PERFORMANCE OF RED SOIL STABILIZED WITH GRAVEL AND LIME IN CONSTRUCTION OF LOW-VOLUME ROADS IN NYERI COUNTY, KENYA

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Performance of Red Soil Stabilized with Gravel and Lime in Construction of Low-Volume Roads in Nyeri County, Kenya

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DECLARATION

This thesis is my original work and has not been presented for a degree in any other University.

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This thesis has been submitted for examination with our approval as the university supervisors.

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DEDICATION

This work is dedicated to the memory of Mr and Mrs John Hinga Ikenye for their tender love and inspiration as my parents.

Educating the mind without educating the heart is no education – Aristotle

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LIST OF ABBREVIATIONS, ACRONYMS AND SYMBOLS

AASHTO	American Association of State Highway and Transport Officials
ACV	Aggregate Crushing Value
AFCAP	African Community Access Program
AfDB	African Development Bank
AICD	Africa Infrastructure Country Diagnostic
AIV	Aggregate Impact Value
Al_2O_3	Alumina as Aluminium Oxide
ARM	Athi River Mining Cement Limited
ARRB	Australian Road Research Board
ASTM	American Society for Testing and Materials
AustStab	Australian Stabilization Industry Association
BLA	British Lime Association
BSI	British Standards Institution
$CaCO_3$	Limestone as Calcium Carbonate
CAH	Calcium Aluminate Hydrates
CaO	Quicklime as Calcium Oxide
$Ca(OH)_2$	Hydrated Lime as Calcium Hydroxide
CBR	California Bearing Ratio
CEC	Cation Exchange Capacity
CKD	Cement Kiln Dust
CL	High Calcium Lime
cm; cm^3	Centimeter; Cubic centimeter

CO_2	Carbon Dioxide
CSH	Calcium Silicate Hydrates
DoFEP	Department of Finance and Economic Planning
DTA	Differential Thermal Analysis
ESA	Equivalent Standard Axles
g; Mg; kg	Gram; Megagram; Kilogram
gTKP	Global Transport Knowledge Partnership
ICL	Initial Consumption of Lime
ICRAF	International Centre for Research in Agro-Forestry
ILO	International Labor Organization
ISRIC	International Soil Reference and Information Centre
JKUAT	Jomo Kenyatta University of Agriculture and Technology
KIPPRA	Kenya Institute for Public Policy Research and Analysis
km; km^2	Kilometer; Square kilometer
kN	Kilonewton
KNBS	Kenya National Bureau of Statistics
L	Liter
LKD	Lime Kiln Dust
LAA	Los Angeles Abrasion
LVRs	Low Volume Roads
LVSRs	Low Volume Sealed Roads
m; m^3	Meter; Cubic meter
MDGs	Millennium Development Goals

$MgCO_3$	Dolomite as Magnesium Carbonate
MgO	Magnesium Oxide
MPa	Mega Pascals
MoPND	Ministry of Planning and National Development
MoRPW	Ministry of Roads and Public Works
MoT	Ministry of Transport
MoTC	Ministry of Transport and Communications
MoTI	Ministry of Transport and Infrastructure
MoWTC	Ministry of Works, Transport and Communication
NCHRP	National Cooperative Highway Research Program
NLA	National Lime Association
NMT	Non-Motorized Traffic
NNW	North-North West
PI	Plasticity Index
ppm	Parts per million
PVC	Polyvinyl Chloride
RAI	Rural Access Index
RGN	Natural Red Soil and Gravel Admixture
RGT	Lime-treated Red Soil and Gravel Admixture
R2000	Roads 2000 Strategic Plan
SDGs	Sustainable Development Goals
SiO_2	Silica as Silicon Dioxide
S-S	Silica-Sesquioxide ratio

SSA	Sub-Saharan Africa
SSE	South-South East
TRL	Transport Research Laboratories
UCS	Unconfined Compressive Strength
UK	United Kingdom
UN	United Nations
USA	United States of America
USAID	United States Agency for International Development
USCS	Unified Soil Classification System
VPD	Vehicles Per Day
XRD	X-Ray Diffraction
XRF	X-Ray Fluorescence
WRMA	Water Resources Management Authority
μm; mm	Micrometer; Millimeter
%	Percentage

DEFINITION OF TERMS

Basic Access	A minimum level of rural access required as a human right to sustain socio-economic activity.
Blending	A form of soil stabilization that give an acceptable granular mixture out of careful proportioning of at least two soils.
Earth Road	Also known as **engineered natural surface**, it is a road built from the natural soil available along the road.
Natural Gravel	Weathered rock existing naturally as mixture of stone, sand and fines that is used for surfacing and construction of Low Volume Sealed Roads (LVSRs).
Isolation	The lack of **accessibility** (that is, need for travel relative to time and cost) and a major characteristic of poverty.
Land Degradation	The slow but cumulative loss of productivity and ecosystem function of land with long lasting impacts on the people.
Low Volume Roads	Roads carrying up to 300 vehicles per day or less than one million Equivalent Standard Axles (ESA) in their design life.
MDGs	The broad-based Millennium Development Goals, now Sustainable Development Goals (SDGs), that fall under the United Nations (UN) initiatives for improved human welfare.
Rural Access Index	Also known as RAI, it is an economic indicator representing the proportion of rural population within 2 km or walking distance of 20-25 minutes to an all-weather road.
Resilient Modulus	Also known as subgrade modulus that represents stress and strain relationship of a soil, it is a measure of resistance to permanent road deformation under repeated traffic loading.
Soil stabilization	Alteration of some soil properties to create an improved soil material possessing the desired strength and other properties.
Subgrade	Usually the native soil that acts as the foundation to a road pavement and is overlain by the protective **Subbase** and **Base** as the secondary and primary load-spreading layers.
Whole-life costs	A decision-making tool that assesses all cost savings or benefits to the society by a road during its design lifetime.

ABSTRACT

Red clay soil is abundant in Kenya's central highlands but is often regarded a poor road construction material. The objective of this study was to stabilize red clay soil using natural gravel and hydrated lime to generate an efficient, affordable and sustainable material for construction of low volume road in Nyeri County. The soil was blended with natural gravel at 20% increments varying from 0 to 100% to give six soil-gravel admixtures. A predetermined amount of hydrated lime was also added to each of the six admixtures. The soil, gravel and lime were subjected to advanced mineralogical and chemical analytical methods, which showed that red soil and gravel consisted mainly of silica at 40.7 and 50.8% respectively while hydrated lime consisted of 72.5% calcium oxide. Kaolinite was also the predominant clay mineral in red soil. The gravel classified as stone Class A after testing for toughness. Three test specimens were prepared and tested in all cases according to applicable standard testing procedures for particle size, specific gravity, consistency limits, activity, free swell, compaction, and California Bearing Ratio (CBR). Both red soil and natural gravel were well-graded according to particle size and classified as laterite soil and lateritic gravel after silica-sesquioxides ratio analyses. Specific gravity dropped marginally for all soil admixtures with increasing gravel content and with addition of lime. Hydrated lime content varied with gravel content from 4.7 to 2.2% that further decreased consistency limits. Red soil had 74.3 % liquid limit and 30.1% plasticity index, and differed from natural gravel with a margin of 36.4% liquid limit, 22.8% plastic limit, 23.5% plasticity index, and 12.4% linear shrinkage. Free swell index ranged between 0 and 6.3% for all admixtures but with a peak of 12.7% in soil-gravel-lime admixtures. All admixtures compacted fairly well and maximum dry density increased by 744 kg/m^3 while optimum moisture content decreased by 22.4% with increasing gravel content in red soil. Similarly, the corresponding soaked CBR rose by 121.7% and swelling factor dropped marginally by 0.55%. Red soil classified as MH and A-7-5 after Unified Soil Classification System (USCS) and American Association of State Highway and Transport Officials (AASHTO) system, and as subgrade class S3. The respective classification of natural gravel was GW and A-2-7. The study successfully yielded optimal mixes applicable for most paved roads. This comprised about 80% red soil, 16% gravel and 4% lime for improved subgrade, and 32% red soil, 65% gravel and 3% lime for subbase of low volume sealed roads (LVSRs).

CHAPTER ONE

INTRODUCTION

1.1 Background to the Study

For as long as the human race has existed, there has been movement of people and goods (or freight) from one location to another by means of trails, road, rail, water and air. The birth of modern road is unknown since it is lost in what O'Flaherty (1997) calls 'the mists of antiquity'. However, the ancient Romans are credited with building of the first elaborate road system; this fell into decay and disuse after collapse of the vast empire due to lack of technology and will by the local peoples (Holt, 2010; Wignall et al., 1999). Today, the industrialized countries have dense and well developed road networks while those in most developing countries are sparse and least developed. The extent and condition of road networks in developing countries is largely a reflection of their past colonial history and small economic capacity. Additionally, the physical condition of road transport network in a country is critical since this dictates to the level of its national economic development and the standard of living for her population. Nevertheless, road transport dominates in developing countries and carries 80-90 percent of passenger and freight in the Sub-Saharan Africa (SSA) and 93% in Kenya; it also supplements the fragmentary railway system and the geographically hindered inland water transport (Jjuuko et al., 2014; Mwaipungu & Allopi, 2014; Ministry of Transport [MoT], 2009).

There are different categories of roads but Low-Volume Roads (LVRs) comprise the greater part of public roads and account for nearly 30 million kilometers or about 60% of the road network globally, over 70% in the SSA and 80% in Kenya (Kamtchueng et al., 2015; Cook et al., 2013; Greening & O'Neill, 2010; Sogomo, 2010). In effect, LVRs are of low social and geographical reach and include tracks and narrow unpaved rural roads that provide the last link of transport network worldwide (Faiz, 2012; Foster & Briceño-Garmendia, 2010; Johannessen, 2008). LVRs substantially hinder movement of the little and light traffic that they carry but remain the only means of access and lifelines that inevitably sustain large rural population in developing countries, mankind and the world economy (Fukubayashi & Kimura, 2017; Faiz, 2012; Gwilliam et al., 2011). Besides transport, the LVRs uniquely transcend, culture, topography, climate and language to play a critical role in disaster management, peace building and political integration of a country (Faiz, 2012). They are also instrumental in realization of the United Nations (UN) Sustainable Development Goals (SDGs) and national initiatives that guide global and national action on sustainable development. Kenya's socio-economic blueprint known as 'Vision 2030' is a good example that focuses on poverty reduction, rural development, and on a clean and safe environment for a high quality of life to citizens.

In tropical areas like Kenya, over 65% of the population live in rural areas and depend heavily on rural roads for transport and on agriculture for subsistence and economic gain (Cook et al., 2013; MoT, 2009; Lennox & Mackenzie, 2008). In this regard, agricultural development is often the main objective of rural access road improvements. However, conventional road construction depends largely on quarrying of soils as the primary construction materials, but these are scarce non-renewable resources. Additionally, paved roads are expensive to construct and not justifiable on LVRs where good soils are scarce, not available close to the road or depleted (Ejeta et al., 2017; Roughton, 2016; Johannessen, 2008). This is usually the case in mountainous areas where the process of soil stabilization has been developed primarily to economically improve the properties of poor local that replace the difficult to obtain good soils. The process itself may sometimes be least understood but is employed to tap on a wealth of benefits in environment and in material use and cost (Holt, 2010; Wilmot, 2006; Cook et al., 2001; Wignall et al., 1999). Most of Kenya is covered by residual soils which are often red clays that are generally poor as road construction materials (Rolt, 1979; Foss, 1973). It was on this background that the highly mountainous but agriculturally-productive and densely populated Nyeri County on the equator in central Kenya was selected as a representative study area for stabilization of the naturally abundant red clay soil.

1.2 Statement of the Problem

Rural transport networks in most developing countries are sparse, underdeveloped and of poor quality by international standards whereby about 85% of the low-volume roads in the SSA and about 50% in Kenya are estimated to be in poor condition (Kamtchueng et al., 2015; Foster & Briceño-Garmendia, 2010; MoT, 2009). Thus, majority of the rural roads are old unsealed earth and gravel roads that account for about 60% of classified road networks globally, over 90% in the SSA and about 96% in Kenya (Johannessen, 2008; Lebo & Schelling, 2001; Wasike, 2001). This leaves about 3 billion people globally without safe, reliable, and sustainable all-weather access to basic services, markets, and economic opportunities (Faiz, 2012; Gwilliam et al., 2011). According to the Department of Finance and Economic Planning [DoFEP] (2018), Ejeta et al. (2017) and Muturi (2015), the situation is no better in Nyeri County where about 15% of rural roads are paved and 85% are unpaved gravel and earth roads. This situation leaves about 91% of the population in the County with no access to paved all-weather roads (DoFEP, 2013; Elsharief et al., 2013). Worse still, the unpaved rural roads often become impassable during rainy season, as demonstrated in Plate 1.1, leading to great economic loss as the rural population and farm produce cannot reach public utilities and markets, respectively.

Plate 1.1: Bad condition of a rural road in Nyeri County during wet season

(Source: The Star, 13 November 2014)

Provision of paved roads generally remains a mirage in many developing countries because they are expensive to construct and maintain, and hence often difficult to justify on LVRs that give very small economic returns (Roughton, 2016; African Development Bank [AfDB], 2011). Road construction cost has technological and environmental drivers but Johannessen (2008) and O'Flaherty (2002) singled out materials as the principal factor, taking about 70% of total cost especially where suitable local materials are scarce or not in close proximity to the road. Further, most design standards, which are foreign origin, preclude use of local materials considered to be inferior or troublesome in pavement construction (Collier et al., 2013; Cook et al., 2013). Such conditions advocates for use of good borrow materials which is expensive due to hauling over long distances. In Nyeri County, engineers encounter the challenges of weak and inherently variable subgrade provided by the fertile and soft red soil coupled with scarcity of suitable natural gravels to replace the poor soils (DoFEP, 2013; Vorobrieff & Murphy, 2003).

O'Flaherty (2002) quipped that road designers and builders in countries now only experiencing the pleasures and problems associated with the motor age have very great challenges and opportunities! Most developing countries are now in this age where one of the basic and essential challenges for road engineers is to find good and affordable engineering solutions to the provision and maintenance of LVRs (Johannessen, 2008; Petts et al., 2006). Thus, the potential of optimizing use of red clay soil stabilized with natural gravel and hydrated lime in paving of LVRs was explored in this study expected to help overcome this challenge in Nyeri County.

1.3 Research Objectives

1.3.1 General Objective

To evaluate the suitability of stabilizing red clay soil with natural gravel and hydrated lime as a construction material for subgrade and sub-base of low-volume roads in Kenya's Nyeri County.

1.3.2 Specific Objectives

1) To determine the chemical and physical properties of red soil, natural gravel, hydrated lime, and their admixtures.

2) To evaluate the compaction characteristics of red soil-gravel-lime admixtures.
3) To establish the bearing strength of red soil-gravel-lime admixtures as pavement construction materials for lightly trafficked rural roads.

1.4 Research Questions

1) What are the properties of red clay soil, natural gravel, and hydrated lime?
2) What are the compaction characteristics of the red soil-gravel-lime admixtures?
3) What is the bearing strength of compacted soil-gravel-lime admixtures as road as road construction material?

1.5 Justification of the Study

Highway engineers have strived continuously to produce better roads at lower costs, a feat that is successfully achieved by stabilization of locally available soils. This study promoted the efficient use of local red clay soils in an affordable and sustainable construction of LVRs (Cocks et al., 2015; Wilmot, 2006; O'Flaherty, 2002). This comes at a time when natural gravel is fast getting depleted and end to the age of graveling earth roads beckons in Nyeri County and the larger central Kenya (DoFEP, 2018; Cook et al., 2013). As Figure 1.1 illustrates, there has been massive graveling of earth roads to about 81% but at the expense of 15% paved roads. In spite of this commendable achievement, gravel roads worldwide also take an enormous share of available resources to maintain and eventually become more expensive than paved roads (Petts et al., 2006; Rolt, 1979). In effect, all the county governments in central Kenya should brace themselves for the inevitable question asked many road agencies of when to start paving the gravel roads (Faiz et al., 2012; Lennox & Mackenzie, 2008; Skorseth & Selim, 2000). The decision to seal a road is a matter of trade-offs and the county governments should have courage and take an objective look at the long-term benefits of lower whole-life costs of sealed roads (Roughton, 2016; Mwaipungu & Allopi, 2014; Collier et al., 2013; AfDB, 2011; Gwilliam et al., 2011).

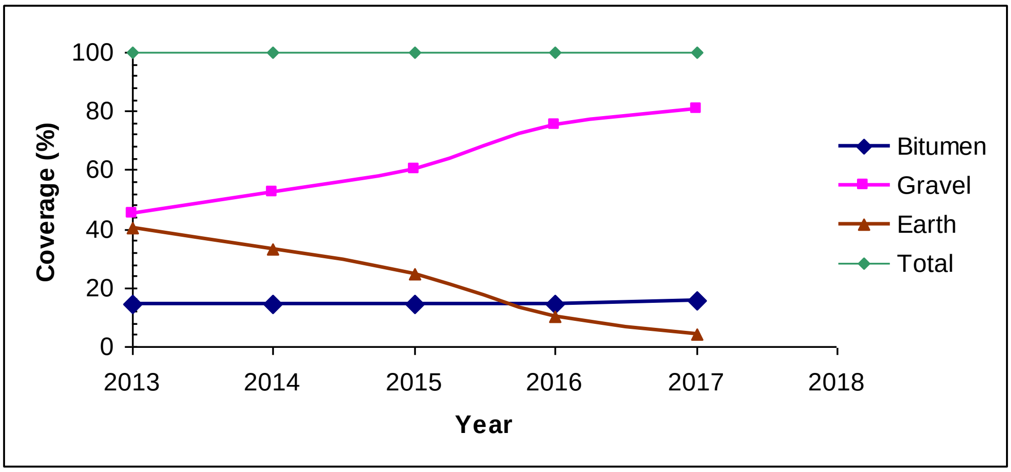

Figure 1.1: Coverage in different types of road surfaces in Nyeri County

(Source: Author based on DoFEP (2018))

Improved LVRs remove obstacles of difficult transport and exceptionally high cost to play the vital role of promoting economic and social life of the large rural populations by. Thus, the roads would translate into economic growth through improved agricultural production and rural development in terms of cottage industries (DoFEP, 2013; Faiz, 2012; Gidigasu, 1991). It is for these reasons that LVR improvement programs are prioritized, for maximum rural access and development impact from limited road funds, in line with high population densities, value of agricultural land and spatial proximity to urban markets (AfDB, 2011; Foster & Briceño-Garmendia, 2010; Riverson et al., 1992). Raising of the Rural Access Index (RAI) to 100% from the current national level of 47% in highest-value agricultural areas like Nyeri County is also advocated for. Moreover, a country requires a minimum 80% RAI for her economic take-off.

Most unpaved LVRs are in poor condition and seasonally impassable, leaving about 91% of the population in Nyeri County without all-weather access (DoFEP, 2013; Pinard & Greening, 2004; Lebo & Schelling, 2001). Without good all-weather LVRs, economic potential is not only inhibited but most rural areas become isolated and rural life stagnates (Gwilliam et al., 2011; Greening & O'Neill, 2010). In the contrary, improved LVRs will reverse these socio-economic challenges by opening up all isolated and economically-lagging areas. This will also foster rural connectivity vital for access to socio-economic services like health, education, amenities and markets (Cook et al., 2013; Faiz, 2012; Petts et al., 2006). The roads will also remove social isolation essential for poverty reduction as envisaged by the universal Millenium Development Goals (MDGs), now SDGs. To attain substantial poverty reduction, and hence achieve all the MDGs, in lowering high levels of hunger, disease, illiteracy and unemployment, a country should have a minimum 7% national economic growth (Foster & Briceño-Garmendia, 2010).

According to DoFEP (2013), Nyeri County is endowed with natural resources where 2.3 million tones of murram/gravel are mined per year for road construction and maintenance. However, the widespread use of gravel in unpaved roads pose health, safety and environmental hazards to natural habitats adjacent to the road corridors (Ejeta et al., 2017; Jawad et al., 2014; Lennox & Mackenzie, 2008). As envisaged in Kenya's Roads 2000 Strategic Plan [R2000], this study aimed at ensuring optimum utilization of the locally available red soil and gravel in construction of stronger LVR pavements capable of carrying the ever increasing size, load and number of modern vehicles (Fukubayashi & Kimura, 2017; Cocks et al., 2015; Greening & O'Neill, 2010; Wilmot, 2006). This would save on the environment, on public funding and even foreign exchange and hence the urgency to provide efficient, affordable and sustainable LVR network in Nyeri County.

1.6 Scope and Limitations of Research

1.6.1 Scope

This study investigated the improvement of red soil using natural gravel and hydrated lime. The red soil and natural gravel were collected from Othaya and Nyaribo in Nyeri County, some 13 km and 10km by road to the south and north of Nyeri Town respectively. A commercial grade hydrated lime suitable for soil stabilization, as recommended by the manufacturer, was obtained from a stockist over the counter. Soils in general are completely different in different locations and hence the red clay soil and natural gravel from the Kenya Highlands were taken to be representative of similar materials found elsewhere. The hydrated lime was also expected to meet universal manufacturing standards.

As an experimental study, select laboratory tests were conducted to determine properties of the red soil, natural gravel and hydrated lime, and also as a measure of their combined soundness and functionality in soil stabilization. Secondary data was also gathered for the purpose of relating laboratory results with the existing body of knowledge. Most of the data was used to establish the optimal stabilized mixes for improved subgrade and subbase of an LVR pavement.

1.6.2 Limitations

The published laboratory data was limited to the research materials but the findings could be applicable to other materials of similar properties. Further, the study was limited to construction of a sealed LVR but the results could be of benefit not only to engineered earth and gravel roads but also to paved roads of a higher category. However, the soundness of these materials was limited to short soaking periods of 4 and 7 days without prolonged exposure to the weather elements.

CHAPTER TWO

LITERATURE REVIEW

2.1 Hierarchy and Structure of Road Pavements

i) Hierarchy

According to Wasike (2001), roads are usually put into three functional classes, namely:

i) Primary roads for mobility and serve as the main national linkages to capital city, airports, the sea, and other countries,

ii) Secondary roads for both mobility and access, connecting regions within the country as departmental, provincial or regional roads, and

iii) Tertiary roads mainly for access and connect towns within one province or region as municipal/urban or local/rural roads. Figure 2.1 summarizes the attributes of different classes of roads.

