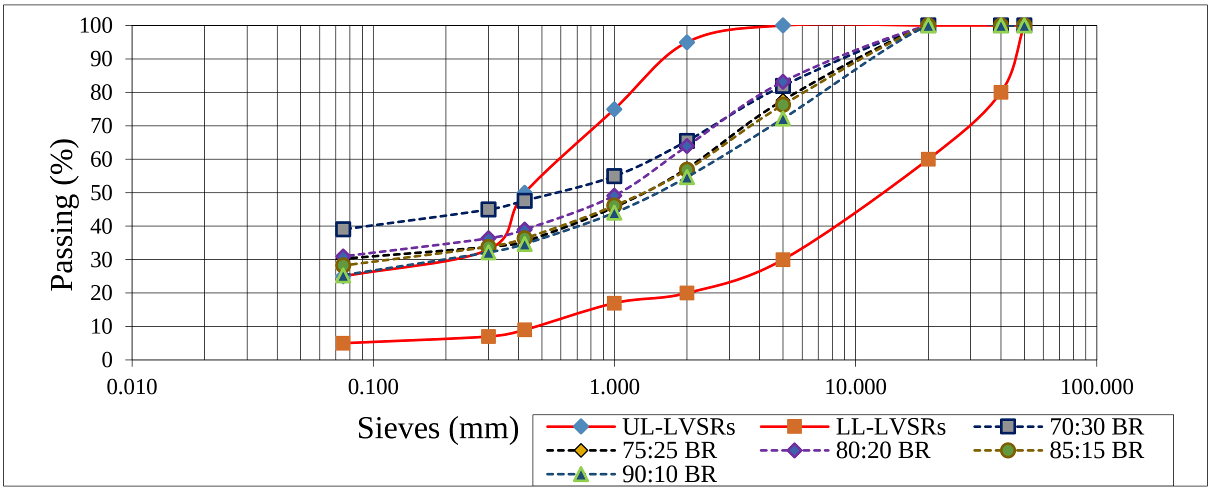

Figure 4.12(a): Grading curves of cinder blended with red soil against grading requirements for subbase materials (G30) for LVSRs

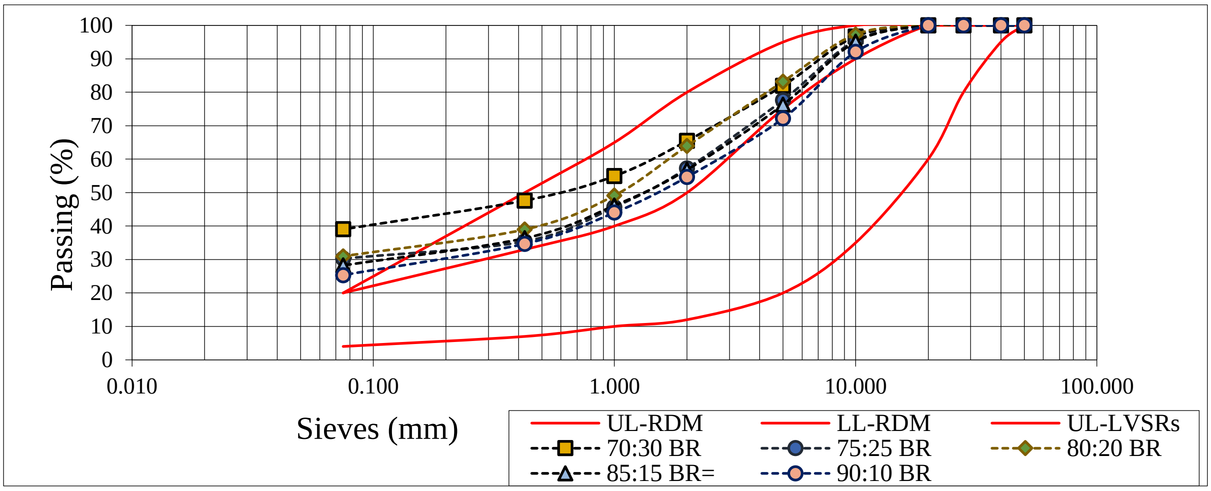

Figure 4.12(b): Grading curves of neat cinder blended with red soil against grading requirements for base materials in RDM-III and (G80) for LVSRs

The grading curves showed that none of the blending ratios conformed with the requirements because red soil was more finer and therefore not suitable to improve the properties of the material.

4.4 Comparison of properties of cinder gravel blended with weathered rock and red soil

Cinder gravel was blended with weathered rock and red soil in similar proportions ranging from 90:10 to 70:30 with increments of 5% and comparison of the properties made in Table 4.11.

Table 4.11: Comparison of properties of cinder gravel blended with weathered and red soil at various blending ratios

Blending ratio	**Cinder gravel blended with locally available fine material**					
	Weathered rock			**Red soil**		
	MDD (Kg/m3)	**Max Unsoaked CBR (%)**	**PI (%)**	**MDD (Kg/m3)**	**Max Unsoaked CBR (%)**	**PI (%)**
70:30	1512	148	19.2	1488	123.5	27.6
75:25	1463	160	15.8	1445	125.5	23.5
80:20	1487	149	14.9	1432	144	24.1
85:15	1451	152	15.5	1449	150	22.6
90:10	1409	123.5	13.1	1492	125	18.1

The results from comparison of the two fine blending material in Table 4.11 showed that even though the MDD and strength of blended material improved significantly with blending with either of the two fine materials, blending with red soil depicted high plasticity. A further comparison graphically in Figure 4.13 showed that plasticity of blended material with red soil ranged between 20-30% while with weathered rock ranged between 13-20% for the five similar blending ratios. According to Smith and Smith (1988), the finer the soil the greater its plasticity index and the results in Table 4.11 showed that red soil is finer than weathered rock. Since blending cinder gravel with red soil didn't depict much positive results in plasticity, the blended material was not pursued further in the study for objective 2 and 3.

It was concluded from the study that improvement of cinder gravel through mechanical stabilization (blending) only with red soil did not meet specifications for pavement material for LVSRs. Therefore, chemical stabilization with lime or cement would be necessary to improve the material which may not be appropriate or cost effective for LVSRs where it is preferred that the initial cost of construction is kept low.

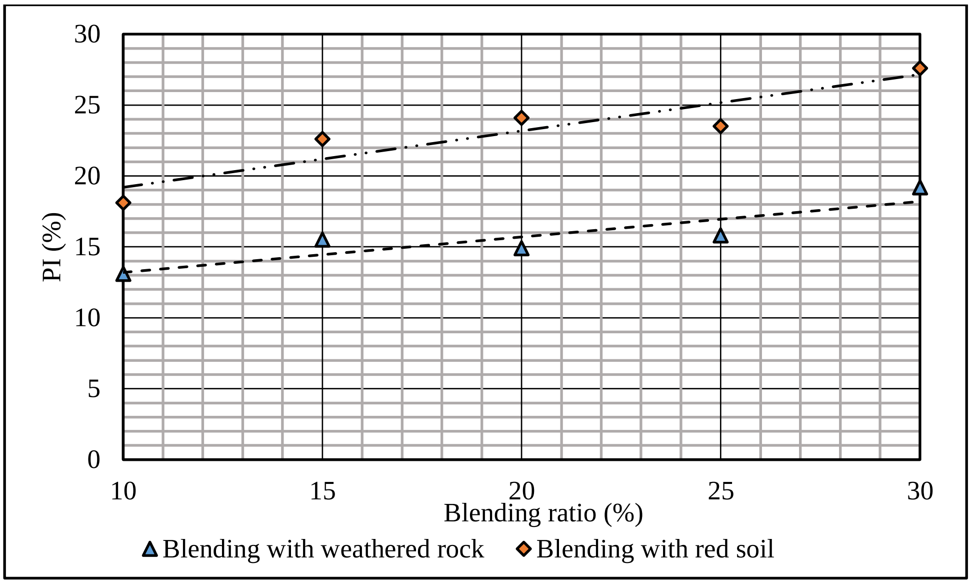

Figure 4.13: Comparison of blending ratio with PI for cinder gravel blended with weathered rock and red soil

4.5 Suitability of cinder gravel blended with weathered rock for road construction

The particle size distribution after CBR for blended cinder was carried out and compared with specifications and the results are presented in Table 4.12 and Figures 4.14(a) and (b).