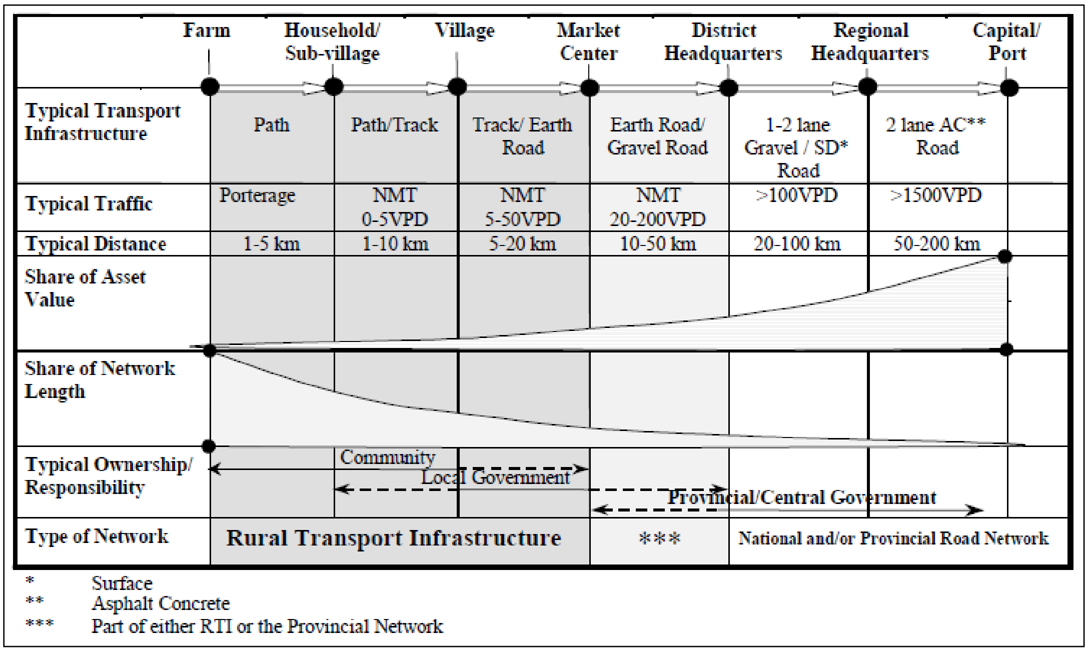

Figure 2.1: Characteristics of different classes of roads

(Source: Lebo and Schelling, 2001)

This study focused on rural roads which are also variously known as district/county roads, local government roads, feeder roads and access roads (Nwachukwu, 2013;

Lebo & Schelling, 2001). As roads of low value, they are designed with very little financial and scientific input, and are constructed from the nearest available materials as possible whereas their condition can be significantly affected by a period of excessive traffic volumes or inclement weather (Ejeta et al, 2017). In most developing countries, the access roads include old earth roads or partly engineered earth and gravel roads where many of them are of poor quality and cannot carry the modern vehicular traffic since (MoT, 2009; Wasike, 2001; Gianfrancisco & Jenkins, 2000). Some of the roads also consist of an elevated and very rough riding surface, side drains and cross-drainage structures like culverts and bridge. According to Rolt (1979), most of the roads usually have problems in rutting, corrugations, erosion, dust and alignment that create safety and traffickability hazards to users. In addition, the effective strength of an unsealed pavement varies daily with moisture content that fluctuates, in turn, with the rainfall and dry periods. Thus, rutting depend critically on the type of soil and weather. In this regard, access roads built on bare clay soils have fairly deep ruts that make the roads impassable during wet weather. Some rutting also form in gravelled roads built on clay soils.

ii) Structure

A typical flexible or bituminous road pavement consists of several layers consisting of a wearing surface, base course, and subbase built over compacted subgrade (or natural soil), as shown in Figure 2.2. According to Robinson and Thagesen (2004), O'Flaherty (2002) and Cook et al. (2001), these layers generally carry and spread the self and imposed traffic load in a manner that they shall not deteriorate to any serious extent within the design life. Nonetheless, each layer plays a critical role in the performance of the pavement. The wearing course of a completed pavement protects the underlying layers from the effects of weather and also provides a riding surface to vehicles. The base and subbase are the respective primary and secondary layers that spread the traffic load safely over the underlying subgrade as the actual foundation. In some limited cases, however, the subbase layer may not used. Also, where the subgrade is an inherently weak soil, the material is typically removed and replaced with a stronger granular material (Holt, 2010).

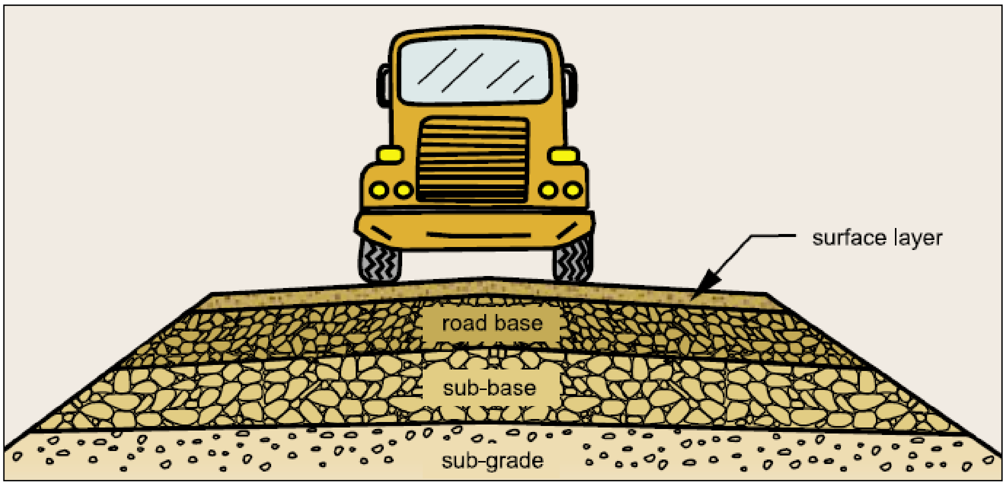

Figure 2.2: Typical section through a road pavement

(Source: Johannessen, 2008)

Paved roads are constructed from a wide variety of materials and mixtures that consist of conventional binders like lime, cement and bitumen together with local materials like soil, gravel and stone (Budhu, 2011; Johannessen, 2008; Cook et al., 2001). The wearing surface is made of high quality crushed stone or aggregates mixed with a binder, usually bitumen. The underlying layers consist primarily of treated and untreated local gravels. The base is constructed mainly of treated gravels for increased strength while the subbase is mostly untreated local gravel. Moreover, the top of the subgrade is sometimes stabilized with either cement or lime, though cement gives a material with a higher tendency to cracking (Hudson, 1997). Thus, greater attention is now being paid to the use of substitute materials such as stabilized soils to meet road construction needs especially for the upper layers (O'Flaherty, 2002; Cook et al., 2001).

The successful construction of roads requires a structure that is capable of carrying the imposed traffic loads safely (Holt, 2010). This is because life of a road depends on strength of the subgrade soil and traffic density. Thus, the road structure is almost impossible to design and maintain even with best materials when it is overloaded or the traffic increases in number (Gianfrancisco & Jenkins, 2000). Accordingly, Nwachukwu (2013) postulated that Low Volume Sealed Roads (LVSRs) need to be effectively and efficiently planned, designed, built, upgraded continuously and preserved by means of integrated policies that respect the environment and still provide the expected socio-economic services. It is therefore becoming recognized that a key objective in LVSRs is to best match the locally available material to its road task and its local environment (Cook et al., 2001).

2.2 The Study Area

2.2.1 Location, Topography and Climate

Nyeri County is situated in Kenya's central highlands with its headquarters at Nyeri town, some 150km by road north of Nairobi. The county borders Laikipia to the north, Meru to the north-east, Kirinyaga to the east, Murang'a to the south, and Nyandarua to the west. It lies between the equator and latitude 0°38'45" south, and between longitudes 36°35'22" and 37°18'29" east. It has a total land area of 3,337 km^2 and lies at an altitude of between 1210 m and 5199 m above sea level suggesting a markedly great topographic variability. Figures 2.3 and 2.4 show the location, topographical and administrative maps of Nyeri County, respectively.

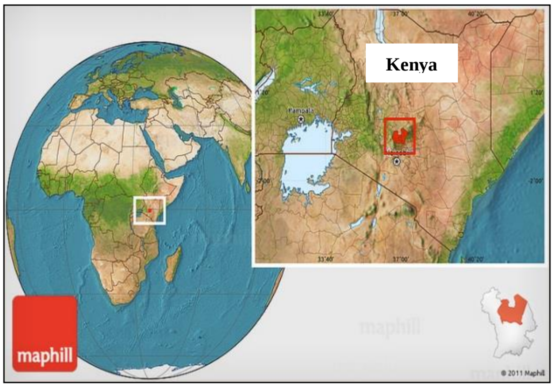

Figure 2.3: Location of Nyeri County on the global, regional and national map

(Source: Maphill, 2011)

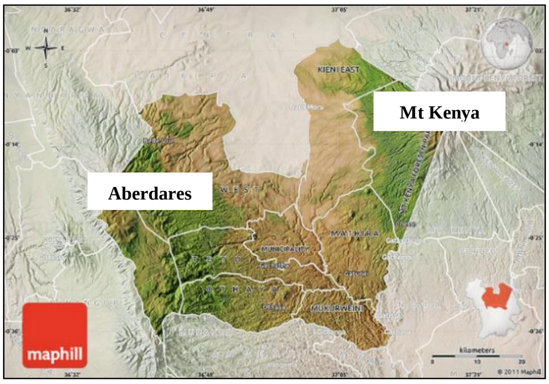

Figure 2.4: Topographic map of Nyeri County with administrative boundaries

(Source: Maphill, 2011)

The county sits largely in the saddle between Mt. Kenya (5199m) to the east and the Aberdare Ranges (3999m) to the west. These volcanic mountains however give way to a central landscape marked by many small isolated and rounded hills like

Tumutumu and Nyeri Hills seen as remnants of old volcanic vents (Muturi, 2015; DoFEP, 2013; MoPND, 2005; Shackleton, 1945). The landscape along the Aberdares is characterized by an undulating volcanic topography with a general easterly slope which is deeply dissected by numerous, closely spaced and almost parallel steep ridges and river valleys. Conversely, the area about Mt Kenya has a gently rolling plain traversed by shallow and rather featureless valleys to the south, and an extensive grass-covered plateau of southern Laikipia to the west. The central landscape is drained by the Tana and Uaso Nyiro river basins with many permanent rivers like Chania and Sagana with an easterly flow, and Naromoru with a predominantly northerly flow (Jaetzold et al., 2006; Baker, 1967; Fairburn, 1966; Thompson, 1964). Plate 2.1 presents the elevation map of Nyeri County as viewed from the south showing the two volcanic mountains that influence its geology, relief, climate, human settlement, economy, and transport networks.

Plate 2.1: Elevation map of Nyeri County viewed from the south
(Source: Maphill, 2011)

According to DoFEP (2013) and Muturi (2015), Nyeri County has conditions of a mountainous and wet area that experiences moderate temperatures with a monthly mean of between 12.8°C and 20.8°C. The county also receives strongly seasonal relief rainfall that occurs mostly at changes of the monsoon in two separate rain seasons that come as the long rains from March to May and as the short rains from October to December. However, the Laikipia Plateau has a trimodal rainfall pattern, with middle rains in July-August intruding from the west (Jaetzold et al., 2006). Accordingly, rainfall ranges from 700 mm in the Laikipia Plateau to 2200 mm in the higher volcanic country. The areas over 1500 m above sea level also experience frequent mists and drizzle caused by the south-easterly trade winds forced up the mountains in the months June-September. Conversely, the whole county experiences a dry season in January-February due to the dry NE Trades from the Somalia deserts that blow over the region (Muturi, 2015; MoPND, 2005).

2.2.2 Geology

According to Budhu (2011), geology is important to a geotechnical engineer for successful understanding of the character of the rocks and soils at a place. The geology and soil formation is influenced by such environmental factors like topography, climate and drainage conditions (Northmore et al., 1992a). Geological information was extracted from relevant old reports. Hinga et al. (2019), Baker (1967), Fairburn (1966) and Shackleton (1945) reported that geology of the larger Kenya Highlands is grounded on the Basement System, a crustal block of the Precambrian or Archaean age. It was transformed from the oldest marine sedimentary rocks and it structurally consists of highly folded and crystalline metamorphic rocks like gneisses and schists. The crustal block has a SSE-NNW general strike and dips 20°-40° dominantly to the west. The Basement System is best represented by a belt of exposed metamorphic hills and ridges to the south east, near Sagana Township. In Kenya east of the Rift Valley, the crustal block provides a planed erosion surface of the sub-Miocene age called peneplain (Bruggemann & Gosden, 2004; Attewill & Morey, 1994). However, the peneplain is largely missing in Nyeri area where a highly irregular pre-volcanic floor is evident instead.

The Basement System is overlain by volcanic rocks where the gneiss hills served as a continuous barrier to the lava flows and form a buried sub-volcanic ridge in the Tumutumu-Kiganjo area; the hills also influences the subterranean drainage pattern of the area (Fairburn, 1966; Thompson, 1964; Terzaghi, 1958; Shackleton, 1945). According to Bruggemann and Gosden (2004), lavas represent the height of volcanic activity with eruptions occurring from localized vents as seen in section 2.3.1. Thus, two intermittent periods of intense volcanic activity are picked up around Mt Kenya-Aberdares region where the Sattima Fault is the only major structural feature east of the Rift Valley. The volcanics thicken westwards and northwards of the metamorphic hills towards the Aberdares and Mt Kenya but thin out to the east of the hills that effectively mark the western limit of exposed Basement System east of the Rift Valley. The volcanics comprise the oldest Simbara (or northern Aberdare) Series of

Tertiary age, the Mt Kenya Suite of Pleistocene age, and the younger Laikipian Series of Pliocene age. The Simbara Series dips to the southeast but thin out northwards and include basalts, agglomerates and trachytes. The Mt Kenya Suite comprises the phonolitic agglomerates and trachytic tuffs like the Nyeri tuffs. The Laikipian Series are associated with the numerous volcanic vents on the plateau and comprise olivine basalts that dip to the southeast.

Finally, the volcanic rocks are overlain by recent soil deposits of Quaternary age. These comprise the red soils in the highlands and the pink soils along the gneiss hills, the black soils, red lateritic earths, natural gravels and the allophanic Naromoru ash (soil) in the lowlands west of Mt Kenya. The name 'Nyeri' reportedly originates from the Maasai word 'nyiro' for the dominant red soil. The natural gravels were found to overlie the Nyeri tuffs in two distinctly colored layers – grey and bluish – quarried for building stone. The geology, topography and climate of the area also generated some soils and rocks used as road construction materials. The red soil and natural gravel for this study were also obtained locally. Figures 2.5 and 2.6 shows the associated lithological and soil maps of Nyeri County respectively.

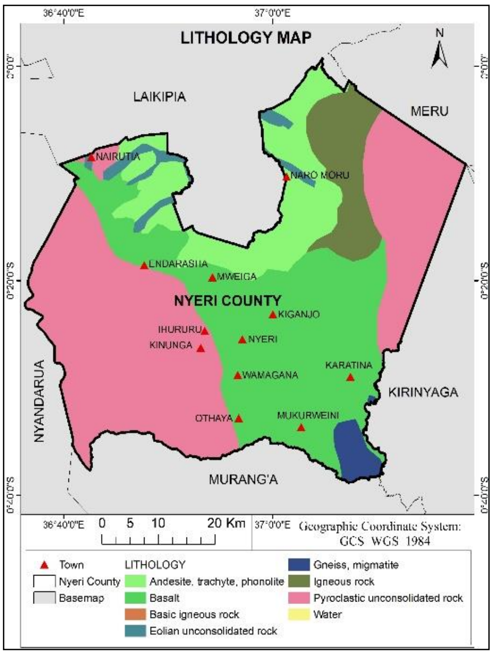

Figure 2.5: Geological map of Nyeri County

Source: GCS, 1984)

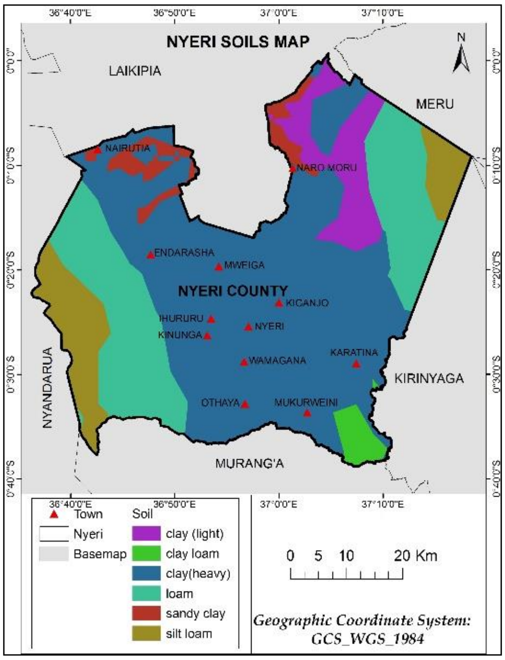

Figure 2.6: Soil map of Nyeri County

(Source: GCS, 1984)

2.2.3 Settlement, Land Use and Natural Resources

The topography, climate and geology of Nyeri County further determine the human settlement patterns, land use and the distribution of transport network. Human settlement and agriculture are highly influenced by natural land fertility and adequate rainfall (DoFEP, 2013). As a result, the Kenya National Bureau of Statistics [KNBS] (2019) demonstrated that the majority of population is found in the humid highlands and minority in semi-arid lowlands that correspond to high and low agricultural potential respectively. The county recoded an average population density of 228 persons per square kilometer but this varied greatly between the high- and low-potential areas that took about 74% and 26% of the total county population.

According to DoFEP (2013), Knoop et al. (2012) and Baker (1967), arable land take about 30% of the total surface area and much of it is used for subsistence and commercial farming, livestock rearing and agro-forestry. In the highlands, the land is used intensively for food crops and cash crops like tea and coffee as demonstrated in Plate 2.2. However, subsistence farming mixed with large-scale farming for horticulture, wheat, and livestock rearing is predominant in the expansive and semi-arid lowlands though water scarcity limits the potential to establish ranches.

Plate 2.2: Intensive land use near Karatina Town in Nyeri County
(Source: International Soil Reference and Information Centre [ISRIC], 2010)

The county also benefits from a forest cover of 10.6% mainly from indigenous and plantation trees, compared to the national forest cover of 6.4%, as it serves as an extensive natural carbon sink and as a source of timber and wood fuel for domestic and industrial use. It is also self-sufficient in surface and sub-surface water for domestic, agriculture and industrial use. However, building stones, gravel and kaolin clay for ceramics are the only economic natural deposits of the county. Additionally, the high population density and construction activities adversely affect the use of natural resources like land, soils and water. For instance, Figure 2.7 illustrates a markedly continued decline in water per capita in the larger Tana Basin.

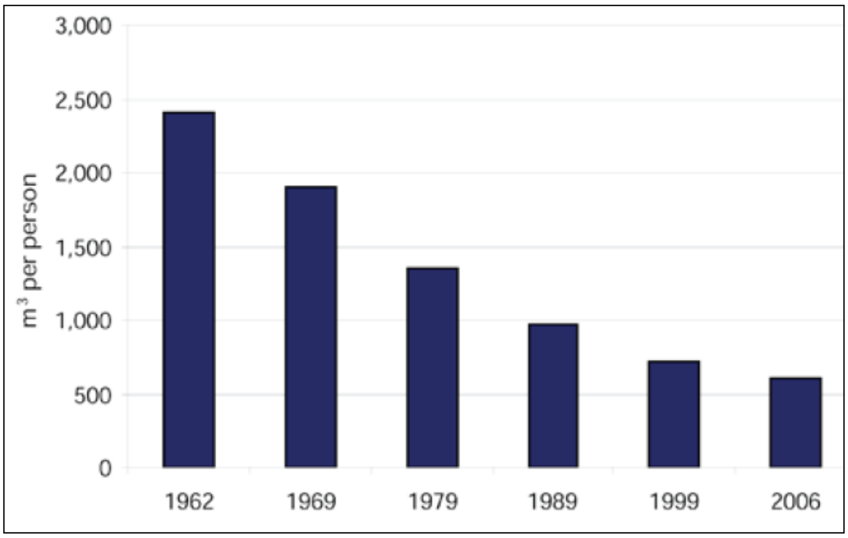

Figure 2.7: Water availability per capita in Tana Basin, Kenya

(Source: Knoop et al., 2012)

2.2.4 Economy and Transport

According to DoFEP (2018, 2013), Muturi (2015), and Jaetzold et al. (2006), there is no potential for large-scale mining activities in Nyeri County but there are small mining of natural gravel, building stone, aggregates, sand and kaolin clay; the annual production of gravel and kaolin is the highest and lowest at 2.3 million and 2,560 tonnes respectively. For this reason, agriculture is the backbone of the local economy with 53% of the population being involved in agricultural production. However, the type of agriculture practiced in an area, and hence its economy, depends on the type of soil and climate. Moreover, there are a number of manufacturing industries in Nyeri County mainly engaged in agro-processing. The tourism industry also thrives especially within Mt. Kenya and the Aberdare ecosystems of the county where game, site-seeing, and sporting are the major tourist attractions.

A good and adequate road network is however vital for efficient transport of people and the highly perishable agricultural produce like tea, milk and horticultural products and hence sustain economic growth in the county. Railway transport is the second most important mode of transport in Kenya with a total of 2,778 km of railway network (MoT, 2009). Nyeri County is served by 3,093 km of classified roads, a derelict railway line, and three operational airstrips. The roads are confined mainly to the ridges in line with the undulating terrain and drainage pattern that raise the cost of opening up of new roads (DoFEP, 2013; Baker, 1967). In this regard, only 24% of the classified roads are paved and the rest are unsealed gravel and earth roads that often become impassable during the rainy season (DoFEP, 2013). Muturi (2015) therefore advocated for improved condition of road network as a deliberate effort to initiate economic growth and social development in the county.