Table 4.12: Grading of neat and cinder gravel blended with weathered rock in comparison with specifications

Material	**Deviation from specifications in grading after compaction**			**Remarks**
	Subbase materials for LVSRs (G30)	**Base materials in RDM Part III**	**Base materials for LVSRs (G80)**	
Neat cinder	< 1.5mm Particle sizes (LL)	< 0.4mm Particle sizes (LL)	< 0.4mm Particle sizes (LL)	Did not meet specifications in fine particles
Weathered Rock blending material	< 0.1mm Particle sizes (LL)	✓	✓	Met specifications
Blended cinder ratio 70:30	< 0.4mm Particle sizes (UL)	< 3mm Particle sizes (UL)	< 0.2mm Particle sizes (UL)	Did not meet specifications

Material	Deviation from specifications in grading after compaction			Remarks
	Subbase materials for LVSRs (G30)	**Base materials in RDM Part III**	**Base materials for LVSRs (G80)**	
Blended cinder ratio 75:25	< 0.4mm Particle sizes (LL)	< 0.2mm Particle sizes (LL)	0.2mm Particle sizes (LL)	Did not meet specifications
Blended cinder ratio 80:20	< 1.2mm Particle sizes (LL)	< 0.35mm Particle sizes (LL)	< 0.4mm Particle sizes (LL)	Did not meet specifications
Blended cinder ratio 80:20 (Repeated)	< 0.3mm Particle sizes (UL)	Outside the UL for all sieve sizes	< 0.15mm Particle sizes (UL)	Did not meet specifications
Blended cinder ratio 85:15	< 0.3mm Particle sizes (LL)	< 0.2mm Particle sizes (LL)	< 0.2mm Particle sizes (LL)	Did not meet specifications
Blended cinder ratio 90:10	✓	> 5mm particle sizes (UL)	✓	Met specifications for subbase and base materials

Note;

LL – Lower Limits of the grading curve specifications

UL – Upper Limits of the grading curve specifications

G30 - denotes natural or blended (mechanically stabilised) granular materials of minimum CBR strength of 30% measured after 4 days soak and so on for G80.

It is deduced from Table 4.12 that;

i. For grading after compaction, cinder gravel blended with weathered rock for 90:10 ratio met specifications requirements for subbase materials (G30) and Base materials (G80) for LVSRs;
ii. Grading of cinder gravel blended with weathered rock improved with decrease in the blending ratios because cinder gravels breaks down to finer particles during compaction.

The grading curves in Figures 4.14(a) and (b) showed that the 90:10 blending ratio met the grading specification requirements for subbase materials (G30) for LVSRs. The other blending ratios (70:30; 75:25; 80:20 and 85:15) were off the requirements in the lower sieve sizes. The optimum blending ratio 90:10 met the grading requirements for base materials (G80) for LVSRs.

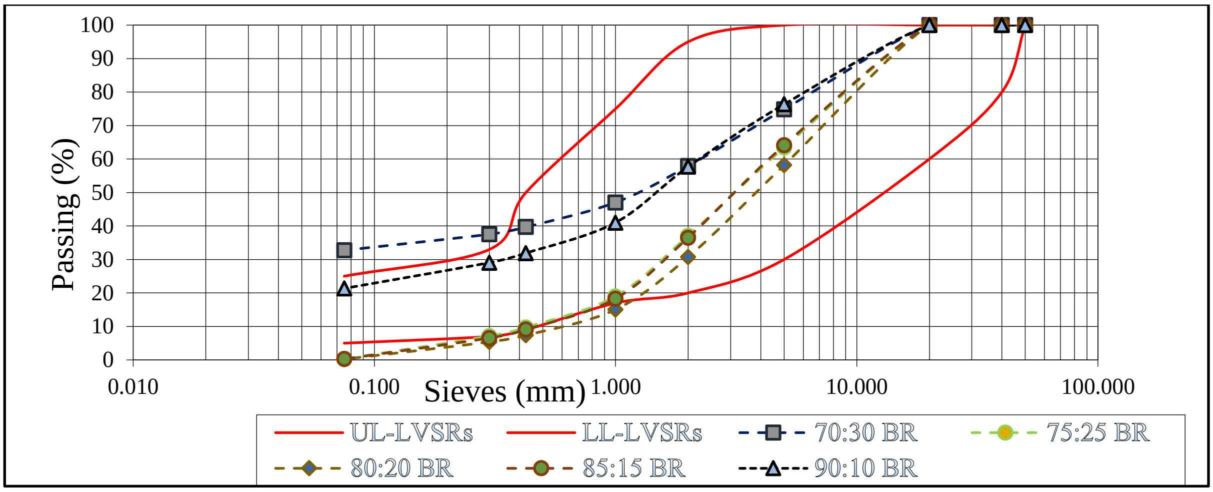

Figure 4.14(a): Grading of blending ratios against grading requirements for subbase materials (G30) for LVSRs.

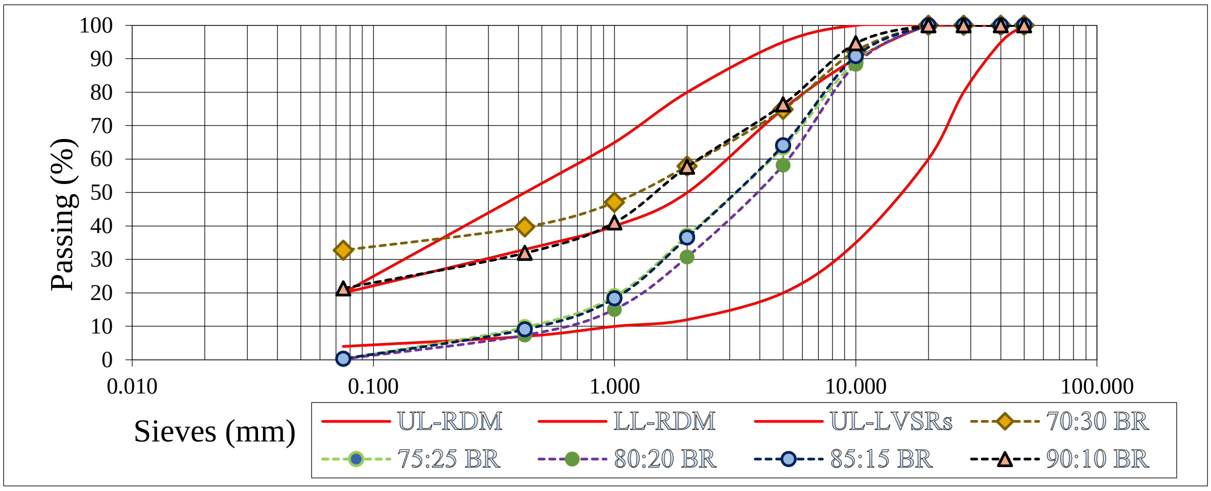

Figure 4.14(b): Grading of blending ratios against grading requirements for base materials in RDM-III and (G80) for LVSRs.

The results are consistent with the findings by Berhanu (2009) that the optimum amount of fine soil that makes up for the deficiency of fines in cinder gravel samples from both Alemgena and Lake Chamo areas in Ethiopia was 12%.

Table 4.13: Suitability of optimum blending ratio (90:10) as natural subbase and base materials for LVSRs

Natural or blended granular materials	Type of Test	Material requirements	Cinder Gravel blended with weathered rock 90:10 blending ratio		Application
			Test results	Meet Specification Requirements?	
G30 Granular Materials	Grading after compaction	Compliance with desirable limits	Meet specifications requirements for subbase materials (G30) for LVSRs	√	• Sub-base material for Design Traffic Class T5-0 • Base material for Design Traffic Class T5-4 & T5-3
	CBR at 95% MDD (AASHTO T180) and 4 days soak (%)	Min 30	• Unsoaked CBR 123.5% • Soaked CBR 86.3%	√	
	Plasticity Index (%)	Max 12	13.1	√; (PI of up to 20% may be tolerated in dry areas)	
G80 Granular Materials	Grading after compaction	Compliance with desirable limits	Meets specifications requirements for base materials (G80) for LVSRs	√	Base material for Design Traffic Class T5-0.
	CBR at 95% MDD (AASHTO T180) and 4 days soak (%)	Min 80	• Unsoaked CBR 123.5% • Soaked CBR 86.3%	√	
	Plasticity Index (%)	Max 10	13.1	√; (PI of up to 20% may be tolerated in dry areas)	

The results in Table 4.13 shows that cinder gravel blended with weathered rock at 90:10 blending ratio met the desirable grading limits for granular materials (G30) and (G80) for LVSRs. The grading curves for all the blending ratios are given in Figures 4.14(a) and (b). Although the plasticity of blended cinder in Table 4.13 was above the maximum requirements for natural subbase and base material for various traffic classes, the Kenyan pavement design guideline for LVSRs permits higher plasticity of up to 20% in dry areas (less than 500mm annual rainfall) where cinder material is found to be predominant in the country.

4.6 Repeated cycle compaction of optimum blended cinder gravel with weathered rock

From the comparative analysis of the various blending proportions of cinder gravel with weathered rock, the 90:10 proportions gave an optimum result of Atterberg's limits, sieve analysis and maximum dry density. The blending ratio conformed with material specifications requirements for blended granular materials (G30) for subbase and blended granular materials (G80) for base for LVSRs. The maximum dry density of blended cinder gravel of 90:10 ratio was 1410 Kg/m^3 and the optimum moisture content was 26.9% as shown in Figure 4.15.

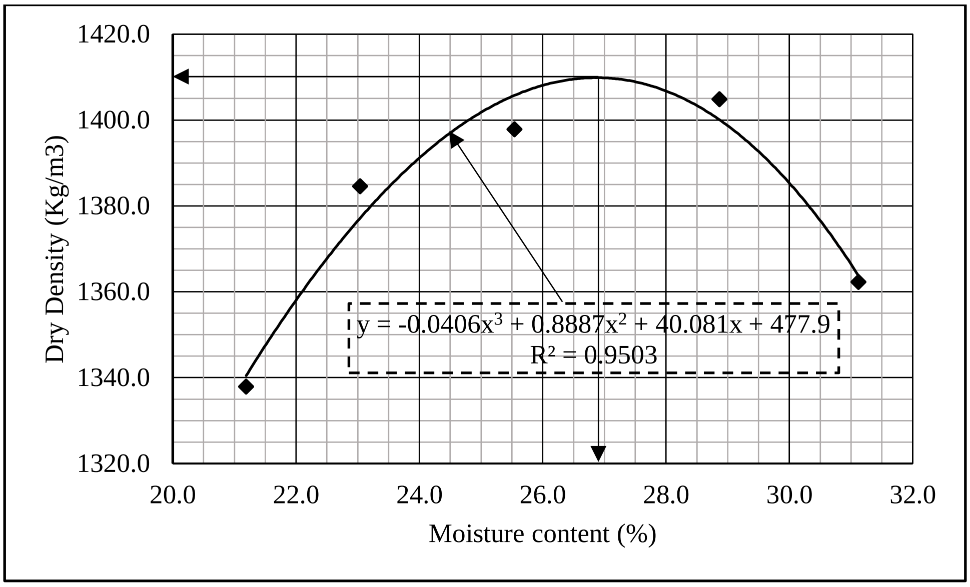

Figure 4.15: MDD / OMC of optimum blending ratio 90:10

The relationship between dry density and moisture content of the optimum blending ratio 90:10 is a parabolic curve represented by the equation;

$\rho_d = -0.0406MC^3 + 0.8887MC^2 + 40.081MC + 477.9$…………………**Equation 4.3**

$R^2 = 0.9503$

where;

ρ_d = Dry density of the optimum blending ratio (Kg/m^3)

MC = Moisture content (%)

R^2 = Coefficient of correlation (variation of observed values to the trend line curve)

The second objective of the study "To evaluate the strength and grading of Cinder gravel at different levels of compaction" was determined by preparing 5 specimens at 90:10 proportions at optimum MDD and OMC. The results of the engineering properties of cinder gravel under repeated cycles of moulding-compaction are given in Table 4.14 and discussed further in sections 4.6 and 4.7.

Table 4.14: Engineering characteristics of cinder gravel under repeated compaction

Compaction Cycles	**MDD (Kg/m^3)**	**Unsoaked CBR (%)**	**PI (%)**
One Cycle Compaction	1355	115	12.4
Two Cycle Compaction	1427	178	13.8
Three Cycle Compaction	1445	164	13.7
Four Cycle Compaction	1472	179	11.2
Five Cycle Compaction	1489	178	15.1

The results in Table 4.14 shows that the dry density/ moisture content, strength and plasticity of the optimum blending ratio of cinder gravel increased with compaction cycles as the material broke down into finer particles.

4.6.1 Comparison of compaction cycles with maximum dry density

Five specimens of 90:10 blending ratio at optimum MDD and OMC were prepared in five moulds (152.40mm diameter). The test was done in accordance with AASHTO T180-D standard procedure and in five factorial compaction cycles. The MDD, CBR, Moisture content, Atterberg's Limits were determined for each stage of compaction cycle. A graph of MDD (ordinate) versus the number of compaction cycles (abscissae) was plotted as shown in Figure 4.16.

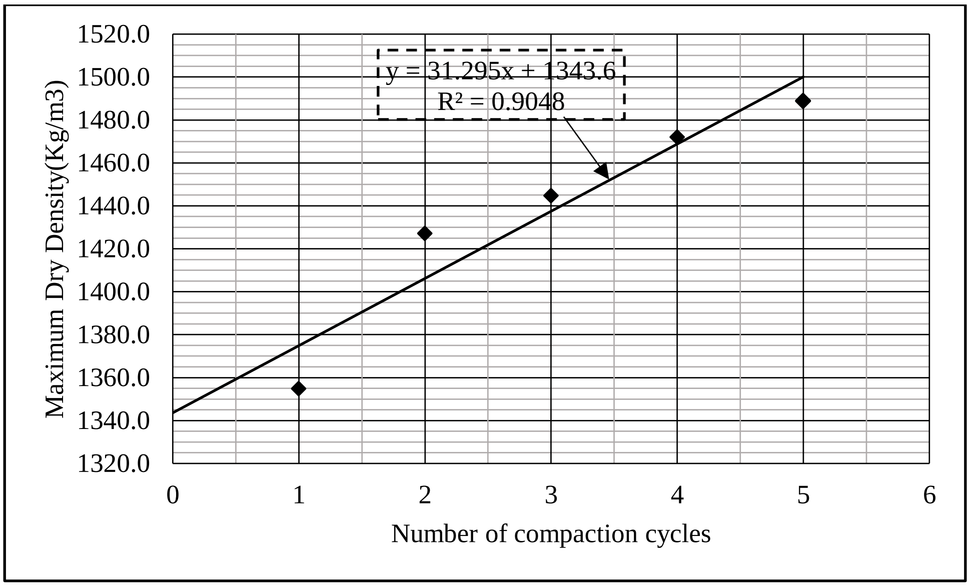

Figure 4.16: Correlation between Number of compaction cycles and MDD for the optimum blending ratio.

The relationship between the number of compaction cycles for the optimum blending ratio and the maximum dry density of the blended material is expressed by the trend line equation:

$MDD = 31.295NC + 1343.6$……………………………………………**Equation 4.4**

$R^2 = 0.9048$

where;

MDD = Maximum dry density in Kg/m^3

NC = Number of compaction cycles

R^2 = Coefficient of correlation (variation of observed values to the regression line)

It is deduced from Figure 4.16 that;

(i) The maximum dry density of the blended material increased with the number of compaction cycles indicating better interlocking of the particles of the material.

(ii) Repeated compaction of cinder gravel significantly improved the properties of the material and there should be a balance between having an improved material and the cost to achieve the improvement in the field during construction of pavement layers.

4.6.2 Comparison of compaction cycles with strength (unsoaked CBR)

The unsoaked CBR of optimum blending ratio 90:10 was determined for each compaction cycle at optimum MDD and OMC. A graph of maximum CBR for each compaction cycle was plotted for comparison and is presented in Figure 4.17.