2.3 Empirical Review on Properties of Red Soil, Natural Gravel and Lime

2.3.1 Red Soil

Red clay soils are tropical residual soils usually predominant in young, sloping and well-drained volcanic highlands and mainly a product of chemical weathering which is abundant in hot and humid climate (Xue et al., 2020; Elsharief et al., 2013; Keter & Ahn, 1986). According to Wesley (2009) and Coleman et al. (1964), red soils are formed mostly, but not always, over basic volcanic rocks like basalt. They are one of the most fertile and widely distributed group of soils in the world that support dense populations (Bommer et al., 2002; Northmore et al., 1992a; Rolt, 1979; Foss, 1973). The volcanic soils are common in parts of Central and South America, Africa and South East Asia, as shown in Figure 2.8. Red clay soil is locally known as ‘red coffee soil’ and is predominant in the central Kenya Highlands including Nyeri County (Hinga et al., 2019; Waweru et al., 1998; Gichaga et al., 1987).

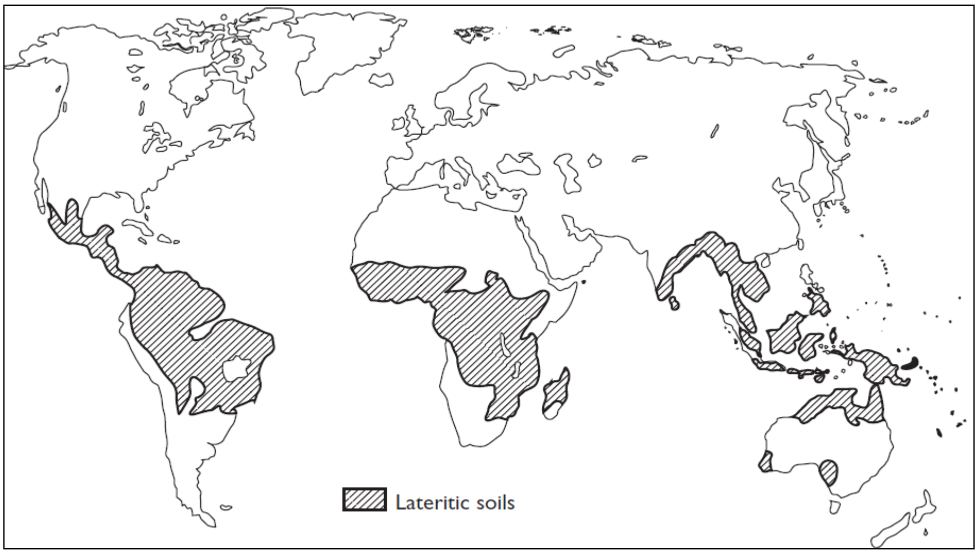

Figure 2.8: Global distribution of red soils

(Source: Robinson and Thagesen, 2004)

In natural state, red clay soil is about 1.5 to 20 metres deep, is well-drained due to a highly porous structure and a granular appearance and have a dry density of 800-1300 kg/m^3 (Waweru et al., 1998; Dixon & Robertson, 1970; Foss, 1973). The soils are fairly soft but break easily after some time of exposure to weather; they also exhibit large cohesion and numerous joints as drying cracks. Moreover, the soils have strong adsorption capacity, are capable of holding large quantities of water and exist at high natural moisture content and degree of saturation subject to large seasonal variations but still remain considerably strong and stable (Xue et al., 2020; Chen & Lu, 2015; Rolt, 1979). The red clay soils consist of very high clay content of 30-80% and up to 30% silt whereby they often classify as silts (Keter & Ahn, 1986; Coleman et al., 1964; Terzaghi, 1958). As a result, red clay soils have abnormally high consistency limits, high shear strength, and high resistance to accelerated soil erosion. The void ratio of 1.2-2.5 of the soil suggests clay of low compressibility whose strength varies with porosity and moisture conditions. The soils are also difficult to compact and this results in abnormally low maximum dry density (MDD) and high optimum moisture content (OMC).

Red clay soils experience a strong mineralogical influence in their behavior, and consist principally of halloysite or kaolinite silicate clay minerals (Bruggemann & Gosden, 2004; Northmore et al., 1992b; Newill, 1961). They may also contain nonsilicate sesquioxides gibbsite and goethite, the hydrated forms of aluminium and iron oxide more common in older volcanic soils and which act as cementing agents. Allophane, an amorphous clay mineral formed from rapid weathering and alteration of volcanic 'glass', is a good example of halloysitic soil. Hydrated halloysite forms in wet areas but converts into the anhydrous metahalloysite upon its desiccation or loss of water (Wesley, 2010; 2009; Coleman et al., 1964). Metahalloysite can be difficult to distinguish from kaolinite as it is poor-crystalline kaolin mineral.

Red color of the soil is attributed to the presence of hematite as amorphous or free iron oxide in a thin porous coating to soil particles (Xue et al., 2020; Sherwood, 1967; Coleman et al., 1964). The soils are also strongly aggregated into clusters by the cementing properties of halloysite and hematite which results in a relatively coarse-grained appearance, friable (powdery) texture and a crumb structure. The presence of allophone also enables majority of red soils to exist at abnormally very high natural moisture content, usually above optimum. In addition, red soils containing allophane, halloysite and gibbsite have a marked sensitivity to drying and the physical properties are highly susceptible to irreversible changes even when partially dried (Bruggemann & Gosden, 2004; Northmore et al., 1992b).

Many volcanic soils including the red clay soils are highly desirable for many engineering uses. According to Xue et al. (2020) and Wesley (2010; 2009), the soils have good engineering properties when hydrated halloysite is the predominant mineral. However, the excellent physical properties are highly unusual when compared with similar soils of temperate climate. Thus, the unusual characteristics of red clay soils can be a source of considerable puzzlement to engineers encountering them for the first time, and even remain a major source of concern to road engineers who are to reluctant to accept this behavior (Elsharief et al., 2013; Wesley, 2009; Rolt, 1979). For instance, many engineers are faced with inability to adequately classify the red clay soils and mistakenly call any red tropical residual soils 'laterite'. Others find the red clay soils particularly troublesome to handle, especially those containing halloysite, and also mistake the soils for a good foundation soil in buildings or the (Kamtchueng et al., 2015; Northmore et al., 1992a; Gichaga et al., 1987). Thus, a higher risk of failure and a short design life are generally adopted in road works for economic reasons; a conservative allowable bearing pressure of 80 kN/m^2 is also recommended in building foundations. Nonetheless, the difficulties encountered in assessing and handling of the soils has been linked to a lack of understanding of their nature, yet they are relatively good and stable as subgrade (Elsharief et al., 2013; Smart, 1973; Dixon & Robertson, 1970).

Coleman et al. (1964) and Smart (1973) summarized the structure as well as chemical and mineralogical composition of the Nyeri red clay, as outlined in Table 2.1. They reported that most of the soil consisted of brown amorphous material some of which occurred as rounded aggregations. These writers and Terzaghi (1958) added that the soil consisted principally of metahalloysite, silica as ordinary quartz, and ferric oxide as hematite together with small quantities of a mineral of the feldspar type and possibly some goethite. Rossiter (2004) proposed a geochemical index represented by the silica-sesquioxide (S-S) ratio of a soil and expressed as ($SiO_2/(Fe_2O_3+Al_2O_3)$). The index defines the degree of laterization (that is, advanced stage of chemical weathering) of tropical aluminosilicate rocks into soil. When applied to the Nyeri red clay as studied by Coleman et al. (1964), the S-S ratio is 0.96 (<1.33) which classifies the soil as a highly weathered or laterite soil.

Table 2.1: Chemical and mineralogical composition of Nyeri red clay soil

Element[a]		Mean Percentage	Mineral[b]	Percentage
Silica	(SiO_2)	40.8	Metahalloysite	50
Ferric oxide	(Fe_2O_3)	22.7	Iron oxide	23
Alumina	(Al_2O_3)	19.6	Quartz & Feldspar	2-3
Ignition loss		12.6	Muscovite	Trace
Titanium	(TiO_2)	1.9	Hornblende	Trace
Potassium	(K_2O)	1.0	Magnetite or Ilmenite	Trace
Sodium	(Na_2O)	0.6	Hematite or Limonite	Trace
Magnesium	(MgO)	0.1	Leucoxene	Trace
Manganese	(Mn_3O_4)	<0.01		
Calcium	(CaO)	<0.01		
Sulphate	(SO_3)	<0.01		
Total		99.3		
Free Ferric Oxide (Fe_2O_3)		10.2		
Organic matter		1.7		

(Source: Coleman et al., 1964[a]; Smart, 1973[b])

2.3.2 Natural Gravel

Gravel is a naturally occurring mineral material that consists of partially weathered rock that is often excavated and used in road construction in relatively unprocessed form in flexible road pavements. According to Robinson and Thagesen (2004), and O'Flaherty (2002), rocks are generally classified into three main groups based on their method of origin, namely the igneous rocks, the sedimentary rocks, and the metamorphic rocks. The igneous rocks are the most common and they are formed by cooling and solidification of hot molten rock material called magma. They are further identified and classified into two main groups according to their manner of formation and texture as:

1) Extrusive or volcanic rocks that include loose pyroclastic materials ejected from volcanic vents with a fine-grained and often glassy or vitreous structure as a result of rapid cooling of magma on the surface of the earth, and
2) Intrusive or plutonic rocks with coarse-grained and entirely visible crystalline texture resulting from very slow cooling of large volumes of magma at great depths within the earth's crust; hypabyssal rocks are formed nearer to the surface of the earth in small cavities and cracks as medium-grained rocks or pophyries.

The igneous rocks are further described geologically as aphanitic, pophyritic and phaneritic rocks if they are fine-grained, medium-grained and coarse-grained in texture, respectively. Robinson and Thagesen (2004), and O'Flaherty (2002) added that the color and classification of igneous rocks is related to silica content. Thus, the rocks are described as ultramafic, mafic, intermediate and felsic as silica content increases from about 30-80% and the rocks become lighter in color. Similarly, the rocks are classified as ultrabasic, basic, intermediate and acidic as silica content increases over the same range. The color and chemical classification of igneous rocks is summarized in Figure 2.9 where other constituent minerals are determined by drawing a vertical line from the known silica content and then reading off their relative proportions at the arbitrary boundaries.

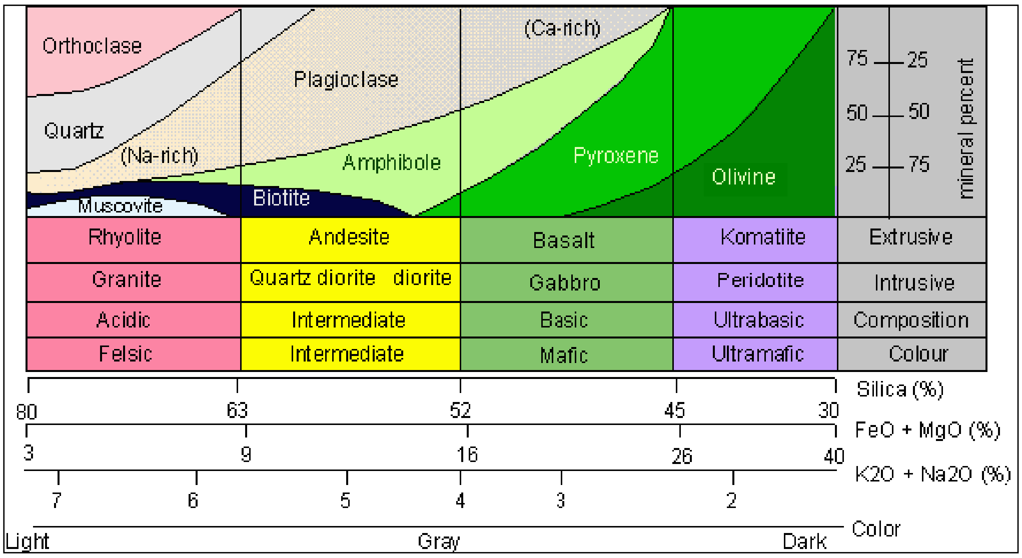

Figure 2.9: Generalized composition of common igneous rocks

(Source: http://earthsci.org/mineral/rockmin/igneous/igneous.html)

Robinson and Thagesen (2004), and O'Flaherty (2002) reported that igneous rocks in general make good road aggregates but pyroclastic materials like tuffs and agglomerates are normally porous and of low strength and usually not satisfactory as construction aggregate. They added that the fine-grained and medium-grained types have better abrasion and impact values though the fine-grained types have poorer polished stone values than coarse-grained types. They also gave dolerite, basalt and granite as the best igneous road stones but suggested that the rocks should be subject to mineralogical investigation in wet tropical regions, unless the quality of the rock is known from previous experience.

As shown in Table 2.2, Thompson (1964) and Baker (1967) outlined the chemical composition of olivine basalt and phonolite belonging to the Simbara Series and Mt Kenya Suite respectively, and obtained from Nyeri. The natural gravel was obtained as partially weathered rock at the southern edge of the Laikipia Plateau and, from field observations, overlay the Nyeri tuffs. The gravel was therefore expected to be Laikipian basalt and close in chemical composition to the olivine basalt.

Table 2.2: Chemical composition of two volcanic rocks from Nyeri

Determination		Olivine Basalt [a]	Phonolite [b]
Silica	(SiO_2)	46.58	52.10
Alumina	(Al_2O_3)	13.67	22.29
Ferric oxide	(Fe_2O_3)	6.15	1.73
Ferrous oxide	(FeO)	6.37	4.10
Magnesium	(MgO)	7.66	1.17
Calcium	(CaO)	11.20	2.42
Sodium	(Na_2O)	2.55	8.60
Potassium	(K_2O)	1.30	4.66
Water	(H_2O^+)	1.70	0.75
Water	(H_2O^-)	0.46	1.00
Titanium	(TiO_2)	2.66	0.3
Phosphorous	(P_2O_5)	0.25	0.46
Manganese	(Mn_3O_4)	Trace	0.23
Total		100.55	99.81

(Source: Thompson, 1964[a]; Baker, 1967[b])

According to McNally (1998), Petts et al. (2006), Gianfrancisco and Jenkins (2000), and Skorseth and Selim (2000), the term 'gravel' is used in the general sense for convenience since gravels are actually gravel-sand-silt mixtures with a small proportion of clay binder that occur in few natural deposits. They added that good gravel is normally scarce in some regions of the world but it is usually worked from the face of dry pits after stripping of the overburden soil. Thus, the quality of gravel varies substantially with location of each borrow pit and within a borrow pit over time in depth of extraction. In this respect, great care is required at source to make them free from vegetation, topsoil, marginal material, segregation, and any over-sized hard material. Thus, quality gravel should comply with grading and plasticity requirements for high mechanical strength and stability to resist breakdown and movement under the effects of traffic loads and weather, but few natural deposits have an ideal gradation without appropriate processing (Johannessen, 2008; Cook et al., 2001). Natural gravels are varied and widespread deposits whose characteristics may be difficult to generalize but they are excessively coarse, porous and moisture-sensitive (McNally, 1998). However, the respective specifications for quality gavel are presented in Tables 2.3 and 2.4 for grading and plasticity guidelines as provided by MoTC (1987) and Intech Associates (2002).

Table 2.3: Natural gravel guidelines for subbase and base

BS Sieve Size (mm)	Nominal Size and Percent Passing (%)		
	Subbase		Base
	60 mm	40 mm	50 mm
75	100	-	
63	95-100	-	-
50	85-100	100	100
37.5	75-95	90-100	95-100
28	60-87	75-95	80-100
20	50-80	60-90	60-100
10	30-67	35-75	35-90
5	23-58	25-63	20-75
2	13-40	15-45	12-50
1	7-32	8-35	10-40
0.425	4-20	4-26	7-33
0.075	0-10	0-12	4-20

(Source: MoTC, 1987)

Table 2.4: Guidelines for gravel plasticity characteristics

Climate	Liquid limit (%)	Plasticity Index (%)	Linear Shrinkage (%)
Moist and Wet Tropical	<35	4-9	2-5
Seasonally Wet Tropical	<45	6-20	3-10
Arid and Semi-arid	<55	15-30	8-15

(Source: Intech Associates, 2002)

McNally (1998) further posited that natural gravels are the most widely used and most economical road-making materials in the world because they are the locally available and incur minimal haulage charges, they require only rudimentary in-pit or on-road processing, and they are flexible and can be worked by simple equipment. Nonetheless, some deposits are shallow and broad leading to significant environmental impact due to its extensive lateral exploitation. Moreover, most pavement material specifications usually regard gravel to be of marginal quality but which might at best be acceptable as sub-base for lightly trafficked rural or local roads. In this regard, quality gravel should generally be composed of virtually all fractured and angular particles, and not rounded particle shapes, for good particle interlock. As a guideline, the gravel should simply consists of a mixture of 35-65% stone for strength, 20-40% sand to fill the voids between stones, give stability and maintain high drainage characteristics, and 10-25% clay or plastic fines to bind the particles together (O'Flaherty, 2002; Skorseth & Selim, 2000). The removal or breakdown of oversize material and sufficient compaction are also considered essential for optimum performance (Paige-Green, 2007).

2.3.3 Lime

2.3.3.1 Manufacture of Lime

Lime is a versatile alkaline chemical used extensively in production of food, paper, pharmaceuticals and construction materials like steel, glass, rubber, leather, paints, and plastics. Its use in construction span over several millennia but it is mainly used today in the stabilization of roads, airfields, building foundations and earth dams (O'Flaherty, 2002; Little, 1995). It is also widely used in the treatment of drinking water and wastewater, treatment of industrial effluent, and in the remediation of contaminated land, and in neutralization of gases generated from power stations for environmental purposes (British Lime Association [BLA], 2010).

The manufacture of lime involves the heating of natural limestone ($CaCO_3$), usually in giant shaft or rotary kilns at atmospheric pressure and temperatures of 950°C, to produce quicklime (Australian Stabilization Industry Association [AustStab], 2002; Little, 1995). The process of burning crushed limestone is known as calcination. Here, the high temperatures lead to evaporation of water in the stone, and to chemical dissociation of limestone into calcium oxide (CaO) and carbon dioxide (CO_2). O'Flaherty (2002) added that when pure or almost pure limestone is calcined, it produces calcitic or high-calcium quicklime. Similarly, dolomitic limestone produces dolomitic quicklime which is a mixture of CaO and magnesium oxide (MgO). After calcining, quicklime is cooled, crushed and stored for shipment in bulk or bags. The production of quicklime is chemically expressed as follows:

$$\text{Limestone} + \text{Heat} = \text{Quicklime} + \text{Carbon Dioxide}$$

$$CaCO_3 + Heat = CaO + CO_2 \text{ (for calcitic quicklime), and}$$

$$CaCO_3{\cdot}MgCO_3 + Heat = CaO{\cdot}MgO + CO_2 \text{ (for dolomitic quicklime)}$$

The manufacture of lime is also illustrated in Figure 2.10.

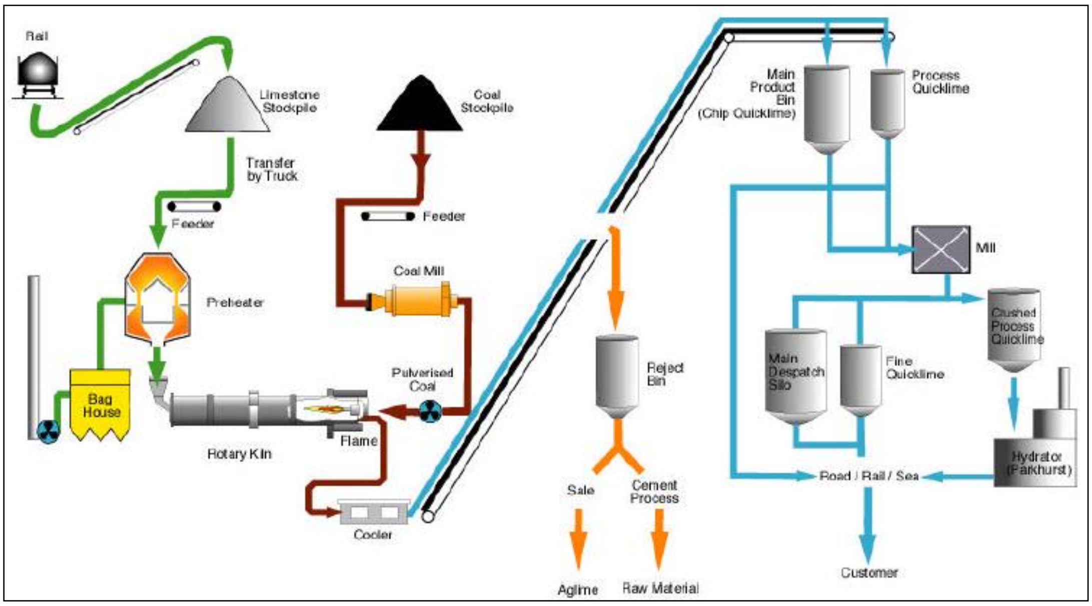

Figure 2.10: Schematic lime manufacturing process

(Source: AustStab, 2002)

According to O'Flaherty (2002), calcination is a reversible reaction. Hence, calcitic hydrated lime is obtained when a limited amount of water is added to calcitic quicklime during production, an exothermic reaction associated with expansion. Dolomitic quicklime does not hydrate so readily under similar conditions and produces dolomitic monohydrate lime. In slaking, however, quicklime is combined with an excess amount of water to produce slaked lime slurry of varying consistency. In this regard, quicklime is only available as a dry granular material while hydrated lime is produced as a fine dry white powder or in slurry form. Hydrated lime contains about 30% water which makes it more expensive than quicklime to handle and transport especially if large quantities and long hauling distances are involved (Powrie, 2004; Little, 1995). The calcined lime is not the same as non-calcined and chemically inactive 'agricultural lime' which is not suitable enough for soil stabilization (National Lime Association [NLA], 2004; Cook et al., 2001).

AustStab (2002) cautioned that commercial quicklime is never 100% calcium oxide since the quality of lime depends on nature of parent material and production process. First, limestone feedstocks for calcination are not pure carbonate and, secondly, over-burning in the kilns results in increased impurities as cement clinker minerals. Thus, kilning processes have inherent inefficiencies that effectively reduce calcium content and the ability of lime to react with water. In this respect, the percent of lime present should be determined in the laboratory if it is not designated on the package. Lime production is a high energy consumption process that is responsible for a considerable amount of carbon dioxide and dust emissions, all that affects the environment negatively (Jawad et al., 2014; Little, 1995).