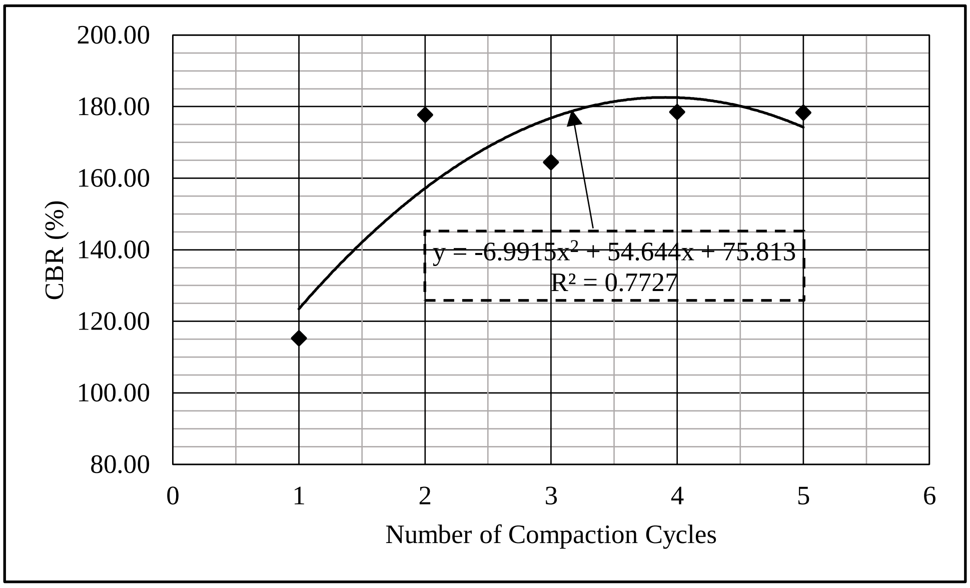

Figure 4.17: Correlation between Number of compaction cycles and CBR of the optimum blending ratio

The relationship between the number of compaction cycles for the optimum blending ratio and the maximum strength (CBR) of the blended material is expressed by the quadratic curve equation:

$CBR = -6.9915NC^2 + 54.644NC + 75.813$ ………………………………**Equation 4.5**

$R^2 = 0.7727$

where;

CBR = Unsoaked maximum strength of the blended cinder gravel (%)

NC = Number of compaction cycles

R^2 = Coefficient of correlation (variation of observed values to the trend line curve)

It is deduced from Figure 4.17 that;

(i) The strength of blended cinder gravel as represented by CBR increased with compaction showing that re-using the specimen is beneficial and therefore repeated scarification and compaction will be beneficial on site.

(ii) The maximum CBR of the optimum blending ratio 90:10 in Table 4.6 was 123.5% which compared with the CBR obtained for one cycle compaction in Figure 4.17 of 123.5%.

A graph of CBR vs maximum dry density obtained in section 4.6.1 and 4.6.2 was plotted as shown in Figure 4.18;

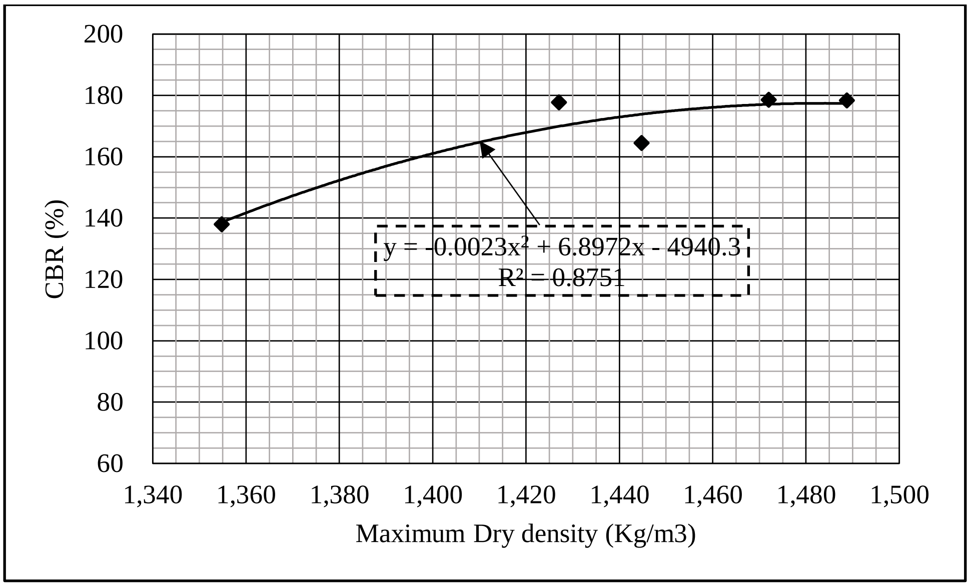

Figure 4.18: Correlation between MDD and CBR for repeated compaction cycles of blended cinder gravel

The relationship between maximum dry density for the optimum blending ratio and the maximum strength (CBR) of the blended material is expressed by the parabolic curve equation:

$CBR = -0.0023MDD^2 + 6.8972MDD + 4940.3$……………………… **Equation 4.6**

$R^2 = 0.8751$

where;

CBR = Unsoaked maximum strength of the blended cinder gravel (%)

MDD = Maximum dry density (Kg/m^3)

R^2 = Coefficient of correlation (variation of observed values to the trend line curve)

It is deduced from Figure 4.18 that strength of the material subjected to repeated compaction cycles as represented by CBR increased with the dry density. This was because cinder gravel breaks down further to finer particles with compaction which results in improved interlocking of particles in the material.

4.6.3 Atterberg's Limits of cinder gravel under repeated cycle compaction

Atterberg's limits tests were carried out for each cycle compaction after determination of maximum dry density and CBR tests. The results are presented in Table 4.15 and deduction made.

Table 4.15: Atterberg's Limits of cinder gravel under repeated cycle compaction

Compaction Cycles	**Sample**	**LL (%)**	**PL (%)**	**PI (%)**	**LS (%)**	**% Passing 425µm Sieve**	**PM**
One Cycle Compaction	A	46.7	30.9	15.8	4.3	33.7	533
	B	48.9	40.0	8.9	5.0	38.3	341
	Average	47.8	35.5	12.4	4.7	36.0	446
Two Cycle Compaction	A	47	31.1	15.9	5.7	38.2	607
	B	47.8	36.2	11.6	5.0	40.4	469
	Average	47.4	33.7	13.8	5.4	39.3	542
Three Cycle Compaction	A	48.2	34.6	13.6	6.4	47.1	641
	B	46.8	32.9	13.9	5.7	44.1	612
	Average	47.5	33.8	13.7	6.1	45.6	625
Four Cycle Compaction	A	45.7	33.4	12.3	5.7	47.1	579
	B	45.9	35.8	10.1	7.1	47.6	481
	Average	45.8	34.6	11.2	6.4	47.4	531
Five Cycle Compaction	A	46.3	34.4	11.9	6.4	51.0	607
	B	46	27.7	18.3	7.1	51.0	933
	Average	46.2	31.1	15.1	6.8	51.0	770

Analysis of the results in Table 4.15 showed that there was a gradual increase in Plasticity Index with the number of compaction cycles to a maximum before it dropped. This was due to breakage of cinder gravels into finer particles with further compaction as illustrated in Figure 4.19. This characteristic corroborates with increase in MDD and CBR in Figure 4.16 and 4.17 due to enhanced interlocking of fine particles with compaction cycles.

The results from Atterberg's limits tests showed that cinder gravel blended with weathered rock subjected to one cycle compaction and four-cycle compaction met the maximum Plasticity Index (PI) requirements for granular materials (G30). The material was suitable as subbase for T5-0 Traffic class and as base materials for T5-4 and T5-3 Traffic class for LVSRs as given in Table 4.13. Also, all the five (5) repeated mould-compact cycles met the maximum requirements of plasticity for pavement materials in dry areas as provided for in RDM Part III and Pavement Design Guideline for LVSRs.

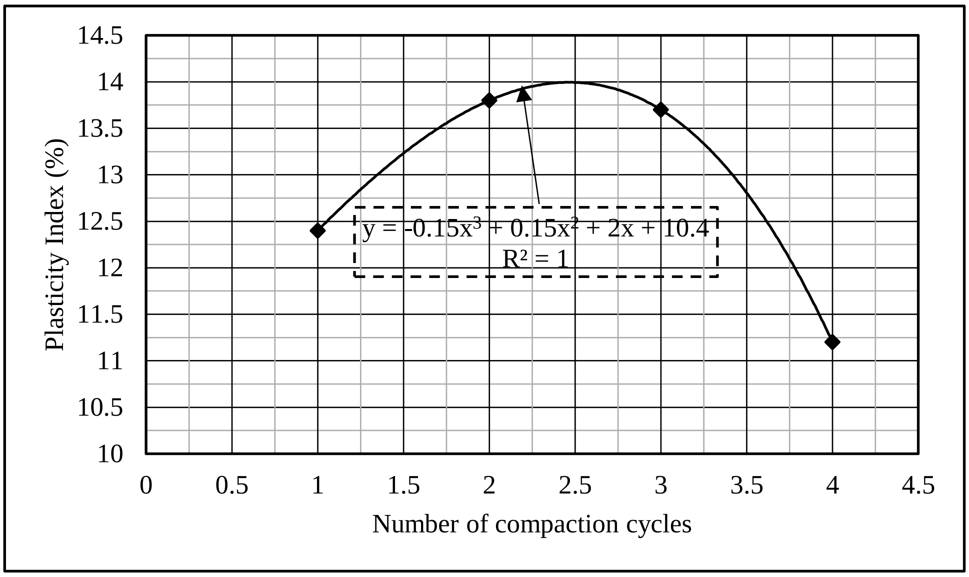

Figure 4.19: Correlation between Number of compaction cycles and Plasticity Index of blended cinder gravel with weathered rock

The relationship between the number of compaction cycles and plasticity index for the optimum blending ratio is expressed by the parabolic curve equation:

$PI = -0.15NC^3 + 0.15NC^2 + 2NC + 10.4$……………………………**Equation 4.7**

$R^2 = 1$

where;

PI = Plasticity index of the blended cinder gravel (%)

NC = Number of compaction cycles

R^2 = Coefficient of correlation (variation of observed values to the trend line curve)

4.6.4 Grading of cinder gravel under repeated cycle compaction

Grading after compaction was carried out for each cycle compaction after determination of maximum dry density and CBR tests. The results are presented in Table 4.16 and grading curves are presented in Figure 4.20(a) and (b) and deduction made.

Table 4.16: Grading of cinder gravel under repeated cycle compaction in comparison with specifications

Compaction Cycles	Deviation from Specifications for Grading after compaction		Remarks
	Subbase materials for LVSRs (G30)	Base materials for LVSRs (G80)	
One Cycle Compaction	✓	✓	Met specifications for subbase and base materials
Two Cycle Compaction	✓	✓	Met specifications for subbase and base materials
Three Cycle Compaction	< 0.4mm Particle sizes (UL)	< 0.3mm Particle sizes (UL)	Did not meet specifications
Four Cycle Compaction	< 0.4mm Particle sizes (UL)	< 0.3mm Particle sizes (UL)	Did not meet specifications
Five Compaction Cycle	< 0.4mm Particle sizes (UL)	< 0.4mm Particle sizes (UL)	Did not meet specifications

Note;
LL – Lower Limits of the grading curve specifications
UP – Upper Limits of the grading curve specifications

It is deduced from Table 4.16 and Figure 4.20(a) and (b) that;

(i) One and two cycle compactions met specification requirements for grading for subbase materials (G30) and base materials (G80) for LVSRs. The higher compaction cycles (three, four and five) had higher percentage of particles passing the lower sieve sizes less than 0.3mm because cinder gravels break down to smaller particles when subjected to repeated compaction.

(ii) Repeated compaction of blended cinder gravel impacts on grading specification of the material due to breakage to finer particles.

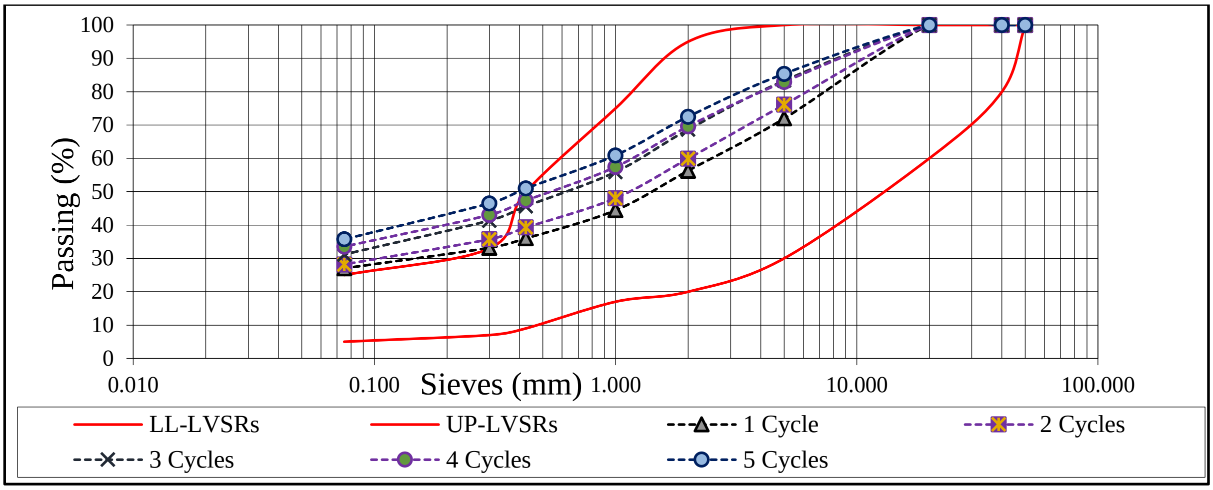

Figure 4.20(a): Grading of compaction cycles against grading requirements for subbase materials (G30) for LVSRs.

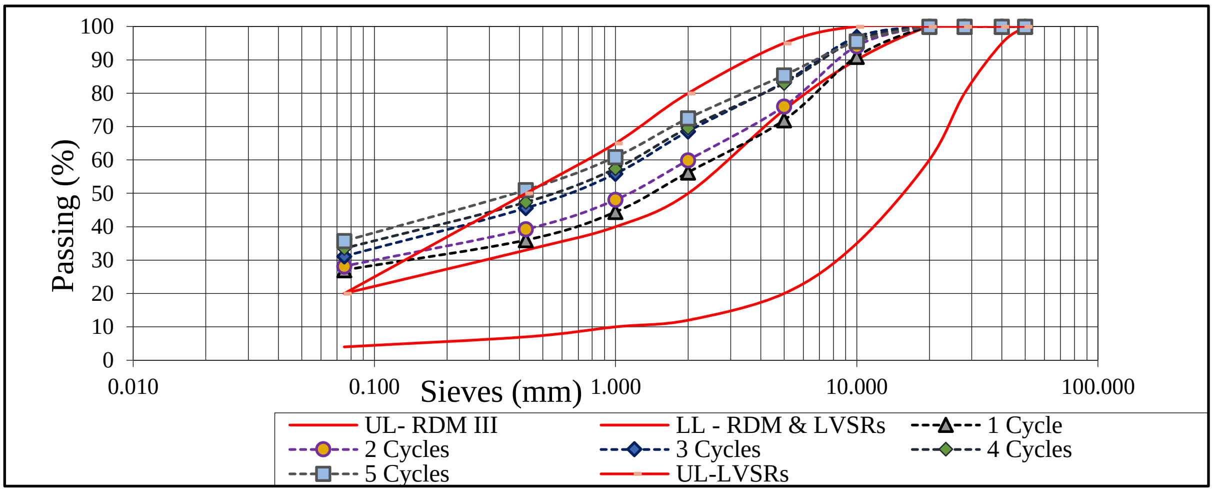

Figure 4.20(b): Grading of compaction cycles against grading requirements for base materials (G80) for LVSRs.

4.7 Effect of soaking on the strength properties of cinder gravels

Neat and cinder gravels blended with weathered rock subjected to one, three and five repeated compaction cycles were moulded at MDD and OMC to check the effect of soaking on swell and strength properties of the material. The initial and final swell reading were recorded, and the percent swell of the specimen determined and given in Table 4.17.

Table 4.17: Percent swell of neat and blended cinder gravel

Specimen	% Swell after soaking			Remarks
	A	B	Average	
Neat cinder gravel	0.01	0.07	0.04	No swell
One mould compaction cycle	0.30	Nil	0.30	No swell
Three mould compaction cycles	Nil	Nil	Nil	No swell
Five mould compaction cycles	Nil	Nil	Nil	No swell

Table 4.17 shows that neat and blended cinder gravel did not swell through soaking an indicator of low moisture absorption or porous nature of the material.

The moulded material in the CBR mould was soaked for 4 days and penetration was done to determine the strength (CBR) of neat and blended cinder gravel when soaked. The CBR at the top of the specimen depicted higher value than the bottom which is the norm for soaked material. A comparison of the strengths after soaking and for unsoaked specimen was made and presented in Table 4.18.

Table 4.18: Comparison of strength for soaked and unsoaked specimen

Specimen	CBR (%)		Remarks
	Average soaked CBR	Average unsoaked CBR	
Neat cinder Gravel	16.11	113	All specimens depicted low strength when soaked
One mould compaction cycle	86.3	115.26	
Three mould compaction cycles	26.05	164.43	
Five mould compaction cycles	14.8	178.32	

It is deduced from Table 4.18 that for soaked specimen the CBR decreased as the cycles of compaction increased. This was because with ingress of water when soaked the finer particles of cinder gravel disperses and loose the interlocking properties observed with repeated compaction where the MDD and CBR was increasing steadily as compaction cycle increases. The findings are further presented graphically in Figure 4.21.