2.3.3.2 Properties of Lime

According to Little (1995), lime is generally white in color, which varies in intensity with its chemical purity. However, hydrated limes are whiter than quicklime. Fineness, specific gravity, bulk density, heat of solution, and solubility are the most important physical properties of lime related to soil stabilization. Normal grades of hydrated lime when air-classified have 75-95% passing 75μm (or 0.075mm) sieve, a specific gravity of 2.0 to 2.2, and a bulk density of 400 to 640 kg/m^3. Quicklime is non-toxic but it is irritant, caustic and has a highly exothermic reaction with water. However, hydrated lime is much less caustic and reactive and hence most often used than quicklime (Guyer, 2011; O'Flaherty, 2002; Hudson, 1997).

Little (1995) also reported that quicklime and hydrated lime are reasonably stable chemical compounds but quicklime is more vulnerable to water than hydrated lime whose composition remains stable or unchanged in presence of water. Moreover, lime in solution gives a 'basic' environment with a peak pH value of approximately 12.45 at a temperature of 25°C, which is of great practical importance in soil stabilization. This environment facilitates the complex reactions between lime and the compounds of silica (SiO_2) and alumina (Al_2O_3) in clay minerals. According to AustStab (2002), these reactions result in the formation of hydrated calcium silicate and calcium-aluminate compounds in the presence of water, similar to those found in cement paste. Jawad et al. (2014) and Little (1995) also reported that lime reacts with carbon dioxide in the air and rainwater in a process known as carbonation where water and fineness of lime act as catalysts. In effect, the process of carbonation robs the construction system of quality lime that is reactive with soil minerals and results in development of lower strengths than otherwise expected. Thus, lime should generally be protected against weather prior to use to minimize carbonation, and hydrated lime is best used within 4-6 months from the manufacturing date otherwise it should be discarded (Johannessen, 2008; O'Flaherty, 2002).

The hydrated lime used in this study was a white powder described as high calcium type CL 60 by the manufacturer and which was made in accordance to KS 1780-3:2010 and certified by both BS EN ISO 14001:2004 and BS EN ISO 9001:2008. The amount of lime was expected to be at 60% as suggested by type on package.

2.3.3.3 Water

Water is generally required to hydrate the binder, improve workability, and facilitate compaction of any admixture. According to Arora (2004), O'Flaherty (2002) and McNally (1998), the water should be potable and its chemical quality in terms of salinity, pH and temperature should be known. The water should also be free from harmful amounts of salts, alkalies, acids or organic matter; sulphates should be less than 0.5%. However, the ability of quicklime to form alkaline solutions or suspensions in water is key to modification of certain soils in such a way that the end result is a benefit to road engineers (AustStab, 2002).

2.4 Soil Stabilization

Soil stabilization is a general term that is common with highway engineers. It is defined as any treatment applied to a soil, including compaction, to alter one or more of its properties for improved engineering performance of earthworks (Mampearachchi et al., 2017; O'Flaherty, 2002; Hudson, 1997). Therefore, the main objective of soil stabilization is to enhance quality of marginal materials and to expedite construction through increased mechanical strength, stiffness, stability, permeability, workability and durability; stability means reduced vulnerability of soil to volume change and compressibility. Accordingly, the general economic and environmental benefits of soil stabilization, notwithstanding the purpose for stabilization, are as follows (Ejeta et al., 2017; Elsharief et al., 2013; Holt, 2010):

1) Use of poor non-renewable natural resources available locally,
2) Reduced demand on non-renewable natural resources,
3) Reduced borrow material to replace weak soils,
4) Reduced hauling cost and deterioration of local road networks, and
5) Cleaner environment from reduced noise, air, and water pollution caused by construction plant, carbon and smoke emissions, and fuel and oil spillage.

Despite the many benefits of soil stabilization, McNally (1998) and Hudson (1997) cautioned that stabilization should be employed when it is more economical to overcome a deficiency in a readily available material than to bring in one that fully complies with the specification requirements. They opined that stabilization should be a last resort for upgrading marginal materials, and should be applicable only where no economic alternative is available. They therefore asserted that the first decision to be made if such borderline materials are encountered is whether stabilization should be attempted at all! Nonetheless, successful stabilization of soils depends on their physio-chemical properties that vary with soil composition (Little, 2009; Cook et al., 2001). Therefore to assist in decision-making, McNally (1998) and Hudson (1997) indicated that suitability of a soil intended for stabilization should be tested in a geotechnical laboratory for its easily measurable attributes like gradation, plasticity, and strength. Too great an emphasis should, however, be placed on obtaining a sufficiently dense mixture that meets stability needs whilst maximizing use of the readily available low-cost soils, and not on achieving the ideal gradation (O’Flaherty, 2002).

2.4.1 History of Soil Stabilization

Soil stabilization has been practiced for several millennia, especially after invention of the wheel by the Sumerians at about 5,000 BC (O’Flaherty, 1997; Wignall et al., 1999). Many ancient cultures like the Persians, Chinese, Incas and the Romans used various techniques to improve soil suitability, some of which were so effective that many of the roadways still exist today. Then, as now, poor soils could not be avoided especially in flood plains, and neither were good materials always available everywhere (Johannessen, 2008). Condition of the earth roads too varied with climatic seasons, becoming rutted and impassable during wet seasons. This required some degree of mitigation over weak sections of the roads and the most likely remedial measure was to cover the muddy spots with granular material. This procedure may have been improved over time to cover entire roads for better quality and comfort. Thus, natural soils were probably partially removed and replaced to a certain depth with a more durable layer of granular material, the probable origin of road graveling seen today. However, lack of granular materials in some areas could have led the ancient Mesopotamians and Romans to separately discover the method of strengthening weak soils using pulverized limestone (O'Flaherty, 1997). The ancient Egyptians and Ethiopians are also associated with use of gypsum. These methods marked the birth of modern mechanical and or chemical (or lime) stabilization of soils.

Modern soil stabilization began in the United States of America (USA) in the 1920's. It was a chance discovery that rose from use of the highly toxic, liquid waste from paper mills as dust palliative in earth/dirt roads. The surprise outcome of this creative way of disposing the waste was a hardened surface to the dirt roads unlike other untreated roads. However, it was not until decades later in the 1940's-60's that the reason for this change begun to be understood. After extensive private research, it was established that the change was caused by a chemical reaction between the waste solution and the clay particles in the soil. It was about this time when general shortages of aggregates and fuel resources forced road engineers to consider alternatives to the conventional techniques of mechanically replacing poor soils with borrow aggregates possessing better engineering properties. This firmly established cement, lime and bitumen as the traditional soil stabilizers globally; as a product of limestone since 1824 AD, Portland cement is similar to lime (O'Flaherty, 2002). Nonetheless, soil stabilization later fell out of favor, mainly due to faulty application techniques and misunderstandings. Lime stabilization in particular suffered this fate notwithstanding its being one of the oldest forms of stabilization that has a long and successful history with clay subgrades (AustStab, 2002).

Soil stabilization has bounced back in recent years to become a popular trend in many parts of the world like in Australia, India, United Kingdom (UK) and the USA (Wilmot, 2006). This is driven primarily by environmental concerns and also by increased global demand for raw materials, fuel and infrastructure like roads. This time, however, soil stabilization is benefiting from advanced research, materials and technology. Advancements in soil testing and research have led to the development of new soil stabilization substances, and of improved original traditional stabilizers with additional ingredients to make them effective in a much wider range of soil types and conditions. The new non-traditional stabilizers include fly ash, cement kiln dust (CKD), lime kiln dust (LKD), synthetic polymers, ionic stabilizers, and salts (chlorides) of sodium and calcium. Research on ashes from agricultural waste, molasses, lignin and tree resins is on-going. Mechanical stabilization has not been left behind as there is also fiber reinforcement of soil using geosynthetics made from polymeric materials like thermoplastics (Shuklar & Yin, 2006). Soil stabilization is achieved by means of confinement of particle movement by the grid which also acts as a tensioned membrane to provide high compressive and tensile strength of the reinforced soil. There is also thermal and electrical treatment as well as grouting of soils. Crushed waste from mining, quarrying and construction activities are also used to a smaller scale in dust control and soil stabilization of local roads.

2.4.2 Methods of Soil Stabilization

There is a wide variety of soils that may be stabilized. A particular soil can often be stabilized in a number of ways, each of which has its particular influence on properties of different soils. This means that certain general correlations exist between soil composition and soil response to various types of additives. Nevertheless, the method of soil stabilization selected should be verified in the laboratory before ordering of materials or construction. The treated soil can be utilized as a modified subgrade, a sub-base replacement or even as base course in a road pavement (Holt, 2010).

According to Rijn (2005), O'Flaherty (2002), Hudson (1997) and Matalucci (1962), the techniques involved in improvement of soils involves compaction, dilution (or blending), cementation, chemical reaction and waterproofing. Some stabilization procedures employ a combination of these techniques. Moreover, all stabilization methods fall into either (1) Mechanical or 'granular' stabilization, or (2) Chemical or 'chemical admixture' stabilization.

2.4.2.1 Mechanical Stabilization

Arora (2004) and McNally (1998) described mechanical stabilization as the process of improving a soil by changing its gradation and sometimes plasticity. The method involves soil blending and compaction where a raw soil is admixed with about 10-50% of another contrasting soil to achieve a dense homogeneous mass (Little, 2009; O'Flaherty, 2002; Cook et al., 2001). Nonetheless, recent works on sand stabilization of clayey soils conducted by Kollaros and Athanasopoulou (2016) and Jjuuko et al. (2011) showed that blending could go as high as 60-80% concentration of the stabilizer. Up to 100% blending was deemed appropriate in this study.

Mechanical stabilization is the simplest method of stabilizing virtually any soil with worldwide application and its essence is the use of locally available materials and should be given first priority (Johannessen, 2008; Hudson, 1997). The process depends on gradation, interlocking of aggregate particles, internal friction, plasticity of some fines, and actual soil compaction. For this reason, the best aggregates are those consisting of hard, durable and angular or ragged particles. Due to soil variability, however, there is no absolute prescription regarding the nature and amounts of fine material that can be used in granular-stabilized pavements.

2.4.2.2 Chemical Stabilization

Chemical stabilization uses hydraulic binders like cement, lime, fly ash, and CKD mainly to alter the chemical properties of a wide range of clayey and granular soils (Holt, 2010; O'Flaherty, 2002; Hudson, 1997). Synthetic chemicals like chloride and sulphate salts can also be used as binders (Guyer, 2011; Cook et al., 2001). According to Johannessen (2008), the soil becomes stabilized when cement or lime reacts chemically with the soil particles to form cementitious hydrate gels that bind them together. The gels are typically calcium aluminate hydrates (CAH) and calcium silicate hydrates (CSH) which are as a result of three basic chemical reactions – cation exchange, flocculation and agglomeration, and pozzolanic reaction. These reactions depend on amount and type of clay mineral and calcium content and all are nearly complete within 48-72 hours of mixing with water (Jawad et al., 2014; O'Flaherty, 2002; Little, 1995). Organic soils cannot be stabilized satisfactorily because they prevent the chemical reaction.

Chemical stabilization has been practiced for decades in road construction, particularly where aggregates are scarce and haulage distances are significant, and it eliminates the need to remove and replace inherently weak soil subgrades (Elsharief et al., 2013; Holt, 2010). Over 90% of all chemical stabilization projects use the traditional cement and lime or their combinations, while fly ash and CKD are typically used as a partial replacement for the more common conventional binders. Thus, the method is costly and should be considered only when using quality natural or processed materials for soil blending is not feasible, more expensive or does not produce a satisfactory treated soil material (Johannessen, 2008; Hudson, 1997).

a) Cement Stabilization

According to Arora (2004), soil-cement is the material obtained by mixing pulverized soil with cement. The important factors that affect soil-cement are type of soil, quantities of cement, water and admixtures added, and conditions of mixing, compaction and curing. The cementing action is due to hydration when cement reacts with siliceous soil. Cement also becomes more responsive in soil stabilization when some common additives like lime, fly ash and salts like calcium chloride are used.

Granular soils with some fines are best treated with cement though presence of organic matter interferes with hydration of cement, especially in sandy soils, causing a reduction in strength (Johannessen, 2008; Arora, 2004). The actual cement content is ascertained by laboratory tests, including unconfined compressive strength (UCS), but a rough guide for different types of soils is 5-10% for gravels, 7-12% for sands, 12-15% for silts and 12-20% for clays in tropical climate.

b) Lime Stabilization

Lime is the oldest chemical stabilizer best used to improve road-making materials in tropical climate (Jawad et al., 2014; Elsharief et al., 2013). It is more useful for clayey soils and silts where it has a more rapid and marked effect (Johannessen, 2008; Arora, 2004). Lime reacts with silica and alumina in the clay to produce a binding cement which creates long-lasting changes of structural benefits to the lime-treated material (Jawad et al., 2014; NLA, 2005, 2006; Powrie, 2004). Like in cement stabilization, this is affected by soil mineralogy, clay content, type and content of lime, soil-lime mixing, compaction, curing conditions and presence of deleterious matter (Guyer, 2011; AustStab, 2002). However, the material becomes less plastic, much easier to compact, stronger, more friable and flexible but brittle.

Hydrated lime is widely used, either as a dry powder or as slaked lime, since it is more safe and convenient to handle than quicklime. The amount of lime required is normally 2-10% of the soil but as a rough guide, about 10% is used for heavy clays, 5-10% for soils with more than 50% fines, and 2-5% for clay-gravel material with less than 50% fines. Additionally, lime is not effective for granular and sandy soils but may be used in combination with clay, 10-20% fly ash or other pozzolanic materials to make it more responsive. Lime has also been found to be more effective with clay-gravel admixtures that exhibit considerable strength gain due to the self-cementing action of the fines, than with cement stabilization (Holt, 2010; Cook et al., 2001; Little, 1995).

c) Salt Stabilization

Salt stabilization mainly uses chlorides of calcium and sodium as well as sodium silicate; calcium is more effective than sodium chloride. According to Arora (2004) and O'Flaherty (2002), chlorides have excellent hygroscopic and deliquescent properties – they absorb moisture from the air and get dissolved in it. They are not cementing agents but are used as good lubricants in already mechanically stable unsealed granular-stabilized pavements to increase compacted density, make the soil impervious, retard development of ravelling and corrugation conditions, and as dust palliative in warm climates. Salt stabilization also applies to treating of base and subbase aggregates for roadwork but has not been tried on subgrades.

Arora (2004) indicated that salt stabilization is relatively inexpensive since the salts are used in very small quantities of about 0.5% of calcium chloride, 1% of sodium chloride, and 0.1-0.2% of sodium silicate, by weight of soil. However, the salts are liable to leaching and their periodical re-application may be necessary. Moreover, chlorides are not cementing agents (O'Flaherty, 2002). Thus, the long-term durability of the method is doubtful and a salt-stabilized pavement must be already of well-graded and densely compacted materials since the treated soil may lose strength when exposed to weather and groundwater. Splash water from salt-stabilized roads also has the undesirable corroding effect upon the steel in motor vehicles.

d) Other Chemicals

Chrome lignin is obtained from wood as a by-product of sulphite paper manufacture. When 5-20% is added to a soil by weight, it slowly reacts to form a gel that causes bonding of particles. According to Arora (2004), lignin is soluble in water and its stabilizing effect is not permanent. Additionally, resins are natural polymers and when added to a soil, reaction takes place to stabilize it: There are natural and synthetic polymers.

2.4.2.3 Bituminous Stabilization

Bituminous stabilization is done with cut-back bitumen or bitumen emulsions as the bonding and cementing agent (Arora, 2004). Its primary objective is to waterproof the soil, reduce moisture and increase strength by bonding granular soil particles together and plugging the voids in fine soils to reduce water absorption. The stabilized product is known as sand-bitumen and soil-bitumen for granular and cohesive soils, respectively. Like cement and lime stabilization, bituminous stabilization is affected by soil type, bitumen content, mixing and compaction. The amount of bitumen required is determined by trial but it is often about 4-7% by weight of soil. However, the method expensive since bitumen is not readily available.

2.4.2.4 Injection Stabilization

Injection stabilization, also known as grouting, is an in-situ method of improving water resistance, strength and other properties of a natural soil deposit. In this method, a bonding material is always forced or injected under pressure into the natural soil deposit. The stabilizer used depends on type of soil and include clay, cement, lime, sodium silicate as 'water glass', chrome lignin, polymer and bitumen (Arora, 2004). The method is used to improve a soil deposit of considerable thickness or depth of earth and cannot be disturbed, or where the treated area is close to existing structures and other facilities.

During injection, stabilizing material moves through and fills the void spaces in the soil. The method therefore depends on permeability of the soil and viscosity of stabilizing material. Thus, the method of injection is not suitable for stabilizers of high viscosity and for clays because of very low permeability. Also, there is always some uncertainty in results obtained since the stabilizer may not be distributed uniformly throughout the soil mass as desired. Moreover, grouting requires use of specialized equipment and is best handled by specialty contractors. Nevertheless, the method is generally economical, effective and friendly to the environment.

2.4.2.5 Thermal Stabilization

According to Arora (2004), thermal stabilization is done either by heating or cooling a soil to markedly improve its properties. Heating leads to loss of free moisture and an increase in strength; heating above 100ºC drives off adsorbed water is and results in further increase in strength. Heating clay soils to about 200°C also significantly reduce the potential for volume change and increase soil strength. When the soil is heated to 400-600°C, it undergoes irreversible changes and becomes non-plastic, non-expansive and the clay clods convert into aggregates. Above this, some fusion and vitrification occurs to produce a brick-like material used as artificial aggregate.

Conversely, cooling causes a small loss of strength of clay soils but further cooling freezes pore water that act as a cementing agent in soil stabilization. Strength of the soil increases as more and more water freezes but no practical and economic methods have been developed (Arora, 2004). Thermal stabilization is used in some special cases like tunnelling.

2.4.2.6 Electrical Stabilization

Also known as electrochemical hardening, electrical treatment of a soil uses electrodes and a direct electric current (DC) to alter the physico-chemical properties of clay soils. A solution with a high concentration of preferred exchangeable cations act as a stabilizing agent by the process of electro-osmosis and base exchange of clay minerals when the electric current is passed. Removal of water considerably increases strength of the soil and reduces both its swell pressure and amount of swell. The method is expensive and is economical only for much localized areas like in drainage of cohesive soils (Arora, 2004).

2.4.2.7 Geotextile Stabilization

Geotextiles are porous or mesh-fabrics of high tensile strength made mainly of plastics, cotton, jute and wool. The fabrics have a grid form and are widely used in areas exposed to flowing water like in embankments and earth dams. Fabrics are a recent development and hence they are expensive and not readily available.

2.4.3 Selection of Method and Stabilizer

The responsibility to select or specify the correct stabilizing method, technique, and quantity of material required for the prevailing conditions lies with engineers (Cook et al., 2001). According to Guyer (2011) and Little (2009), selection of a given stabilizer should be based on its effectiveness to improve properties of the selected soil. O'Flaherty (2002) and Hudson (1997) added that the decision on method and type of stabilizer to use is primarily a financial one though the skills, resources, construction equipment and alternatives available are often considered. Some care is also required in selecting a suitable stabilizer. Therefore, it is important to establish type of chemical admixtures available for use, and if any special equipment or training is required to successfully incorporate the selected admixture.

There may be more than one candidate stabilizer applicable for one soil type. Figure 2.11 represents a simple and excellent analysis of the technical procedure involved in selection of the best stabilizer for fine soils.

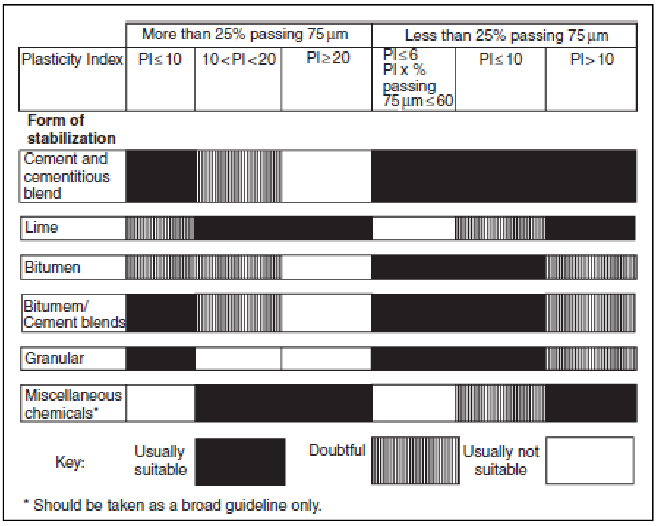

Figure 2.11: Guideline to select suitable method of soil stabilization

(Source: O'Flaherty, 2002; Cook et al., 2001)

Guyer (2011), Holt (2010) and Little (2009) asserted that representative soil samples must be prepared and tested in the laboratory for index properties and binder concentration once a suitable stabilizing agent is determined. Presence of certain types of salts and organic matter in the soil inhibits normal hydration process and pozzolanic reaction, retarding the hardening and strength gain in stabilized soils. In particular, soils with an organic content of 1-2% by weight may be difficult to stabilize or may require uneconomical quantities of binder to stabilize. Similarly, soils containing sulphates in excess of 0.3% or 3,000 ppm have potential to react with calcium and aluminium to form expansive minerals. This would create soil distress in lime stabilization, while heaving and disintegration may also occur, leading to loss of strength. In this regard, Jawad et al. (2014) and Cook et al. (2001) recommended that the possible impact of deleterious components of the soil must be considered once an additive, particularly lime, has been selected. Thereafter, laboratory results are used to develop a design mix for field stabilization that serve to ensure optimum addition of a binder for purposes of meeting the desired minimum engineering performance criteria.

2.4.4 Compaction

The technique of soil compaction normally accounts for less than 5% of the total construction costs and is perhaps the least expensive method of improving soils (Budhu, 2011; Rijn, 2005). It is also the most widely recognized form of stabilization that improves mechanical stability and strength to varying degree. Nonetheless, compaction alone is often not enough especially with fine-grained, cohesive soils which are susceptible to swelling (Little, 1995). Despite this reality, a common method of mechanically stabilizing an existing clay soil is to add gravel or other granular materials that do not soften when wet or pulverize under traffic.