It is observed from Figure 4.21 that the proving ring dial reading was higher for one cycle compaction than for three, five and neat cinder gravel in that order. This showed that the material had higher strength when optimally compacted and weak when over compacted when soaked.

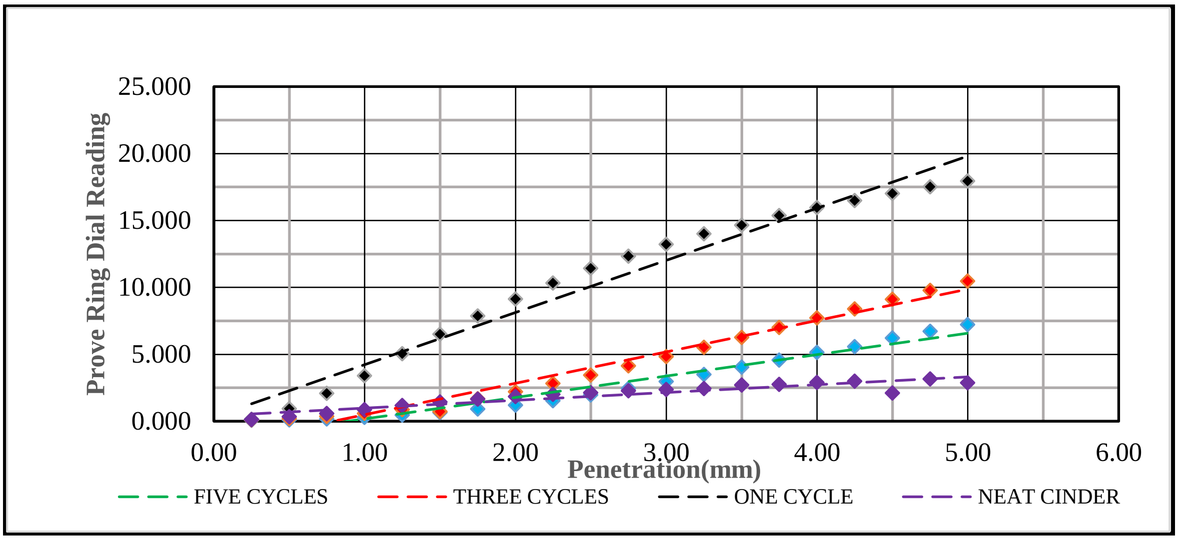

Figure 4.21: Variation of strength with compaction cycles for soaked specimen

4.8 Relationship between shear strength of cinder gravel and particle size using shear box test

The third of objective of the study was "To investigate the relationship between shear strength of cinder gravel and particle size using shear box test." The test was carried out to establish the effect of compaction on the shear value of cinder gravel. The MDD and OMC of the optimum blending ratio 90:10 was determined before moulding the specimen for Shear box test. This was because the blending ratio conformed with material specifications requirements for subbase granular material (G30) and base granular materials (G80) for LVSRs. The maximum dry density of blended cinder gravel of 90:10 ratio was 1435Kg/m^3 and the optimum moisture content was 25.5% as shown in Figure 4.22.

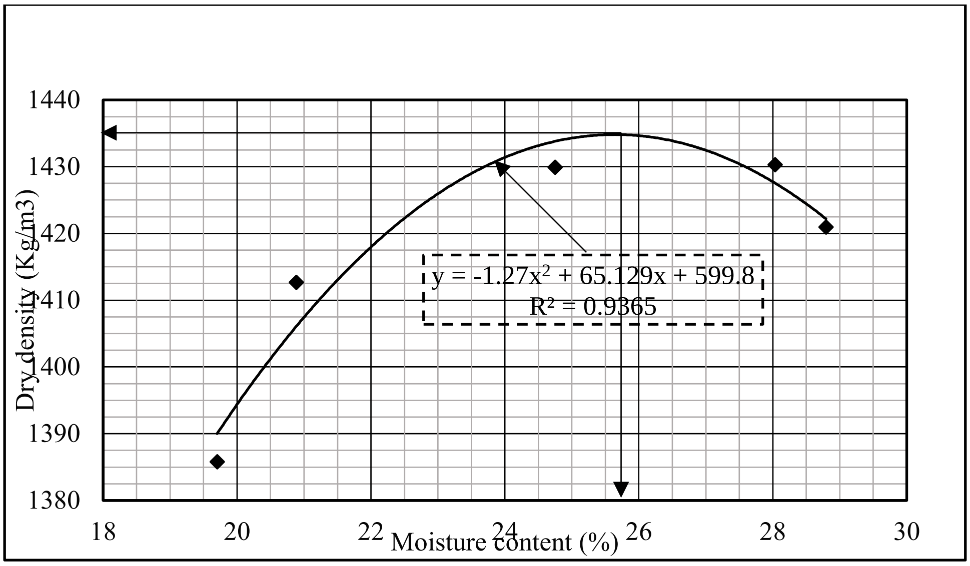

Figure 4.22: MDD and OMC of optimum blending ratio 90:10 for shear box test

The dry density / moisture content parabolic curve is expressed by quadratic curve equation;

$\rho_d = -1.27MC^2 + 65.129MC + 599.8$………………………………**Equation 4.8**

$R^2 = 0.9365$

where;

ρ_d = Dry density (Kg/m^3)

MC = Moisture content (%)

R^2 = Coefficient of correlation (variation of observed values to the trend line curve)

The mould was prepared at optimum MDD and OMC for each compaction cycle. At least three (3) specimens were extracted from compacted and remoulded cinder gravel for each Shear box test per compaction cycle as detailed in the methodology chapter. The shear value of each moulded specimen was determined by the direct shear test in the laboratory and the raw data obtained have been presented in **Appendix A**. It is deduced from the data that the shear stress of the optimum blended cinder gravel 90:10 ratio reduced with increased compaction cycles. As the cinder gravel particles break down to finer particles, the shear stress reduced even though not at significant proportions. The reduction in shear stress was more pronounced for the higher normal stress (3.04 Kg/cm^2) compared to the lower normal stress (1.02 Kg/cm^2). The results of normal stress verses shear stress in **Appendix A** are further presented in Figure 4.23. A graph of shear stress (ordinate) versus normal stress (abscissae) was plotted for each compaction cycle as shown in Figure 4.23. From the graph the angle of shearing resistance (φ) and unit of cohesion "c_u" were determined, and values compared for the five (5) compaction cycles. It is deduced from the graph that the shear stress of blended cinder gravel reduced with increased compaction cycles though not at significant proportions.

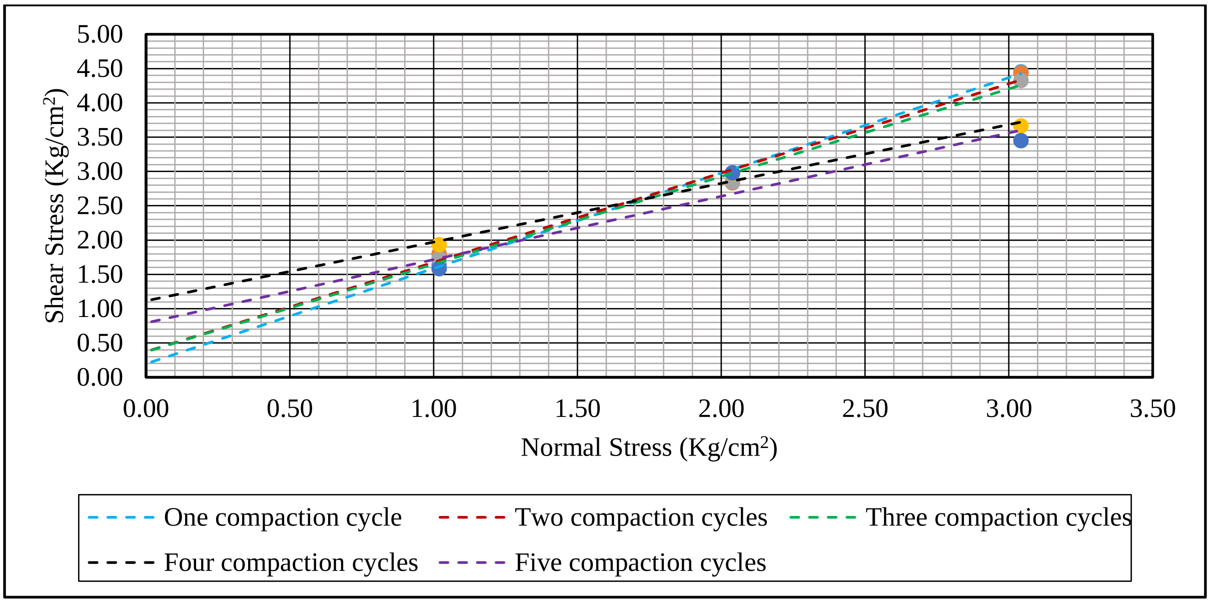

Figure 4.23: Normal stress (σ) verses Shear stress (τ) for various compaction cycles

According to Lupini et al (2015), cohesion is the component of the shear strength of soil that is independent of interparticle friction. Repeated compaction breakdown the larger particles down and when recompacted this brings the densification of the sample resulting in increase in the cohesion of the soil. A further analysis of graphs of shear stress verses normal stress of the optimum blended cinder gravel ratio for determination of the unit of cohesion (c_u) and the angle of shearing resistance (φ) of cinder gravel at different cycles of compaction is given in Table 4.19.

Table 4.19: Comparison of cohesion and shearing resistance of cinder gravel with cycle compaction

Compaction cycles	**Unit of cohesion (c_u) (Kg/cm²)**	**Angle of shearing resistance (φ°)**	**Shear strength (Kg/cm²)**
One cycle compaction	0.2	54.28	4.43
Two cycle compactions	0.4	52.45	4.35
Three cycle compactions	0.4	51.96	4.29
Four cycle compactions	1.1	40.57	3.7
Five cycle compactions	0.8	42.72	3.61

Results in Table 4.19 showed that the unit of cohesion (c_u) of cinder gravel increased with the number of compaction cycles. As the material becomes finer because of compaction, the particles became more cohesive and plastic to resist shear failure. Better cohesion was due to mutual attraction existing between fine particles that tends to hold them together in a solid mass. Thus, it indicates that the cohesive properties of the cinder gravel material improved with compaction as further shown in Figure 4.24.

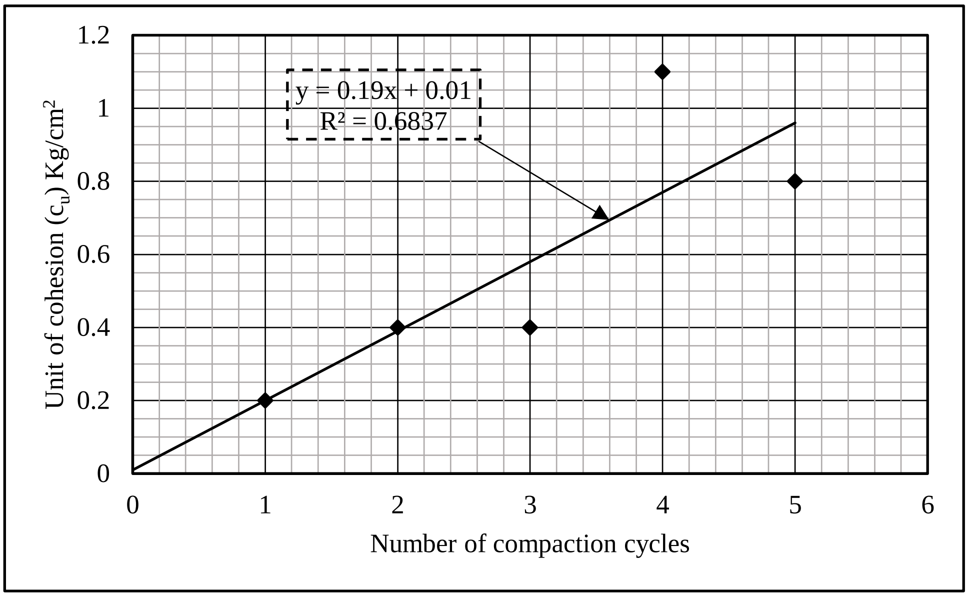

Figure 4.24: Correlation between unit of cohesion (c_u) with number of compaction cycles of cinder gravels

From Figure 4.24 it was observed that a linear relationship exists between number of compaction cycles (NC) and unit of cohesion (c_u) expressed by the trend line equation;

$C_u = 0.19NC + 0.01$…………………………………………………**Equation 4.9**

$R^2 = 0.6837$

where;

c_u = Unit of cohesion (c_u) in Kg/cm^2

NC = Number of compaction cycles

R^2 = Coefficient of correlation (variation of observed values to the trend line)

Further analysis of results in Table 4.19 showed that the angle of shearing resistance (φ) decreased with compaction cycles. This was also observed graphically in Figure 4.23 where the trendline is steeper for lower compaction cycles. According to Alias et al (2014) the angle of friction is dependent on the particle size, the particle-to-particle contact provides an interlocking surface that resist the shearing resistance. In repeated cycle compaction as the cycles of compaction increases the angle of shearing resistance decreases. This is a result of breaking down of larger particles into smaller particles which reduces the surface area for interlocking of the particles. This showed that amount of friction between soil particles decreased with compaction and so the interlocking of particles as shown in Figure 4.25.

The result is consistent with the findings by Wang et al (2013) that the angle of shearing resistance is generally increasing with increasing median particle diameter and gravel content. Wang et al (2013) investigated the effects of particle size distribution on shear strength of accumulation soil and concluded that larger size particles produced higher effective internal friction angle and developed high shear strength. These results agreed with previous studies by Wang et al (2013), Charles and Watts (1980) and Nakao and Fityus (2008) that the value of friction angle increases with increase in particle size.

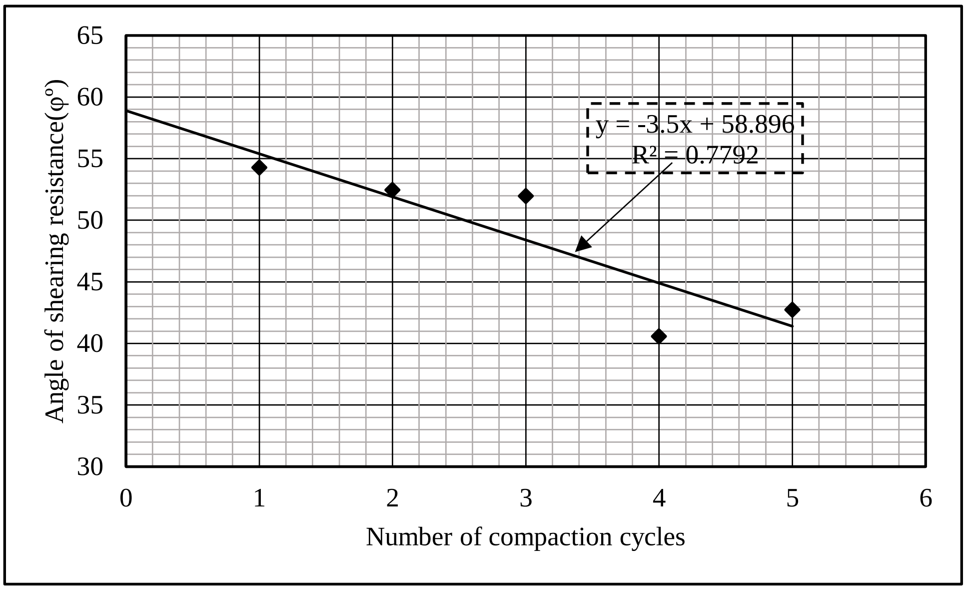

Figure 4.25: Correlation between angle of shear resistance (φ) with number of compaction cycles of cinder gravel

The relationship between the angle of shear resistance (φ) with number of compaction cycles of cinder gravels is expressed by the trend line equation;

$\varphi^o = -3.5NC + 58.896$……………………………………………**Equation 4.10**

$R^2 = 0.7792$

where;

φ^o = Angle of shear resistance

NC = Number of compaction cycles

R^2 = Coefficient of correlation (variation of observed values to the trend line)

The results in Table 4.19 showed that the shear stress at failure / shear strength of cinder gravel decreased with number of compaction cycles due to comparable decrease in the angle of shearing resistance with compaction as was observed in Figure 4.25. The enhanced cohesive component of the fine particles due to repeated compaction as illustrated in Figure 4.24 was not able to counter the decrease in shear strength due to decrease in the angle of shearing resistance. Even though it was observed in section 4.6.1 that the maximum dry density of cinder gravel increased with compaction cycles it was established from the study that it did not contribute to higher shear strength for cinder gravel as shown in Figure 4.26.