Nevertheless, compaction of soil stabilized with hydraulic binders is often marked by an increase in OMC and a decrease in MDD but with higher strengths. These compaction characteristics are illustrated in Figure 2.12 where slope of the curve also represents sensitivity of soil admixture to addition of water (Jjjuko et al., 2014).

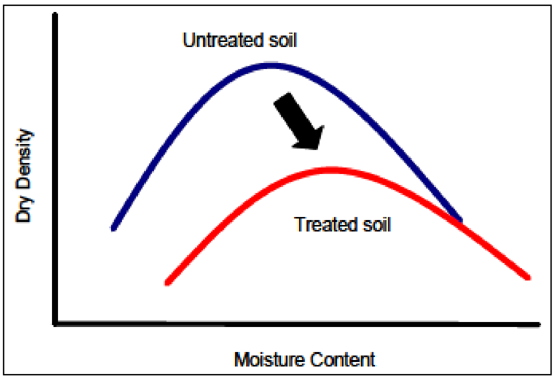

Figure 2.12: Effect of lime on compaction characteristics

(Source: Holt, 2010)

Lime is used in soil treatment for a variety of reasons and for a wide range of soils (O'Flaherty, 2002, Wignall et al., 1999; Little, 1995). This versatile soil stabilizer is used mainly to improve the plastic properties of the soil and is effective in most cohesive and excessively clayey soils and plastic aggregates. Lime also substantially reduces the ability of clay to hold water and creates a water barrier to capillary moisture from below making it a significant and an unparalleled aid that is used to treat wet and weak clay subgrades (NLA, 2001). Quicklime is particularly very effective in drying wet soils and minimizing weather-related construction delays as it hydrates to save time and money. Lime treatment is also used to increase stability, impermeability, and load-bearing strength of weak soils. In particular, lime treatment improves the tensile and flexural strength of the soil combined with reduced shrinkage potential all of which substantially increases the gap-spanning capabilities and reduces cracking tendencies of the stabilized soil respectively.

Lime stabilization is used primarily to upgrade poor clay soils into adequate subgrade support or capping and is an economical approach to clay subgrade with plasticity index (PI) exceeding 5%. Thus, lime stabilization may transform clay soils into subbase, and even form a strong and high quality base course with marginal granular base materials (Guyer, 2011) as envisaged in this study. It is therefore applicable particularly where the natural soils are excessively (i) clayey and no better material is economically available, (ii) wet and cannot be dried out (Ministry of Transport and Communications [MoTC], 1987; Ministry of Roads and Public Works [MoR&PW], 1986).

Nevertheless, lime may be used as a preliminary additive to reduce the plasticity or alter gradation of a soil before adding the primary stabilizing agent (Hudson, 1997). Vorobief and Murphy (2003) and O'Flaherty (2002) also held that there is normally little value in adding lime to silty, sandy or gravelly soils with less than 10% clay contents as the pozzolanic reaction will be minimal. Moreover, there is a critical lime content beyond which there is no more strength gain and the strength either declines or remains constant during the curing period which should be maintained at temperatures above 10°C to avoid halting of the pozzolanic reaction. Lime also gains strength quite rapidly initially but this slows to a fairly constant rate in the long-term. It therefore requires about 14 days in hot weather and 28 days in cool weather to gain significant strength but the effects of lime stabilization are typically measured after 28 days or longer (NLA, 2006; Hudson, 1997).

2.5 Material Testing

Gianfrancisco and Jenkins (2000), Skorseth and Selim (2000), and Morin et al. (1971) held that all roads should be built of materials that will make them passable in all kinds of weather. Nonetheless, different uses require different types of material while the question of quality and adequacy of the available materials in road structures has never been fully investigated in the tropics. Generally, one can simply tell a little about them by feel or visual inspection but it is only by testing that real quality of a material and its suitability for specific use can really be determined. For this reason, Hudson (1997) discouraged the use of the 'rule of thumb' that is based on visual inspection or apparent similarity to other soils in selecting a soil for particular use. He also cautioned that misunderstanding soils and their properties can lead to construction errors that are costly in effort and material.

Testing is often done prior to use of material and at the time of construction for several benefits. Most importantly, testing increases knowledge of materials that forms the basis for appropriate road design, for quality control purposes by empowering decision-makers to specify good materials and know when to accept or reject materials, and for better communication with contractors, consultants, and others involved in road construction (Robinson & Thagesen 2004; Skorseth & Selim

2000). According to Gianfrancisco and Jenkins (2000) and Intech Associates (2002), testing reduces the risk of future problems while research enables development of more accurate predictions. However, the essential knowledge on the overall characteristics of a material is no substitute for local experience. Additionally, quality control is as important to the final product as is proper design.

Soils are naturally the oldest but most complex engineering materials that are never homogeneous in character throughout the world, even when apparently similar to soils in other regions (Das, 2011; Northmore et al., 1992a). According to Budhu (2011), laboratory tests on invariably disturbed soil samples allow for better control of the test conditions applied to the soil than in situ tests where sufficient care must be taken to reduce testing disturbances that can significantly affect the test results. To obtain good laboratory results, it is then absolutely critical to get adequate and truly representative sample of materials in the field. However, sampling and testing is costly whereas poor sampling techniques have often led to more controversy in material testing than any other factor (Cook et al., 2001; Skorseth & Selim, 2000).

Laboratory testing programs are carried out mainly to determine properties that normally reflect the expected performance characteristics of the material in service. This includes the physical, mechanical, chemical and mineralogical characteristics that are needed to determine strength, settlement, and stiffness parameters of soils for design and construction (Budhu, 2011). They are also needed to classify soils as shown in Figure 2.13, which are broadly described as ‘fine’ and ‘coarse’ depending on the predominant size of particles within the soil. The fundamental investigative laboratory tests for soils meant for roadworks include gradation, specific gravity, plasticity, swelling, shrinkage, compaction, and shear strength as UCS, and California Bearing Ratio [CBR] (Day, 2010; O’Flaherty, 2002; Cook et al., 2001; Hudson, 1997; Wambura et al., 1991). The soils may also be tested for mineralogy, clay activity, permeability and consolidation while granular materials may be tested for particle shape, wearing resistance and crushing strength.

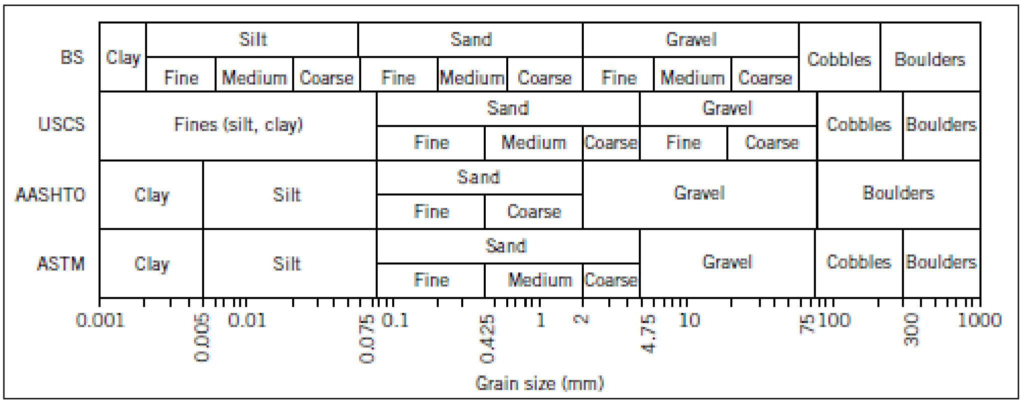

Figure 2.13: Common soil description systems based on particle size

(Source: Budhu, 2011)

Grading and plasticity of soils in particular are the intrinsic or ‘fingerprint’ properties of a soil that serve as very useful tools for soil classification, and as a benchmark for evaluating and comparing the probable soil behavior, and for providing correlations with engineering properties (Das, 2011; Wesley, 2010; Northmore et al., 1992b). Standards for the acceptance or rejection of materials placed in roads are therefore invariably derived from particle-size distribution, Atterberg limits and the CBR which are perhaps the best methods of making a rapid assessment of the properties of a soil (Morin et al., 1971; Dixon & Robertson, 1970). Particle size analysis has many uses in engineering and especially in the selection of soils and aggregates for construction purposes since it highly influences the engineering properties like maximum density, optimum water content, and strength and is also frequently used for mix-design when stabilizing soils (Budhu, 2011; Day, 2010; O’Flaherty, 2002). The activity of soil as defined by Skempton (1953) also serves as a good indicator of potential shrink-swell problems associated with expansive soils (Das, 2011).

2.5.1 Soil Mineralogy

Besides the difference in grain size that alone tells very little about the physical properties of fine soils, different types of soils can be identified with the aid of color, mineralogy and chemical composition of a soil (Day, 2010; Verrujit, 2001). Clay mineralogy, chemical composition and micro-structure are very significant factors that largely control clay soil behavior and must clearly not be ignored in the assessment of its behavior (Northmore et al., 1992a; Dixon & Robertson, 1970).

However, chemical composition does not give much quantitative information of a soil but serves as a warning of its characteristics. In contrast, Budhu (2011) and Little (1995) held that a look at soil mineralogy shows that grouping of soils according to size is only a beginning in understanding soil behavior. Moreover, knowledge in soil mineralogy actually provides an excellent background for an engineer seeking to understand the unique phenomena involved in soil stabilization.

Clay minerals are essentially very tiny crystalline substances evolved primarily from chemical weathering of certain rocks and are dominantly hydrous aluminosilicates (Das, 2011; O'Flaherty, 2002; Powrie, 2004; Smoltczyk, 2003). Additionally, clay minerals also contain other loosely bonded and exchangeable metallic ions called cations which form the basis of soil stabilization with chemicals like lime. Clay minerals of greatest interest comprise the very stable kaolinite, montmorillonite (or smectite) with one of the largest specific surface and cation exchange capacity (CEC), and illite. There are also lesser and rare clay minerals generally considered to be amorphous and non-crystalline and include allophane and halloysite which are particularly common in volcanic soils. Halloysite has a unique tubular shape. Table 2.5 shows characteristics of some common clay minerals.

Table 2.5: Typical characteristics of some clay minerals

Mineral	Specific Surface Area (m^2/g)	Specific Gravity, Gs	Clay Activity, A	CEC (mEq/100g)
Kaolinite	10-20	2.6	0.5	3-15
Halloysite	40	2.0-2.55	0.5-1	5-50
Chlorite	5-50	2.6-2.9	-	10-40
Illite	65-100	2.8	0.5-1	10-40
Allophane	-	-	0.5-1.2	25-50
Montmorillonite	100-800	2.65-2.80	1.7	60-150
Vermiculite	5-400	-	-	100-150

(Source: Das, 2011; Smoltczyk, 2003; Geological Society of London [GSL], 1990)

All clay minerals are colloidal-sized and consist of many repetitive and flake-like crystal sheets. Thus, the methods for studying mineralogy and geometrical structure of clay particles are mostly X-Ray Diffraction (XRD), Differential Thermal Analysis (DTA), optical microscopy, and scanning electron microscopy (Verrujit, 2001; Holtz & Kovacs, 1981). Chemical analysis of soil samples is done by the rapid method of

X-Ray Fluorescence (XRF) spectroscopy (Das, 2011; Day, 2010). However, these processes are rather complicated, expensive, and involve special equipment that is not readily available to the geotechnical engineer. Thus, the predominant clay mineral present in a soil can easily and inexpensively be determined from its activity (A) as shown in Table 2.6, and from its plasticity index (PI) as illustrated in Figure 2.14 for some tropical residual soils. However, this approach may be inaccurate where more than one clay mineral is present (Day, 2010; O'Flaherty, 2002).

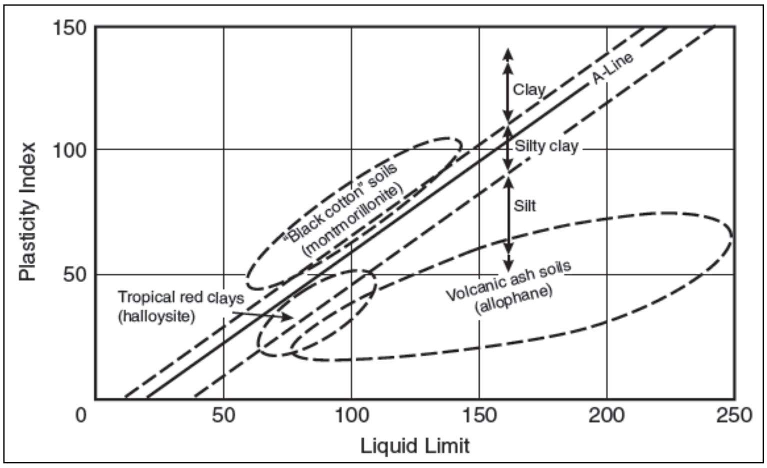

Figure 2.14: Conventional plasticity locating some tropical residual soils

Source: Wesley (2010)

2.5.2 Lime

According to Little (2009, 1995) and O'Flaherty (2002), aggregates are used in road pavements either on their own or in combination with a cementitious material like lime but the percentage used must be determined by recognized laboratory testing or empirical methods. It is however important to appreciate that different design contents may be determined for the same soil as this depends upon the design procedure used for testing and upon the objectives of the lime treatment. For instance, the mix design protocol may be designed to optimize the potential for long-term strength gain and durability of lime stabilized soils.

AustStab (2002) gave the common tests carried out in lime stabilization for roads as:

1) Determination of the quality of lime,
2) Determination of Available Lime Index, that is, the Lime Content.
3) Lime Demand Test,

4) Determination of California Bearing Ratio (CBR) and swell potential of compacted material, and
5) Determination of the unconfined compressive strength.

2.5.2.1 Lime Quality and Available Lime

The quality of lime represents its grade and may be expressed in terms of its inherent reactivity, its fineness (particle size), and its degree of purity. The grade indicates the amount of lime that is chemically available to the users for most reactions. According to NLA (2006), lime can react with moisture and carbon dioxide and hence careful storage is required to maintain its integrity and produce reliable results.

AustStab (2002) defined the Available Lime Index as a measure of the amount of calcium oxide present in a lime. The available lime is expressed as a percentage of the ‘available quicklime [CaO]’ or ‘available hydrated lime [$Ca(OH)_2$]’. However, it varies with source and manufacturer which are represented by the brand name.

2.5.2.2 Initial Lime Consumption

Most lime develop a highly alkaline environment of about 12.4 pH when placed in a water solution and the lowest lime content required to create this level of alkalinity in the soil-lime solution is known as the Initial Consumption of Lime (ICL) or simply the initial design lime content (AustStab, 2002; Cook et al., 2001; Hudson, 1997). According to NLA (2006), it is important to appreciate that the ICL is also the minimum lime percentage for stabilizing the soil. When introduced into a soil, lime reacts with the soil and generally causes a significant change in the soil texture, plasticity and compaction characteristics of the host soil proportional to concentration of the binder (Jawad et al., 2014; Holt, 2010; Powrie, 2004). In this regard, both the pH and the PI effectively become good indicators of the desirable lime content of a soil-lime mixture.

Similarly, there are two methods for the determination of initial design lime content which are presented as (Guyer, 2011; AustStab, 2002; Hudson, 1997):

1. **Lime Demand Method** – This is the preferred method which aims to obtain the quantity of lime required to reach the desired pH level for satisfactory cation exchange and to produce long-term chemical reactions.

2. **Plasticity Method** – This is the alternate method which is based on the plasticity index and fraction of soil passing 425 μm (or No. 40) sieve, as illustrated in Figure 2.15 for hydrated lime and soil with at least 10% fines. These basic properties are easily obtained from the laboratory consistency and gradation tests of the untreated soil. The method was adopted in the research for its simplicity and possible expedient construction in the field when adopted.

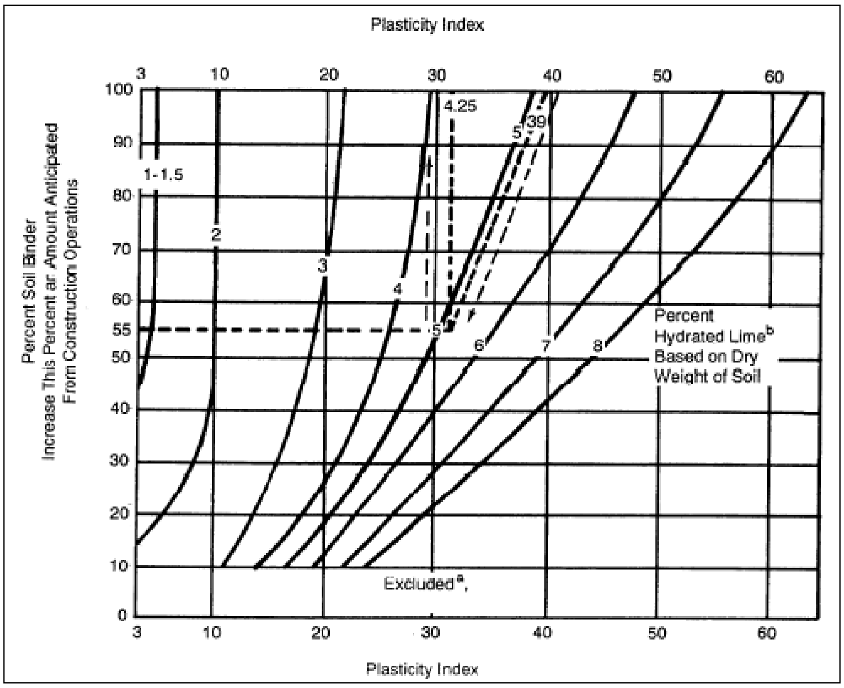

Figure 2.15: Plasticity method for determining initial lime content

(Source: Hudson, 1997)

2.5.2.3 Lime Mix Design

According to O'Flaherty (2002) and Little (1995), the total benefits of lime stabilization are reduced if insufficient lime is used since the proper stabilization may not occur. For this reason, they asserted that generalizations and guesswork to the amount of lime to be specified should never be allowed. The amount of lime is then determined by conducting traditional laboratory tests on selected engineering properties to establish the optimum amount required for each soil. Holt (2010) added that one of the most important aspects of the mix design is to determine the OMC of the blended or treated mix. Compaction is then one of the tests since lime addition changes the compaction characteristics of the soil (Guyer, 2011; NLA, 2006).

In this respect, compaction should essentially be conducted on the lime-soil mixture after estimating the initial lime content. To establish the ultimate design lime content, Guyer (2011) and Hudson (1997) reported that the soil mixtures for compaction test should be prepared at lime contents equal to ICL+ 2% and ICL + 4% for the standard pH method, and at ICL ± 2% for the alternate plasticity method. They added that each mixture should cure in sealed container for between one and two hours before compaction. According to O'Flaherty (2002) and AustStab (2002), however, the lime contents determined for hydrated lime should be reduced by about 25% to determine the equivalent design content for quicklime, since the ratio of atomic masses for CaO and $Ca(OH)_2$ of 56 and 74 is (56/74=0.76) or 76%.

A small construction tolerance typically of 0.5-1% lime is normally added to the laboratory-determined design lime content to allow for mixing inefficiencies in the field. Moreover, the OMC for maximum strength of a lime-treated soil is not necessarily the same as that for MDD. O'Flaherty (2002) also observed that the OMC for strength tends to be higher than that for dry density with clayey soils while the opposite may be true with silty soils.

2.5.3 Strength Tests

Holt (2010) advanced the view that strength criteria for soil stabilization is not universal but is dependent upon the type of soil to be treated, where the soil is treated, and the intended end use of the stabilized soil. He however reported that the effect of soil stabilization is commonly assessed in terms of strength gain over a certain period of time or cure. The shearing resistance also increases with compaction, and with angularity and size of particle (O'Flaherty, 2002).

According to Little (1995) and Morin et al. (1971), the CBR and the Unconfined Compression Strength (UCS) tests are the most common procedures used to assess strength gains in stabilized soils and as design parameters in road pavements universally. In particular, the UCS provides all the information necessary about the mix design mixtures since the tensile and flexural strength characteristics, which are also important considerations in pavement design, can be reliably predicted from the test. Nonetheless, the UCS test has limited application as the basis for design of stabilized materials and does not appear to have any clear correlation with CBR test.

Thus, the CBR approach is preferred by many pavement designers (Vorobieff & Murphy, 2003).

2.5.3.1 California Bearing Ratio

Devised by O.J. Porter in 1938, the CBR test is an empirical index to evaluate subgrade strengths but has been extended to cover the design and control of the entire road pavements. The performance of pavements has also been correlated with the CBR test more than with any other test. According to AustStab (2002), Hudson (1997) and Morin et al. (1971), CBR test is the common strength test method that provides a rapid and effective method for assessing the suitability of compacted soils. It is also the best means of evaluating the effectiveness of stabilized materials. Thus, considerable experience has been accumulated with the test in many parts of the world, and the test is still popular for pavement design in most African countries.

The CBR is dependent on the type of soil used and its density and moisture content. It is therefore basically a measure of the stiffness and shear strength for a compacted material tested under simulated field conditions of pressure and highest moisture (Venkatramaiah, 2006; Vorobieff & Murphy, 2003; O'Flaherty, 2002). According to Blake (1994), the CBR is expressed as a percentage of a given penetrative force to the standard force for crushed stone at penetrations of 2.5 and 5.0 mm respectively and the higher of two values is adopted. Moreover, it forms the basis for determining the thickness of soil and aggregate layers used in the structural design of pavements, often known as thickness design.

2.5.3.2 Pavement Thickness Design

According to Cook et al. (2001), the purpose of structural design is to limit the stresses induced in the subgrade by traffic to a safe level. It involves the determination of the type and thickness of capping layer, subbase and base and is dependent on the subgrade bearing capacity and the strength of paving materials (Robinson & Thagesen, 2004; Little, 2009; Wignall et al., 1999). According to Vorobieff and Murphy (2003) and (Johannessen (2008), existing pavement design procedures are essentially an empirical science based primarily on CBR selection criteria that also accommodates the use of marginal materials. They however held that the expected levels of traffic density determine the choice of materials and thickness of pavement layers. Accordingly, different road authorities have developed design curves for flexible pavements to determine the appropriate thickness of materials overlying a subgrade under different wheel loads and traffic conditions, as given in Equation 2.1 (Venkatramaiah, 2006):

$$d = \sqrt{\frac{W}{0.57(CBR)} - \frac{A}{\pi}} \qquad (2.1)$$

where d = Total thickness of construction (cm),

W = Maximum load on one wheel (kg),

A = Contact area of one tyre (cm^2), and

CBR = Subgrade strength index.