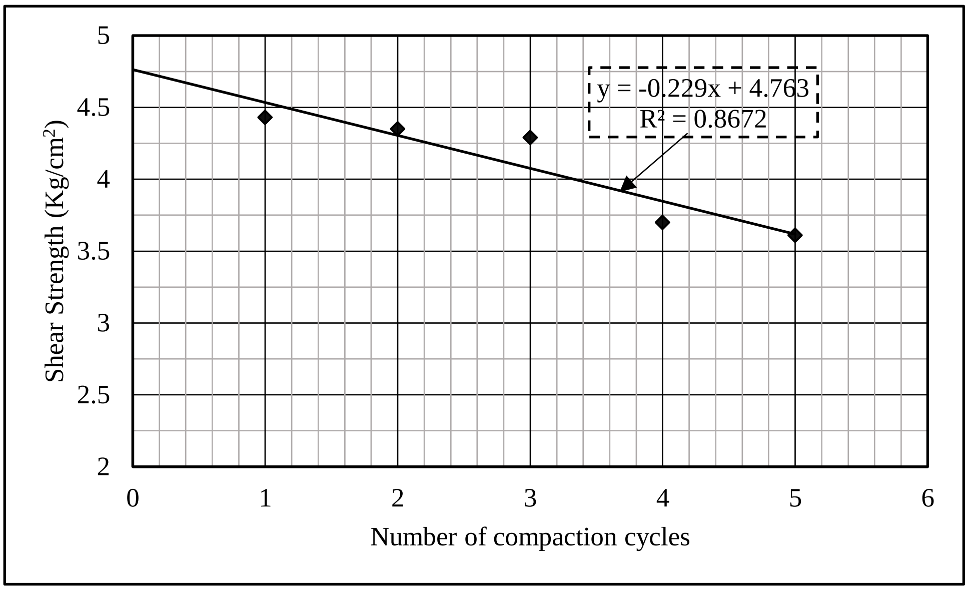

Figure 4.26: Relationship between shear strength (τ) with compaction cycles of cinder gravel

The linear relationship between the shear strength (τ) with number of compaction cycles of cinder gravels is expressed by the trend line equation;

$\tau = -0.229NC + 4.763$……………………………………………. **Equation 4.11**

$R^2 = 0.8672$

where;

τ = Shear strength

NC = Number of compaction cycles

R^2 = Coefficient of correlation (variation of observed values to the trend line)

Chapter Five: Conclusions and Recommendations

5.1 Conclusions

5.1.1 Overview

This chapter ascertains whether or not the findings of the study, have answered the research questions and objectives outlined in chapter one. In addition, the study draws several important conclusions, which form a basis for the recommendations. The overall objective of the study was to investigate the engineering characteristics of cinder gravel as road pavement construction material. To achieve the objective, the study evaluated the engineering properties of neat and blended cinder gravel sourced from Meru County, evaluated the strength and grading of cinder gravel at different levels of compaction and investigated the relationship between shear strength of cinder gravel and particle size using shear box test. The conclusions made were in accordance with the specific objectives of the research study.

5.1.2 Suitability of cinder gravel as road pavement construction material

Laboratory tests were carried out on neat and blended cinder gravel from Thanantu B/pit, red soil blending material and weathered rock blending material from Ntoombo B/Pit and from analysis of tests results the following conclusions were made on the engineering characteristics of the material;

- Cinder gravel sourced from Thanatu B/P was suitable for road pavement construction material for LVSRs when blended with weathered rock sourced from Ntoombo B/P at the optimum blending ratio of 90% cinder + 10% weathered rock. The blended granular material met the specifications requirements for subbase and base materials for LVSRs.
- Cinder gravel sourced from Thanantu borrow pit blended with red soil sourced from borrow pits adjacent to the cinder gravel borrow pit showed high plasticity and lower MDD and CBR of similar proportions with weathered rock. The improvement of cinder gravel through mechanical stabilization (blending) with red soil did not achieve the specification requirements for road pavement construction for LVSRs. This

requires the use of chemical stabilisation (lime, pozzolans, cement and others) which may portend prohibitive for LVSRs for LVSRs where it is preferred that the initial cost of construction is kept low.

- The maximum CBR of cinder gravel blended with either weathered rock or red soil in the study occurred before the MDD/OMC meaning that higher strength of the material was achieved at lower moistures. It was therefore concluded that cinder gravel was suitable as road pavement construction material in dry areas (ASAL) where the annual rainfall is less than 500mm.
- The highest CBR value of blended cinder gravel did not occur at MDD/OMC and this should be taken into consideration in the specifications for the material.

5.1.3 Strength and grading of cinder gravel at different levels of compaction

An evaluation of the strength and grading of the material at different levels of compaction was done for the optimum blending ratio 90% cinder + 10% weathered rock that conformed with material specifications requirements for subbase and base materials for LVSRs and the following conclusions were made;

- Repeated compaction of cinder gravel significantly improved the properties of the material and there should be a balance between having an improved material and the cost to achieve the improvement in the field during construction of pavement layers.
- The strength of cinder gravel decreased with repeated compaction when soaked due to the dispersal nature of the particles of the material with water ingress. This confirmed suitability of the material for dry compaction and use as a pavement material in dry areas (ASAL) where the annual rainfall is less than 500mm.
- Where cinder gravel has been used for pavement construction, the layer should be properly sealed with an impervious overlying material to prevent water ingress.

5.1.4 Relationship between shear strength of cinder gravel and particle size using shear box test

An evaluation of the shear strength of cinder gravel and particle size at different levels of compaction was done for the optimum blending ratio 90% cinder + 10% weathered rock that conformed with material specifications requirements for subbase and base materials for LVSRs and the following conclusions were made;

- The cohesive properties of cinder gravel improved with compaction cycles as the particles became finer.
- Repeated cycle compaction of cinder gravels was not beneficial for slope stability in high embankments in fills due to decreased frictional resistance and reduction in shear strength.

5.2 Recommendations

The recommendations are given in two parts; the first part covers recommendations for use of cinder gravel as road pavement construction material from the study while the second part covers recommendations for further research.

5.2.1 Recommendations from the study

i. The research was based on laboratory tests of cinder gravel where conditions are controlled / regulated and there is need to validate the findings in the field where the material has been used to construct pavement layers.
ii. Improvement of cinder gravel through mechanical stabilization (blending) only with red soil did not meet the plasticity specifications for LVSRs. It is recommended that chemical stabilization with lime or cement be done for suitability of the blend for road pavement construction for LVSRs.
iii. It is recommended that cinder gravel is suitable for use as road pavement construction material in dry areas (ASAL) where the annual rainfall is less than 500mm.

5.2.2 Recommendations for further research

(i) Conducting field undisturbed tests on the road where cinder gravels have been used as a pavement material to predict road pavement performance of LVSRs.
(ii) The finding from the study calls for further investigation of the strength and densification properties of cinder gravels before specifying requirements of the material for pavement construction.

(iii) Carry out repeated cycle compaction of cinder gravel for higher blending ratios with locally available fine materials to compare the strength properties of the material with soaking.

(iv) During investigation it was established that cinder gravel varied considerably in physical properties and characteristics for different quarries. It is therefore recommended that further research be carried out for suitability of cinder gravels sourced from other parts of the country where the material is readily available for comparison with the findings from this study.

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Appendices

Appendix A: Shear stress verses normal stress of blended cinder gravel

Compaction cycles	Sample No.	Load applied (Kg)	Weight of Hanger (Kg)	Total Weight (Kg)	Proving Ring (Div)	Normal Stress (σ) (Kg/cm^2)	Shear Stress (τ) (Kg/cm^2)
One cycle compaction	90:10 A	32.2	4.5	36.7	795	1.02	1.64
	90:10 B	68.9	4.5	73.4	1446	2.04	2.98
	90:10 C	105	4.5	109.5	2159	3.04	4.45
Two cycle compactions	90:10 E	32.2	4.5	36.7	872.5	1.02	1.80
	90:10 F	68.9	4.5	73.4	1373.5	2.04	2.83
	90:10 G	105	4.5	109.5	2150.5	3.04	4.43
Three cycle compactions	90:10 H	32.2	4.5	36.7	843.5	1.02	1.74
	90:10 I	68.9	4.5	73.4	1380	2.04	2.85
	90:10 J	105	4.5	109.5	2097.5	3.04	4.32
Four cycle compactions	90:10 K	32.2	4.5	36.7	937.5	1.02	1.93
	90:10 L	68.9	4.5	73.4	1441	2.04	2.97
	90:10 M	105	4.5	109.5	1066.2	3.04	2.20
Five cycle compactions	90:10 N	32.2	4.5	36.7	768	1.02	1.58
	90:10 O	68.9	4.5	73.4	1444	2.04	2.98
	90:10 P	105	4.5	109.5	1672.5	3.04	3.45