The CBR approach is fast being replaced by use of the resilient modulus of the subgrade whose laboratory procedures are tedious but can be approximated for 2-13% CBR thus (Venkatramaiah, 2004; O'Flaherty, 2002; AustStab, 2002):

$$E = 17.6 \times (CBR)^{0.64} \qquad (2.2)$$

where E = Resilient Modulus in MPa,

CBR = Subgrade strength index.

The conventional thickness design for a road pavement based on CBR and estimated design traffic is illustrated in Figure 2.16.

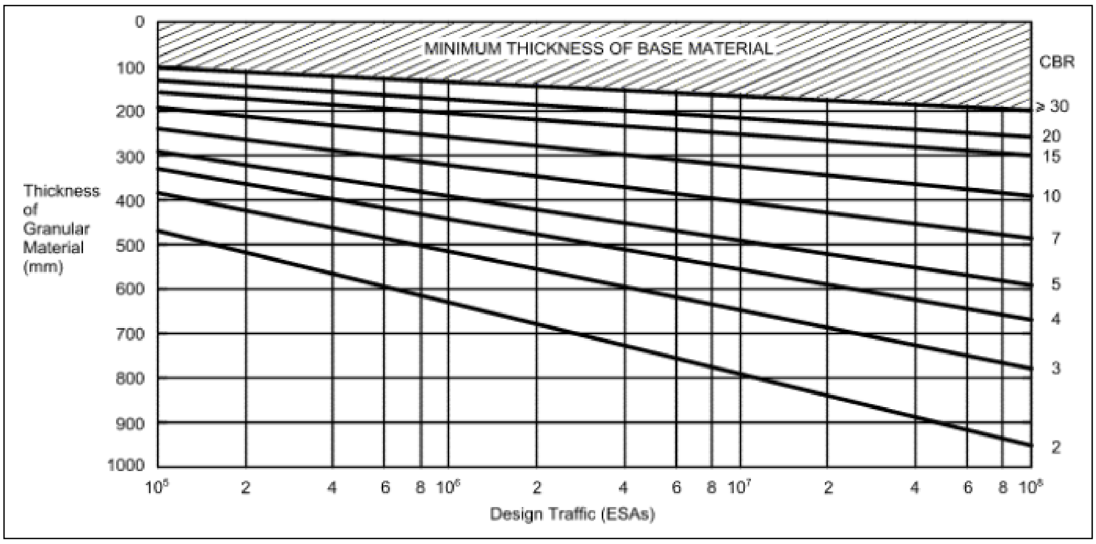

Figure 2.16: CBR thickness design chart for a granular road pavement

(Source: Vorobieff & Murthy, 2003)

As illustrated in Figure 2.17, the intensity of traffic stress normally spreads in a pyramidal pattern throughout the depth of the pavement (Wignall et al., 1999). The strength and quality of material required is therefore greatest at the wearing surface and least at the subgrade levels. In this respect, MoTC (1987) listed six subgrade bearing strength classes based on soaked CBR as shown in Table A7.7 of Appendix III. Moreover, the qualifying strength for subbase and base are given in terms of certain soaked CBR values. Locally, these are 30 and 80% respectively for natural materials, and 60 and 160% for treated materials. Nevertheless, Cook et al. (2013; 2002) and Morin et al. (1971) observed that standards are set to ensure quality and safety but there shall be circumstances in which full compliance with the normal standards will lead to very high costs or environmental impact. They therefore concluded that the conventional CBR requirements are often exceptionally high for LVSRs, with treated base performing satisfactorily with reduced CBR of as low as 80-100% in many countries including Kenya. In effect, many countries have come up with relaxed guidelines for LVSRs. Kenya has come up with the MoTI (2013) design guidelines that accordingly comprise fourteen (14) types of LVSRs and five (5) classes of traffic load to choose from.

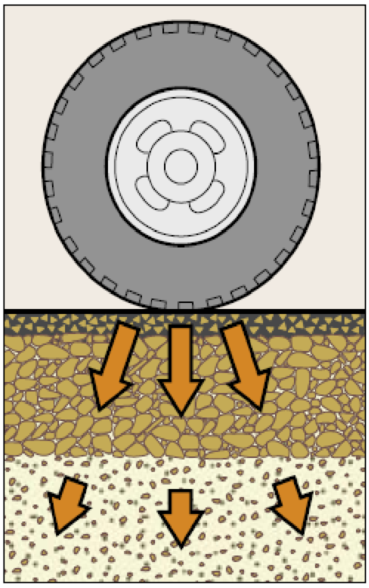

Figure 2.17: Schematic distribution of traffic load through road pavement

(Source: Johannessen, 2008)

2.6 Summary of Empirical Review

Soils are distributed worldwide in a variety of types that depend on the geology, topography and climate of a place (Kamtchueng et al., 2015; Johannessen, 2008; Frempong & Tsidzi, 1999). In this respect, soils are never homogeneous in character but are radically different from one another even when apparently similar to soils in other regions (Das, 2011; Northmore et al., 1992a). Thus, tropical conditions lead to a variety of natural soil deposits that vary considerably and are completely different in material and engineering properties even in two very close locations (Jjuuko et al., 2014; Venkatramaiah, 2006; Rollings et al., 2002; Verrujit, 2001). According to Northmore et al. (1992a), it is therefore a misconception to consider red clay soils as forming a distinct and clearly defined soil type whose properties can be broadly generalized. Hence, most red soils of India, for instance, are quite different in character from those found in Kenya in that they are generally hard, granular, infertile and of low water holding capacity and hence difficult to cultivate (Qayoom et. al., 2020; Aravind et al., 2019). In contrast, Kenyan red 'coffee' soils are cultivable since they are soft, fine, highly hygroscopic and rich in nutrients (Wesley, 2009; Bommer et al., 2002; Rolt, 1979).

Soils are the oldest and most complex engineering materials that usually take up about 70% of cost in road construction projects (Budhu, 2011; O'Flaherty, 2002). As the principal road-making material, suitable local soils for earthworks are often scarce and unavailable within economic haul distances from the road corridor (Qayoom et. al., 2020; Johannessen, 2008). The red clay soil is abundant, reasonably strong and stable but it is considered a marginal material in road construction due to its unusual properties and unpredictable behavior (Elsharief et al., 2013; Wesley, 2010; Waweru et al., 1998; Northmore et al., 1992a). Additionally, soils are non-renewable natural resources critical for our existence but which today's society has exploited in many ways in an unsustainable manner (Budhu, 2011; Hazelton & Murphy, 2007). In this regard, soil stabilization is any technique employed to promote the more efficient and sustainable use of locally available soils in road construction (Mwaipungu & Allopi, 2014; Cook et al., 2013; Foster & Briceño-Garmendia, 2010).

According to Johannessen (2008) and Wilmot (2006), soil stabilization uses a wide range of techniques to improve properties of almost any soil and obtain a high quality product that meets certain performance requirements. Generally, soil stabilization is not only cost effective but it also reduces demand on resources, land degradation and carbon emission (Holt, 2010; Vorobieff & Murphy, 2003; AustStab, 2002). However, the quality of treated soil is governed by soil type, its composition and mineralogy, and the stabilization technique employed (Guyer, 2011; Arora, 2004; AustStab, 2002). Success of soil stabilization highly depends on testing and verification that a soil is treatable in an economic manner (AustStab, 2002). The selection of method of stabilization is a preserve of the engineer but it is dependent upon the known characteristics of a soil, and on intended use and cost of the treated material (Guyer, 2011; Little, 2009).

2.7 Theoretical Review of Past Studies on Red Clay Soil

A considerable body of knowledge exists on geotechnical properties of tropical red clay soils, gathered from research activities conducted in several countries of the SSA, South East Asia, and Latin America. In Kenya, such works have been concentrated on the mineralogy, the abnormal behavior in some properties, and on suitability of the soils as a construction material for embankments notably earth dams covered by Terzaghi (1958), Dixon and Robertson (1970), and Bruggemann and Gosden (2004). The results have shown that engineering properties of red clay soils vary according to the nature of the parent rocks and local climatic regime, and that the soils are of fair quality as raw materials for road construction (Kamtchueng et al., 2015; Smart, 1973). According to Gidigasu (1991), however, transport and road research in the tropics has received relatively little attention, and hence stabilization of the red soils has been investigated to an even more limited extent. A comprehensive review on this subject could therefore not be adequately prepared from a handful of relevant past studies available.

Wesley (2009) indicated that red clay soils may be referred to as lateritic soils but he cautioned that they should not be confused with the crusty laterite (locally known as murram) itself. The two soils have different geological history, characteristics and local variations but only share a common red color. In this regard, investigative results from the widely researched laterite were considered to be inapplicable here. Thus, the following is an attempt to give a brief summary on past research findings by different researchers on red clay soils:

- Waweru et al. (1998), Foss (1973) and Rolt (1979) are among many researchers who worked on red clays and established that the soils in Kenya are found between altitudes of 1200 and 2500 metres above sea level in areas like Nyeri County with relatively high rainfall, good drainage conditions, fertile soils and dense population densities.
- Wesley (2009), Bruggemann and Gosden (2004), Foss (1973), Coleman et al. (1964), and Terzaghi (1958) found red clay soil to contain 70-100% clay minerals, principally 45-60% halloysite or kaolinite of kaolin group weathered from feldspars and 14-23% iron oxide. Depending on humidity, halloysite may be present as hydrated halloysite or metahalloysite, and iron oxide as goethite (iron hydroxide) or haematite. The remainder is made up of small fractions of goethite, gibbsite, and feldspars like orthoclase, plagioclase and microcline.
- Xue et al. (2020), Elsharief et al. (2013), West & Dumbleton (1970), Coleman et al. (1964), and Newill (1961) found the soil to have 12-82% clay, up to about 30% silt and 2-3% unweathered rock fragments as sand particles, specific surface area of 90-100 m^2/g and clay activity of 0.29-1.14 for soils containing kaolinite. With an exception some Malaysian soils, red clay soil classifies as silt of high plasticity (MH) and often plots abnormally below the A-line on plasticity chart.
- Xue et al. (2020), Chen and Lu (2015), Brink (2015), Bruggemann & Gosden (2004), Waweru et al. (1998), Northmore et al. (1992a), and Keter and Ahn (1986) found red soil to have unusual properties, mainly attributed to the presence of hydrated halloysite and relatively high amount of free iron oxide. This includes high resistance to erosion and high slope stability at abnormally high natural moisture content of 25-97% linked to the porous structure and large specific area. Others are high index properties with specific gravity of 2.75-2.94, liquid limit of 48-118%, plastic limit of 29-58%, plasticity index of 11-74% which is much lower than for sedimentary clay with equal liquid limit.

- Xue et al. (2020), Elsharief et al. (2013), Bruggemann & Gosden (2004), Dixon & Robertson (1970), and Terzaghi (1958) further found red soil to have higher permeability and angle of internal friction and lower compressibility than the corresponding properties for a sedimentary clay with equal liquid limit. The average coefficient of permeability was reported to be about 1×10^{-3} cm/sec in natural state, and 4 $\times10^{-7}$cm/sec in compacted state. The soil has cohesion and an exceedingly high shear strength with small friction angle ϕ' of 11-41° and cohesion c' of about 26-62 kN/m^2.
- Wesley (2009), Waweru et al. (1998), Rolt (1979), Smart (1973), Coleman et al. (1964), and Terzaghi (1958) found red soil to have a well-developed but porous crumb structure chiefly formed of spongy particles cemented together by free iron oxides. The soils have low compressibility, a low free swell of about 0.1-1.7%, low MDD of about 1100-1450 kg/m^3 and high OMC of 34-55% due to the high natural moisture. They difficult to compact due to very poor workability but are still much easier to handle than would otherwise have been with other clays.
- Elsharief et al. (2013), Smart (1973), Coleman et al. (1964), and Terzaghi (1958) further found the red soil to have low bearing strength when wetted. The low soaked CBR value ranging 5-13% defines a relatively stable and good material for subgrade and/or embankment in road construction.
- Kamtchueng et al. (2015), Elsharief et al. (2013), Johannessen (2008), and Rolt (1979) proposed replacement or improvement of the weak natural subgrade provided by red soils. It was recommended that the simplest and most common remedial measure for such subgrade is to cover it, or to partially remove and replace it, with granular material. Nevertheless, it was also established that quality materials suitable for subbase and base are usually scarce along a road corridor and are only available in selected areas.
- O'Flaherty (2002) revealed that granular stabilization of a soil usually takes 10-50% of the better blending material. However, findings by Jjuuko et al. (2011) and Kollaros and Athanasopoulou (2016) on sand stabilization of clayey soils indicated that this could go as high as 60-80% of the mechanical stabilizer.

- Newill (1961) and Beshewski (1963) investigated the suitability of stabilizing Kenya's red clay soil using Portland cement and hydrated lime. Matalucci (1962) also successfully stabilized clay soil with gypsum. The report by Beshewski (1963) is unavailable but the study was reportedly successful and implemented in Mauritius for similar soils. Nonetheless, Newill (1961) stabilized the soil for base to lightly-trafficked roads or LVRs using 5-10% Portland cement and hydrated lime. He found 5% binder content as inadequate in effectively stabilizing the soil for intended purpose and recommended use of 10% of either stabilizer.

2.8 Research Gap

According to Northmore et al. (1992a) and Morin et al. (1971), red clay soils are very common throughout the tropical and sub-tropical regions of the world but the question of quantity and quality tropical materials and their performance in road pavements has never been fully investigated. In this regard, there has been limited in-country and regional research on the soil, coupled with the problem of unpublished and/or inaccessible data held especially by institutions (Gwilliam et al., 2008; Rossiter, 2004; Dixon & Robertson, 1970). This conditions lead to a lack of understanding on the character and adequacy of the soil as road construction material. For this reason, the soil continues to be a major source of concern to many road engineers and also a hindrance to fast road development (Jjuuko et al., 2014; Rolt, 1979).

From the only available study on chemical stabilization of Kenyan red clay soils, Newill (1961) recommended a 10% amount of either Portland cement or hydrated lime for base course of LVRs. Matalucci (1962) established that lime stabilization of a highly plastic 'clay' soil and that lime is effective in clay-gravel mixes due to a pozzolanic reaction between lime and the available silica and some alumina in the clay. He also found that about 2-5% hydrated lime by mass, almost without exception, markedly increased strength and reduced plasticity of such soil. Similarly, O'Flaherty (2002) indicated that much of the observed increase in strength in lime-treated soils is obtained most commonly with about 3% hydrated lime for kaolinitic soils and about 8% for montmorillonitic soils. Thus, the finding by Newill (1961) was on the higher side and not very economical to use.

Wilmot (2006) affirmed that subgrade soil is an integral part of the road pavement structure and that many pavement problems are attributed to subgrade rather than on overlying road pavement layers. In particular, subgrades of red clay soil are inherently weak and seasonally variable in strength and therefore provide one of the major challenges with any pavement design in assignment of the design subgrade strength (Chen & Lu, 2015; DoFEP, 2013; Vorobrieff & Murphy, 2003; Rolt, 1979). However, Wilmot (2006) opined that almost all subgrades, including those with less than 8% CBR, can be improved by stabilization to realize a host of structural, economic and environmental benefits attached to use of stronger and reduced amount of materials. According to Elsharief et al. (2013), and Vorobieff and Murphy (2003), subgrade stabilization is much cheaper than to remove and replace a poor subgrade material. Kollaros and Athanasopoulou (2016) added that subgrade of suitable material also plays an important role in safe and cost effective pavement construction.

Due to lack of recent research in stabilization of Kenyan red clay soil for use as subgrade and subbase of lightly-trafficked roads, the findings by Newill (1961), Matalucci (1962) and O'Flaherty (2002) provided the research gap for this study. In this respect, performance of red clay soil both as subgrade and subbase of these low-volume roads was enhanced by a combination of granular and chemical stabilization. At first, O'Flaherty (2002) and Cook et al. (2001) were uncertain on suitability of granular stabilization for highly plastic soils like red clay soil but Johannessen (2008) later opined that it is more feasible to improve properties of local soils by mixing rather than import materials from far away. This led Kollaros and Athanasopoulou (2016) and Jjuuko et al. (2011) to successfully perform sand stabilization of clayey soils. Pegged on this success with many practical benefits, this study tapped on plasticity, cohesion and particulate size of red soil for bonding and filling into the natural gravel skeleton. with Its high shear strength would further be enhanced by interlock of the angular and tougher gravel that also acts as strong skeleton to the finer soil to produce a a tight, dense and stronger matrix (O'Flaherty, 2002).

According to AustStab (2002), type of binder selected is of primary concern as it has its own unique benefits and also impacts on project cost benefits. Thus, lime was the preferred binder as it is cost effective and very efficient in stabilizing fine and highly plastic soils and even soil-gravel admixtures. It also has long-lasting structural and environmental benefits in a road project. Crack development is a source of weakness and the beginning of pavement failure but lime, unlike cement, produces a more flexible material less susceptible to cracking (Holt, 2010; NLA, 2005).

2.9 Conceptual Framework

According to Jabareen (2009) and Miles and Huberman (1994), conceptual framework is usually a visual presentation like a diagram or flowchart that provides an outline and flow of a study. As a mind map or blueprint, it forms the essence, heart or foundation of a given study by explaining the main things to be studied (concepts or variables) and the presumed relationship among them, and also by providing an interpretative approach to reality rather than offering a theoretical explanation. Nevertheless, the same authors concluded that conceptual frameworks do not provide us with knowledge of hard facts due to their indeterminist in nature and neither do they enable us to predict an outcome!

Figure 2.18 represents the conceptual framework for this study that undertook to stabilize red soil for use in subgrade and subbase layers of a rural road. Thus, the effectiveness of natural gravel and hydrated lime as stabilizers was investigated in a two-stage laboratory process. The first was on the physical and chemical properties of red soil, natural gravel and hydrated lime. The second stage was on select physical and mechanical characteristics of the soil-gravel-lime admixtures as implied by the vertical arrow in Figure 2.18 under ‘independent variables’. The test results or output were then checked for compliance with local specifications for road construction materials.

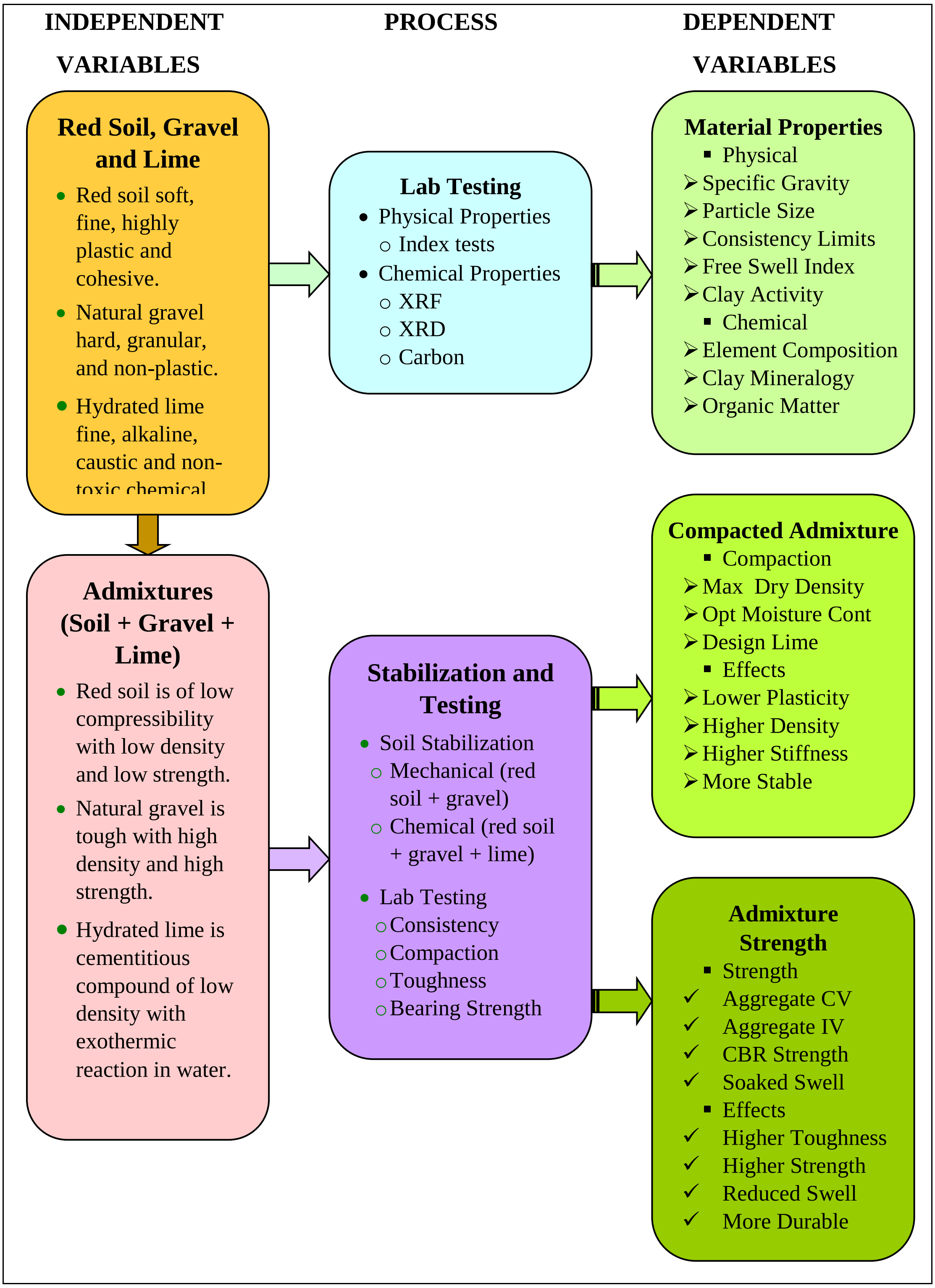

Figure 2.18: Conceptual framework for the study

CHAPTER THREE

MATERIALS AND METHODS

3.1 Research Design

Research design is a practical framework that deals with one of many possible paths used to ‘answer’ the research questions, as outlined in section 1.4. The tool is developed in an iterative process to provide critical information and insights required for the task. Figure 3.1 shows the research design framework for this study.

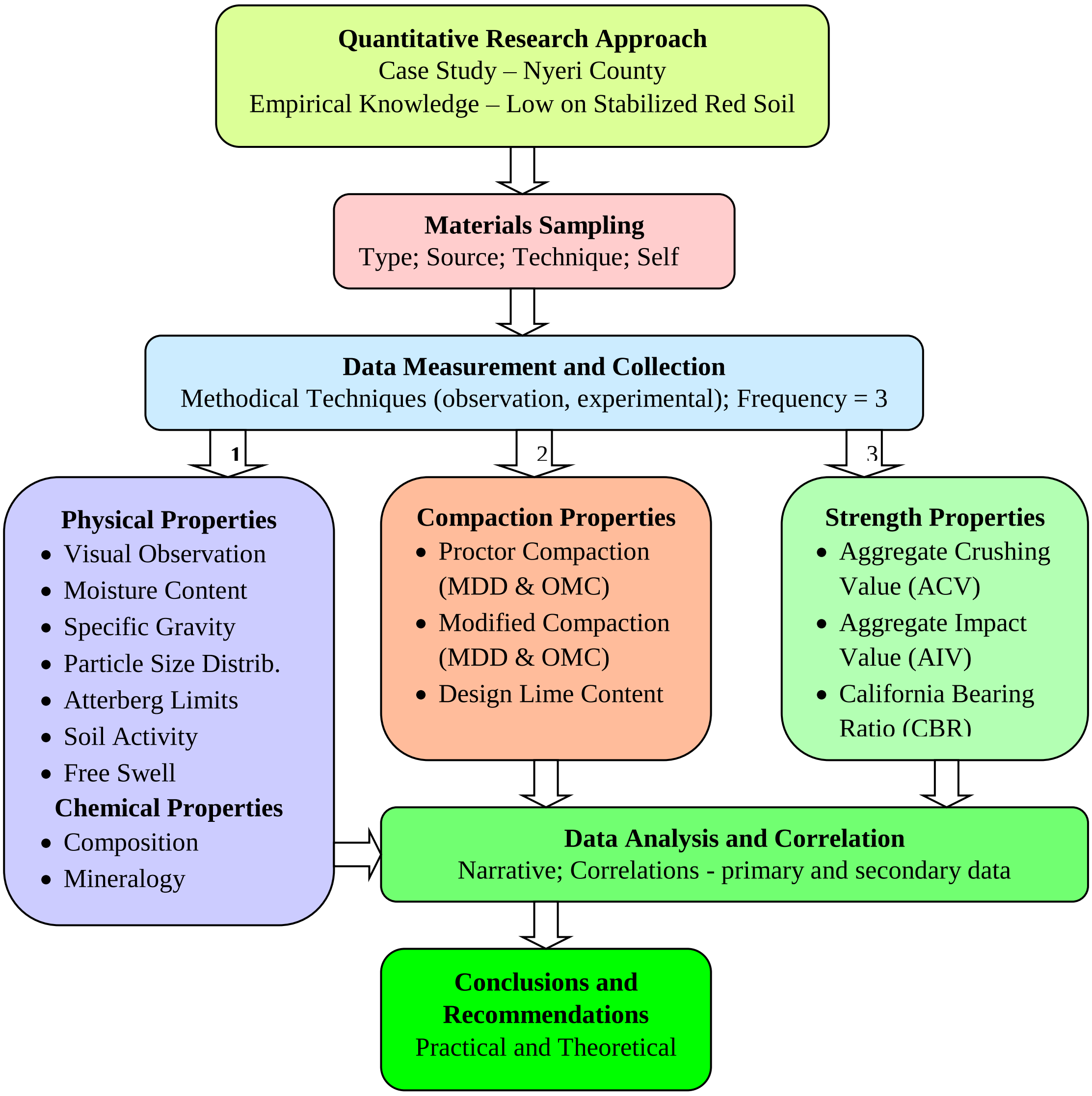

Figure 3.1: Research Design Framework

3.2 Materials Acquisition and Preparation

The red soil used in this study was obtained from a freshly opened building site about 13 km by road south of Nyeri Town in Othaya area of Nyeri County. Similarly, the natural gravel was obtained from a commercial quarry situated about 10 km by road north of Nyeri Town at Nyaribo trading centre. Both the red soil and natural gravel were taken as bulk samples but at different depths of about 1.50m and 20m, respectively, from the ground level. These depths were necessary to ensure that the soil samples were fairly representative of similar red soils and natural gravel. Care was also taken to avoid any contamination by the deleterious organic matter and to preserve field moisture levels, the shape and distribution of particles (Smoltczyk, 2003). The hydrated lime was obtained commercially as Rhino Lime manufactured and packed in 25 kg bags by the Athi River Mining (ARM) of Nairobi. The manufacturer classified the high calcium lime (CL) as CL 60 and recommended it for soil stabilization and road construction, among other uses.

According to Day (2010), it is risky to put blind faith in laboratory tests, especially when they are few in number. Thus, all laboratory test specimens were prepared in triplicate (3 No) mainly from air-dried bulk samples. Other test specimens for soil admixtures were prepared by blending the red soil with natural gravel at step percentages of 0, 20, 40, 60, 80 and 100% tested without or with a predetermined amount of hydrated lime. Distilled and tap water was also used where required.

3.3 Materials Physical and Chemical Characteristics

Information on the physical and chemical characteristics of materials used in the study was obtained from the following two sources:

1) Visual examination during time of sampling in the field, and
2) Laboratory testing.

3.3.1 Visual Examination

According to Wesley (2010), the role of observation in understanding and evaluating residual soils cannot be overemphasized. Some of the following visual examinations were therefore considered during sampling of the red soil and natural gravel (Budhu, 2011; Venkatramaiah, 2006; Smoltczyk, 2003; Whitlow, 1995):

i) The natural color, feel and smell.
ii) Apparent consistency at field moisture content like very soft, soft, firm, stiff, hard, crumby.
iii) Compactness instance loose, dense, or slightly cemented that is used for estimation of field strength.
iv) Degree of weathering in newly exposed soil deposits such as unweathered or slightly-, moderately-, highly- and fully-weathered.
v) Structure or manner of arrangement and state of aggregation of soil grains such as homogenous (one in color and texture), inter-stratified (alternating layers or bands of bedding planes), intact (non-fissured), or fissured (direction, size and spacing).
vi) Texture or apparent particle size, shape and grading of the soil like flaky, well-rounded, rounded or angular, and sub-rounded or sub-angular; well-graded, poorly-graded or uniform/gap-graded.

3.3.2 Laboratory Tests

The laboratory tests were carried out mainly in accordance with procedures outlined in British Standards [BS] 1377 (1990) and BS 1924 (1990) for the natural (RGN) and lime-treated (RGT) soil admixtures respectively. The following types of laboratory tests were conducted:

1) Physical tests that consist of index tests which are the most basic laboratory tests like moisture content, specific gravity, particle size distribution, Atterberg limits, and also clay activity and free swell index (Day 2010).
2) Chemical tests that comprised the XRF and XRD tests for chemical and mineralogical composition.

The free swell index was conducted in accordance with the method proposed by Holtz and Gibbs (1950). The XRF and XRD tests require properly trained personnel, extreme care, and specialized laboratories. They were therefore carried out in Nairobi at the State Department of Mines and Geological and at ICRAF respectively.

3.3.2.1 Moisture Content

(a) Experimental Setup

The moisture content, sometimes referred to as water content, was determined by the conventional oven-drying or definitive method in accordance with Clause 3 of BS 1377-2:1990 and Clause 1.3 of BS 1924- 2:1990. This gravimetric method for moisture content was carried out on small specimen of at least 30g, representative of either field or laboratory condition. The specimen was dried for at least 12 hours using in an electric drying oven capable of maintaining a temperature of 105±5°C.

(b) Data Collection and Analysis

Water content was determined by gravimetric method as the ratio of water in the soil (M_w) to that of the dry soil particles or solids (M_s), as shown in Equation 3.1 thus (BS 1377, 1990):

$$\text{Moisture Content, } w = \frac{M_w}{M_s} = \frac{M_a - M_b}{M_b - M_c} \times 100 \tag{3.1}$$

where M_a = Mass of tin + wet soil specimen (g)

M_b = Mass of tin + oven-dry soil specimen (g)

M_c = Mass of moisture content tin (g)

Moisture content is often as expressed as a percentage and was reported to the nearest 0.1%.

3.3.2.2 Specific Gravity

(a) Experimental Setup

The specific gravity of soil solids was determined by the small 100 ml glass pycnometer (density bottle) method as outlined in Clause 8 of BS 1377-2:1990. The method used 100g of soil particles finer than 2 mm from oven-dried specimen; any particles larger than this were first crushed and passed through the 2 mm sieve. Distilled water was also used but kerosene or white spirit is often preferred where soils contain soluble salts.

(b) Data Collection and Analysis

The specific gravity of the soil solid particles was determined as shown in Equation 3.2 thus:

$$\text{Specific Gravity, } G_s = \frac{M_2 - M_1}{(M_4 - M_1) - (M_3 - M_2)} \tag{3.2}$$

where M_1 = Mass of empty density bottle (g)

M_2 = Mass of bottle + oven-dry soil (g)

M_3 = Mass of bottle + soil + water (g)

M_4 = Mass of bottle full of water (g)

According to Smith (2014), Day (2010) and Craig (2004), the specific gravity is numerically equal to particle density (ρ_s). The two parameters are often reported to two decimal places and are related as represented by Equation 3.3 thus:

$$G_s = \frac{M_s}{V_s \rho_w} = \frac{\rho_s}{\rho_w} \tag{3.3}$$

3.3.2.3 Particle Size Distribution

(a) Experimental Setup

Grading for particle size distribution was performed on the red clay soil and natural gravel only as outlined in Clause 9 of BS 1377-2:1990. Depending on nominal particle size of a soil, the definitive wet sieving and sedimentation methods were applied. The sieve analysis method was generally used on soil particles greater than 75 µm (or 0.075 mm) whereas the hydrometer analysis was used on particles finer than this size. At least 200g-4kg of oven-dried soil was used for sieve analysis and about 50g of soil passing 2mm was used for sedimentation analysis. The sieving and sedimentation methods are illustrated in Figure 3.2.

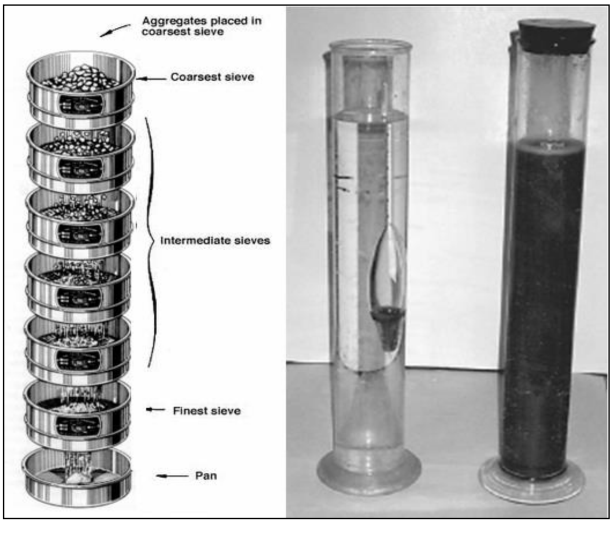

Figure 3.2: Sieve stacking and hydrometer set for soil grain size analyses

(Source: Day, 2010)

(b) Data Collection and Analysis

For sieve analysis, the percent finer or passing by dry weight was calculated using Equation 3.4 as follows (Day, 2010):

$$P = 100 - 100\left(\frac{R_{DS}}{M_S}\right) \qquad (3.4)$$

where P = percent dry soil passing a given sieve size

R_{DS} = cumulative amount of dry soil retained on the sieve (g), as total mass on sieve and all coarser sieves (g).

M_s = dry mass of the soil at the start of the test (g)

In sedimentation analysis, particle size for each corrected hydrometer reading and its corresponding percent finer or passing by initial dry weight of soil specimen was calculated on the basis of Stokes Law that assume spherical soil particles (Day, 2010).

The combined results for sieve and sedimentation analyses – where applicable – were plotted as a smooth curve on the conventional semi-logarithmic graph with the sieve or particle size as abscissa on a logarithmic scale, and the percentages finer as ordinate on an arithmetic scale (Murthy, 2012; O'Flaherty, 2002; Smoltczyk, 2003). Certain grading characteristics proposed by Hazen (1893) and known as the uniformity coefficient (C_u) and the coefficient of curvature (C_c) were then computed using Equations 3.5 and 3.6 thus:

$$\text{Uniformity coefficient, } C_u = \frac{D_{60}}{D_{10}} \qquad (3.5)$$

$$\text{Coefficient of curvature, } C_c = \frac{(D_{30})^2}{D_{60} \times D_{10}} \qquad (3.6)$$

where D_{10}, D_{30}, and D_{60} = particle diameters for soil mass finer than or passing 10, 30 and 60 per cent, respectively.

3.3.2.4 Consistency Limits

(a) Experimental Setup

The consistency limits were formulated by Atterberg (1911) and comprise the liquid limit (LL), the plastic limit (PL), linear shrinkage (LS) together with their related derivatives like plasticity index. All the tests were carried on the fraction of air-dry soil or admixture passing through the 425 µm (or 0.425 mm) sieve. About 300 g of this material was taken and tested in accordance with either BS 1377- 2:1990 or BS 1924-2:1990 for natural and treated specimens, respectively.

i) Liquid Limit

The liquid limit test was determined by the definitive cone penetrometer method as illustrated in Figure 3.3 and in accordance with Clause 4 of BS 1377-2:1990 and Clause 1.4 of BS 1924-2:1990.

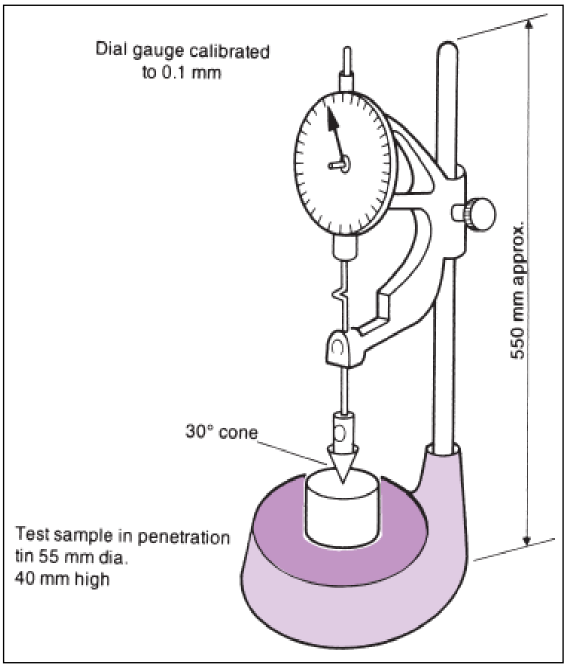

Figure 3.3: Fall cone penetrometer for liquid limit test

(Source: Smith, 2014)

ii) Plastic Limit

The plastic limit test was determined in accordance with Clause 5 of BS 1377-2:1990 and Clause 1.4 of BS 1924-2:1990 by the only conventional method.

Plasticity Index

The plasticity index (PI) was determined in accordance with Clause 5 of BS 1377-2:1990. This is a derived parameter based on the results obtained for liquid and plastic limits.

iii) Linear Shrinkage

The volumetric method for shrinkage limit outlined in Clause 6 of BS 1377-2:1990 was skipped due to personal safety and environmental concerns. The alternative one-dimensional linear shrinkage was determined in accordance with the same clause using a half-cylinder mold similar to one illustrated in Figure 3.4.

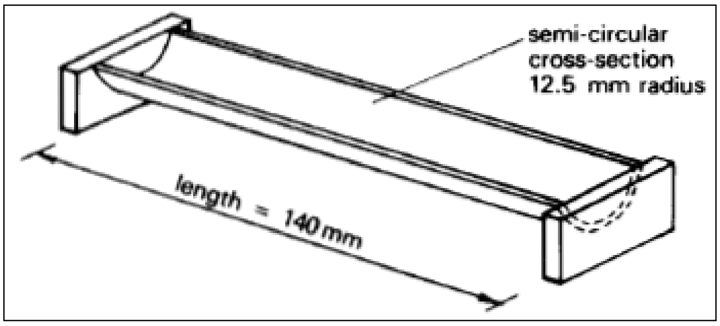

Figure 3.4: Schematic illustration of linear shrinkage mold

(Source: Whitlow, 1995)

Budhu (2011) demonstrated that shrinkage limit of a soil can be estimated from its liquid limit (LL) and plasticity index (PI) using Equation 3.7 thus:

$$\text{Shrinkage Limit, } SL = 46.4\left(\frac{LL-45.5}{PI-46.4}\right)-43.5 \qquad (3.7)$$

(b) Data Collection and Analysis

i) Liquid Limit

Liquid limit was determined from a linear graph as moisture content at 20 mm penetration of a standard cone.

ii) Plastic Limit

Plastic limit of soil was determined as the average moisture content of crumbled pieces.

iii) Plasticity Index

Plasticity index (PI) was computed as the numerical difference between the reported liquid and plastic limits of the soil. It was obtained in percentage form using Equation 3.8 thus:

$$\text{Plasticity Index (\%), } PI = LL - PL \qquad (3.8)$$

iv) Linear Shrinkage

Linear shrinkage, expressed as a percentage, was computed as the ratio of change in length to initial specimen length of soil using Equation 3.9 thus:

$$\text{Linear shrinkage, } LS = \frac{L_0 - L_f}{L_0} \times 100 \tag{3.9}$$

where L_0 = Initial wet length of specimen (= internal length of mold)

L_f = Final oven-dry length of specimen

3.3.2.5 Soil Activity and Free Swell Index

(a) Experimental Setup

i) Soil Activity

Soil activity is an index that represents plasticity of the entire soil mass due to presence of clay minerals (Arora, 2004). It is derived in the manner introduced by Skempton (1953) using both the plasticity index of the soil and its graded percentage by weight of particles less than 2 µm.

ii) Free Swell Index

Fine soils are susceptible to volume change in the presence of water and this depends on amount of fines and type of clay mineral present (Budhu, 2011). Free swell index is an indicator to volume but its determination is not covered by some of the international standards institutions like the British Standards Institution (BSI) and the American Society for Testing and Materials (ASTM). It was determined in accordance with the test procedure proposed by Holtz and Gibbs (1956). About 60 g of soil specimen passing through 425 µm sieve was obtained from air-dry soil. The test was conducted in duplicate using distilled water and kerosene. Exactly 10 cm^3 of the soil specimen was carefully poured into 100 cm^3 graduated glass measuring cylinder filled with either distilled water or kerosene. The final volume of settled soil in either case was measured after 24 hours.

(b) Data Collection and Analyses

i) Soil Activity

The activity of soil was derived from information on particle size and consistency of a soil. It was obtained as the ratio of plasticity index to the percentage by weight of soil particles less than 2μm (or clay), as illustrated in Equation 3.10 thus:

$$\text{Activity, } A = \frac{PI}{Clay(\%)} \tag{3.10}$$

However, the expression for activity was modified by Seed et al. (1962), especially for artificially prepared specimen, to Equation 3.11 thus:

$$\text{Modified Activity, } A_m = \frac{PI}{Clay - n} \tag{3.11}$$

where n = 5 for natural soils, and n = 10 for artificial mixtures.

ii) Free Swell Index

Holtz and Gibbs (1956) defined the free swell index as the difference in volumes of soil in water and in kerosene expressed as a ratio to volume of soil in kerosene. Thus, free swell index is expressed by Equation 3.12 as follows:

$$\text{Free Swell Index} = \frac{V_w - V_k}{V_k} \times 100 \tag{3.12}$$

where V_w = Final volume of soil in water (cm^3 or ml), and

V_f = Final volume of soil in kerosene (cm^3 or ml)

3.3.2.6 Chemical and Mineral Composition Tests

(a) Experimental Setup

i) Chemical Composition

The XRF test was conducted using a handheld automatic machine type 'Bruker, model S1 Titan' in accordance to standard operating procedures. The machine used test specimens of about 20 g in powder form. Thus, red soil and lime were simply passed through an appropriate sieve size while the gravel was converted into a fine powder by passing it through a jaw-crusher and milling machine.

ii) Mineralogical Composition

The XRD test was carried out using a computerized 'D2 Phaser Diffractometer' according to its standard operating procedure. Accordingly, the total and organic carbon for red soil were determined using the Flash 2000 Elemental Analyzer manufactured by 'Thermal Scientific'.

(b) Data Collection and Analyses

i) Chemical Composition

The automatic XRF machine type 'Bruker model S1 Titan' tested and analyzed specimens internally. It therefore displayed the results on a screen and also stored the same in memory for retrieval and printing but only printed copies were provided.

ii) Mineralogical Composition

The complete mineralogical composition results for the red soil were provided in a tabulated form and also as a diffractogram plot.

3.4 Materials Compaction Characteristics

3.4.1 Materials

The test specimens comprised the neat red soil and also soil-gravel admixtures at step percentages of 20, 40, 60, 80 and 100%. Other specimens consisted of soil-gravel admixtures treated with predetermined amount of hydrated lime. Appropriate amount of tap water was also used during mixing.

3.4.2 Laboratory Tests

The laboratory tests comprised the following:

1) Standard compaction test on natural red soil for lack of the more common vibrating hammer,
2) Modified compaction test on the natural red soil and on stabilized soil-gravel-lime admixtures, and
3) Design lime content for lime-treated admixtures through compaction.

3.4.2.1 Compaction Tests

(a) Experimental Setup

Two methods of compaction were carried out in accordance with Clause 3 of BS 1377-4:1990 for the neat soil-gravel admixtures and Clause 2.1 of BS 1924-2:1990 for lime-treated admixtures. Figure 3.5 illustrates the details of standard or Proctor compaction. Modified compaction uses the same mold but a 4.5kg rammer to compact material into 5 layers.

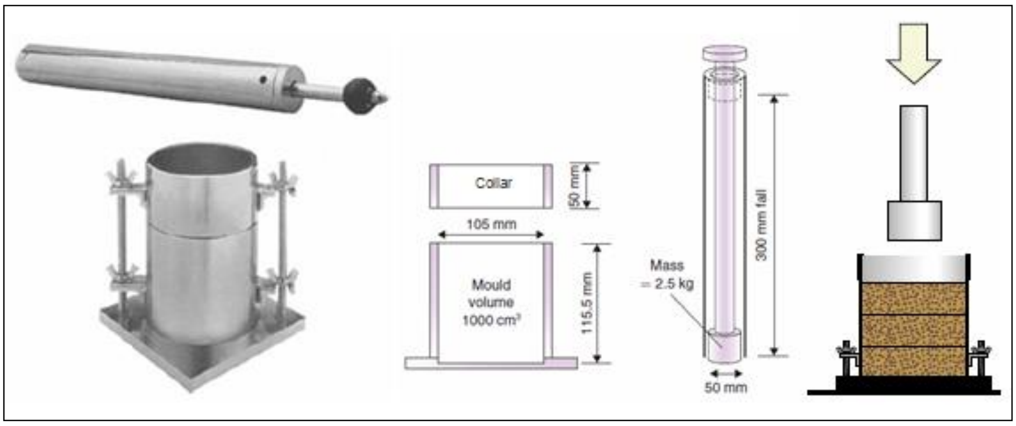

Figure 3.5: Apparatus for and standard method of compaction

(Source: Smith, 2014; Budhu, 2011; Johannessen, 2008)

(b) Data Collection and Analysis

The bulk density of compacted specimen was computed using Equation 3.13 thus:

$$\text{Bulk Density, } \rho_w = \frac{M_2 - M_1}{V} \tag{3.13}$$

where M_2 = Mass of mold + compacted wet specimen (Mg)

M_1 = Mass of empty mold (Mg)

V = Internal volume of mold (m^3)

The dry density was determined using Equation 3.14 thus:

$$\text{Dry Density, } \rho_d = \frac{\rho_w}{\left(1 + {w}/{100}\right)} = \frac{100\rho_w}{100 + w} \tag{3.14}$$

where ρ_w = Bulk density of compacted specimen (Mg/cm^3)

w = Moisture content of specimen

The dry density was plotted against moisture content to obtain a smooth compaction curve for each soil admixture. The readings at the apex of the curve were taken as MDD and OMC for the soil.

3.4.2.2 Optimal and Design Lime Content

(a) Experimental Setup

Based on the premise that a given initial lime content does not necessarily produce the utmost MDD, the procedure proposed by Hudson (1997) was used. Thus, modified compaction tests were conducted about the laboratory MDD for lime treated soil admixtures at ICL ± 2% for the plasticity method as described in section 2.5.2.3.

(b) Data Collection and Analysis

The MDD for each of the three points was determined and the results plotted against the lime content. Strength of soil normally increases with density and hence the lime content was taken as the amount that produced the highest MDD. The design lime content was adjusting by a small margin of 0.5-1% to cater for field tolerances (O'Flaherty, 2002; AustStab, 2002) and was reported to the nearest 0.1%.

3.5 Materials Strength

3.5.1 Materials

The test specimens comprised the neat red soil and also soil-gravel admixtures at step percentages of 20, 40, 60, 80 and 100%. Other specimens consisted of soil-gravel admixtures treated with predetermined amount of hydrated lime. Appropriate amount of tap water was also used during mixing.

3.5.2 Strength Tests

The following mechanical laboratory tests were conducted to determine different types of strength of the materials:

1) The Aggregate Crushing Value (ACV) test on natural gravel,
2) The Aggregate Impact Value (AIV) test on natural gravel, and
3) The CBR on soil-gravel-lime admixtures.

3.5.2.1 ACV Test

(a) Experimental Setup

The ACV test was carried out on clean and oven dry aggregate passing 14 mm sieve and retained on 10 mm sieve in accordance with the procedure outlined in BS 812-110:1990.

(b) Data Collection and Analysis

The mass of aggregate filling the mold was determined. The total mass of crushed material passing 2.36 mm sieve was also obtained. Thus, the ACV value was determined and expressed as a percentage using Equation 3.15 thus:

$$\text{Aggregate Crushing Value, } ACV = \frac{M_2}{M_1} \times 100 \tag{3.15}$$

where M_1 = Mass of oven-dried aggregates filling the mold (g)

M_2 = Mass of crushed fraction passing 2.36 mm sieve (g)

The average ACV was reported to the nearest whole number.

3.5.2.2 AIV Test

(a) Experimental Setup

The AIV test was carried out on clean and oven dry aggregate passing 14 mm sieve and retained on 10 mm sieve in accordance with the procedure outlined in BS 812-112:1990. The AIV testing machine is shown in Plate 3.1.

Plate 3.1: The AIV testing machine

(b) Data Collection and Analysis

The mass of aggregate filling the steel cup was determined. The total mass of impacted material passing 2.36 mm sieve was also taken. Thus, the AIV value was determined and expressed using Equation 3.16 thus:

$$\text{Aggregate Impact Value, } AIV = \frac{M_2}{M_1} \times 100 \qquad (3.16)$$

where M_1 = Mass of oven-dried aggregates filling the cup (g)

M_2 = Mass of impacted fraction passing 2.36 mm sieve (g)

The average AIV was reported to one decimal place.

3.5.2.3 CBR Test

(a) Experimental Setup

The CBR test was carried out on all types of soil admixtures in accordance with Clause 7 of BS 1377-4:1990 for natural soil admixtures and Clause 4.5 of BS 1924-2:1990 for lime-treated soil admixtures. The specimen was allowed to mature and then soaked in water as shown in Figure 3.5, and its swelling monitored.

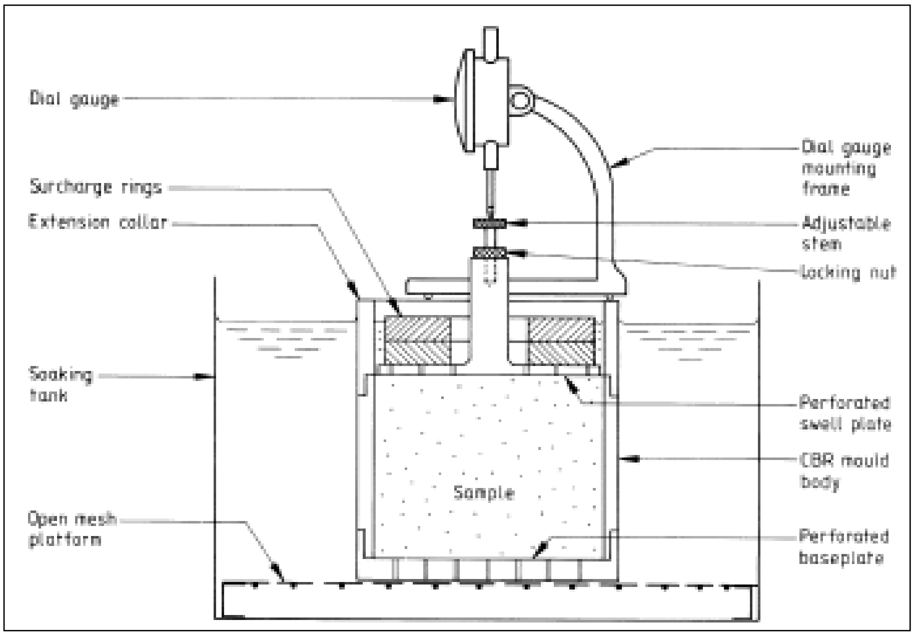

Figure 3.6: Schematic illustration of CBR soaking assembly

(Source: BS 1377, 1990)

(b) Data Collection and Analysis

The forces resisted by, or the load applied on, the compacted material was measured and recorded against specific plunger penetrations. The load was plotted against penetration and corrections made to this stress-strain curve (Jjuuko et al., 2014) where necessary. The arbitrary CBR coefficient was then computed at a penetration of 2.5 mm and 5.0 mm as the ratio of the applied force to a standard force, expressed as a percentage according to Equation 3.17 thus:

$$\text{California Bearing Ratio, } CBR = \frac{A}{B} \times 100 \qquad (3.17)$$

where A = Penetration load on test specimen (kg)

B = Penetration load on standard sample (kg)

The average CBR was computed at each penetration and the higher of the two values so obtained was reported as the CBR value for the material (Blake, 1994). Figure 3.7 illustrates the laboratory setup for testing a specimen for CBR.

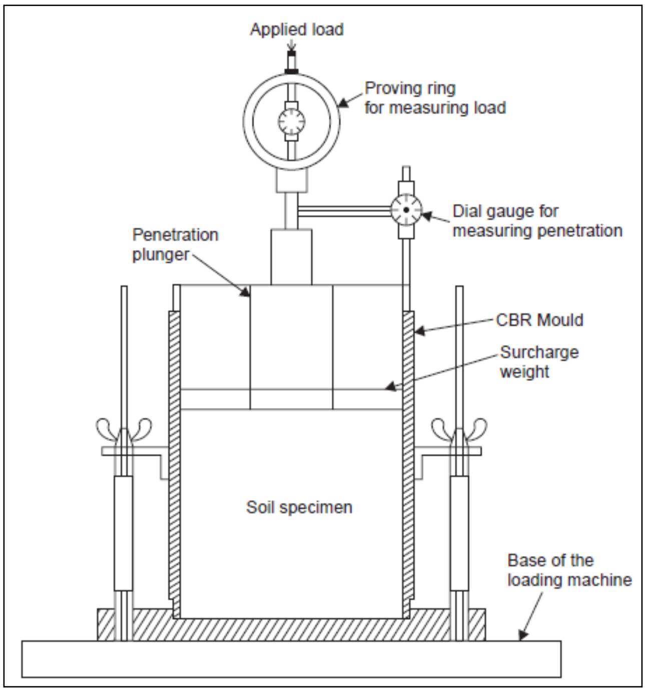

Figure 3.7: Schematic illustration of CBR test assembly

(Source: Budhu, 2011)

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 Material Properties of Red Soil-Gravel-Lime Admixtures

The characteristics of red soil, natural gravel and hydrated lime, as outlined in section 3.3, are presented under the following sub-headings: – Field Observations, Physical and Chemical Properties, and Classification.

4.1.1 Field Observations

Visual examinations, as discussed in section 3.2.1, were carried out, without any instrument or laboratory facility, to identify soil in the field (Das, 2011). Such observations that define the mass characteristics of a natural soil deposit were made on the two residual soil deposits at the point and time of sampling. The observations are summarized in Table A7.3 of Appendix I.

According to Craig (2004) and Smoltczyk (2003), mass characteristics describe the in-situ soil structure and generally indicate the likely behavior of soil. These characteristics can influence the engineering behavior of soil to a considerable extent. More specifically, soil color and smell are very important in identifying different types of soil just like its texture does (Day, 2010; Venkatramaiah, 2006). Nevertheless, mass characteristics of a soil are not frequently used in geotechnical engineering since they only qualify as secondary material characteristics.

4.1.1.1 Red Soil

Based on visual examination, the clay soil had a light reddish-brown color when dry, and a granular appearance with particles less than about 10 mm all that agreed with findings by Gichaga et al. (1987), Coleman et al. (1964) and Newill (1961). The soil was fully weathered, homogeneous in color and texture and also had a firm, dense and unstratified a structure. It lacked smell and vegetative matter when freshly exposed which were indicative of pure and non-organic subsoil. Moreover, the red color was typical of haematite probably derived from basalt rock, pointing to an oxidized soil as presented by Coleman et al. (1964) and Budhu (2011), respectively.

4.1.1.2 Natural Gravel

The natural gravel lacked smell when fresh and was predominantly grey in color but with a greenish tinge. The gravel was moderately-weathered and exhibited an inter-stratified structure and a well-marked columnar jointing and hackly fracture when freshly loosened. Its visual examination revealed a hard and slightly cemented, material composed of coarse-grained and very angular particles. According to Budhu (2011), the grey color and lack of smell suggested an unoxidized and non-organic material respectively. The greenish tinge pointed to origins in the olivine basalts of Laikipian Series (Hinga et al., 2019; Fairburn, 1966; Shackleton, 1945).

4.1.1.3 Hydrated Lime

The hydrated lime was obtained commercially as a fine white powder; the color indicated a calcareous material as suggested by Budhu (2011). From the packaging, it was found to be comparatively lighter in weight than cement. It however proved to be irritating when inhaled especially during mixing with dry soil-gravel admixtures.

4.1.2 Physical Properties

The physical properties were determined experimentally as outlined in section 3.3.2 and constitute the principal material characteristics of a soil. They represent laboratory tests used to identify type soil and are well known as index properties that include water content, specific gravity, gradation and Atterberg limits. These parameters are also used to design foundations like subgrade and to determine the use of soils as a construction material (Budhu, 2011).

4.1.2.1 Moisture Content

Water forms a fundamental part of natural soil and often contain dissolved minerals (Budhu, 2011; Day, 2010) The average moisture content for the bulk samples were obtained as 23.5% for the red soil and 11.7% for the natural gravel. In this regard, the bulk samples were extracted in a fairly dry state while the low values suggest that the soils were largely inorganic. Also known as water content, the moisture for red soil also fell below the high field moisture contents for tropical volcanic soils of 60-100% reported by Rollings et al. (2002).

According to Northmore et al. (1992b) and Day (2010), moisture content of a soil represents the weight of free water contained in a soil sample and can vary from essentially 0% for dry soil up to 1200% for organic soil such as fibrous peat. Water also has a greater effect on the engineering properties of the soil than any other constituent (Budhu, 2011). According to Gianfrancisco and Jenkins (2000) and Terzaghi (1958), water content provides valuable information on possible problems like foundation settlement and compaction of earthworks.

4.1.2.2 Specific Gravity

The specific gravity is a dimensionless number that relates the intrinsic density of solid mineral particles present in a soil to the density of water (Smith, 2014; Day, 2010; Northmore et al., 1992b). The average specific gravity for the red soil, natural gravel and hydrated lime were obtained as 2.79, 2.73 and 2.35, respectively. Specific gravity for blended soil-gravel admixtures (denoted as RGN) ranged from 2.79-2.73 whereas that of soil-gravel-lime admixtures (denoted as RGT) ranged from 2.77-2.71. Figures 4.1 represent the specific gravity results and are also tabulated in Table A7.8 of Appendix IV.

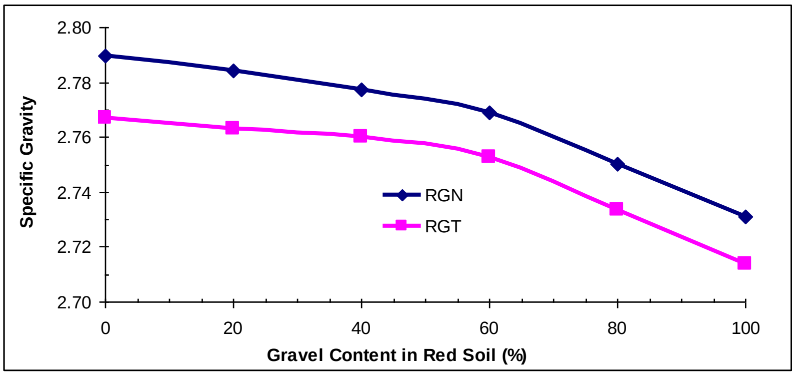

Figure 4.1: Variation in specific gravity with addition of gravel and lime

Specific gravity generally decreased with increasing gravel content and with addition of hydrated lime in soil-gravel admixture. It, however, declined markedly after about 60% gravel content as the soil-gravel-lime admixtures became more gravelly in character. Results for majority of soil-gravel-lime admixtures were above the normal range of 2.55-2.75 spelt out by Smith (2014) and Day (2010) for most soils. This range generally indicates an organic matter and metallic material at its lower and higher ends, respectively. With least value of 2.71, all the admixtures classified as metallic materials. Specific gravity of 2.79 for red soil agreed with values reported by Coleman et al. (1964) and Newill (1961) for similar soils. The specific gravity for hydrated lime of 2.35 lay slightly above its normal range of 2.0-2.2 given by Little (1995) but probably due to manufacturing and/or laboratory limitations.

According to O'Flaherty (2002), specific gravity depends mainly on soil mineralogy but is independent of unbound moisture content and voids It. is numerically equal to particle density which is defined as the average mass per unit volume of the solid particles with units as Mg/m^3. It is also required for design and construction purposes particularly in the computation and interpretation of certain test results like sedimentation and voids ratio, respectively (Blake, 1994; Northmore et al., 1992b).

In practice, specific gravity of 2.65 for quartz as the most abundant type of soil mineral is commonly used but a slightly higher value of 2.70 is often assumed for common clay particles. Kamtchueng et al. (2015) added that specific gravity is proportional to the degree of laterization of tropical soils and can be applied to determine the extent of chemical weathering of such soils. Thus, the red clay soil with a higher specific gravity was apparently more evolved or weathered than the natural gravel, as detailed in section 4.2.2.6. Moreover, Gidigasu (1974) indicated that specific gravity is useful in rating the field performance of lateritic aggregate as road construction material. Thus, soil-gravel-lime admixtures with specific gravity above 2.85 were rated as 'excellent', those within 2.85-2.75 bracket as 'good', and those within the range of 2.75-2.58 including natural gravel as 'fair'. Nevertheless, existing design standards are silent on possible specific gravity values for different local road construction materials.

4.1.2.3 Particle Size Distribution

The particle size distribution of a soil is best presented as a graphical curve that actually defines the relative amounts of clay, silt, sand and gravel within the soil (Hazelton & Murphy, 2007; Craig, 2004; Smoltczyk, 2003). According to Sherwood (1967), results are then reported in terms of percentage clay, silt, sand and gravel particles in the soil. As shown in Table A7.3 of Appendix I, red soil comprised 34% clay, 20% silt, 44% sand and 2% fine gravel that described the soil as 'silty clayey SAND with some fine gravel'. Similarly, the natural gravel consisted of 1% silt, 10%

sand and 89% gravel described as 'sandy GRAVEL with trace silt'. Moreover, slope and shape of the grading curve is described by certain geometric values known as uniformity (C_u) and curvature (C_c) coefficients. The effective size (d_{10}) was established for the gravel only as 2 mm and hence its C_u and C_c coefficients were obtained as (>5) at 9.2 and within 1-3 at 2.3, respectively, for a well-graded soil. The grading curves for the red soil and natural gravel are presented in Figure 4.2 and evaluation of gravel as an aggregate in Figure A7.1 of Appendix I.

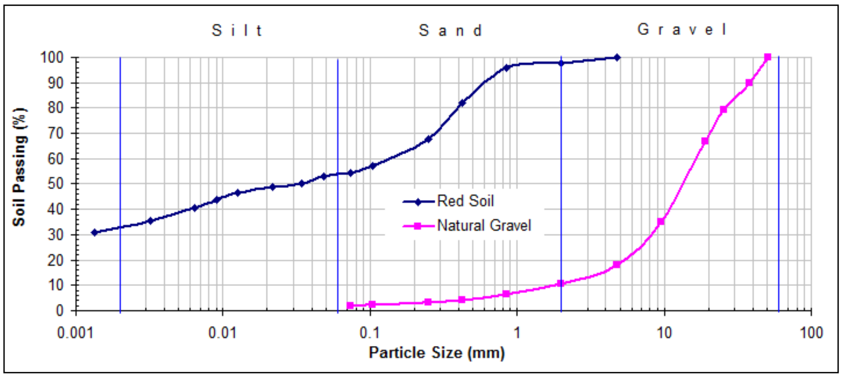

Figure 4.2: Combined grading curves for red soil and gravel

A grading curve is not only useful in itself as a means of describing a soil but it is also closely related to many of its geotechnical properties like textural classification, drainage and compaction (Budhu, 2011; Hazelton & Murphy, 2007; Smoltczyk, 2003). From the shape of grading curves and/or coefficients, both the red soil and natural gravel were well-graded. Based on the coefficients, the gravel qualified for use as natural subbase material as specified by MoRPW (1986). Similarly, the soil satisfied the requirements for maximum particle size of 0.5-10 mm given by MoTC (1987). Both red soil and natural gravel also met the maximum content of particles finer than 0.075 mm of 50 and 40%, respectively. Nonetheless, the 34% clay content of red soil was below the 50-90% reported for similar soils by Sherwood (1967), Newill (1961) and Terzaghi (1958). They explained that such low clay content and resultant high silt or sand contents are likely due to errors in the test and not due to aggregation of soil particles which is eliminated by chemical treatment before soil testing. At over 80% fines passing 425 µm, the soil far exceeded the specified minimum 15% required to justify its stabilization with lime according to MoTC (1987) and Newill (1961). Moreover, the maximum size of 50mm for natural gravel was beyond the 10-15 mm recommended for lime treated materials (MoTC, 1987). This necessitated the removal of the more coarse particles during soil stabilization to reduce the amount of sand and fines required to fill the voids in the red soil-gravel admixture, as reported by Frempong & Tsidzi (1999).

Ejeta et al. (2017) and O'Flaherty (2002) advanced the view that too great an emphasis should not be placed on achieving the ideal gradation with regard to blending combinations but rather on obtaining a mixture that is sufficiently dense to meet stability needs whilst maximizing the use of readily available low-cost soils. In this respect, the grading curves for red soil and natural gravel served as the respective lower and upper limits or grading envelopes for all soil-gravel admixtures. The red soil served as a filler and binder to the cohesionless natural gravel, and aided in reduction of the voids and permeability and in an increase in the density and strength of the soil-gravel admixtures (Budhu, 2011; Johannessen, 2008; O'Flaherty, 2002). The gravel also improved bearing strength of the admixtures through increased internal friction and particle interlock.

4.1.2.4 Consistency or Atterberg Limits

Atterberg (1911) proposed the consistency limits which generally represent the plasticity characteristics of the whole soil, referred to as 'plastic modulus', and are considerably affected by moisture content (Day, 2010; Powrie, 2004). Table 4.1 represents results for all soil consistency limits, PI and its associated ICL as discussed in section 4.3.2.

Table 4.1: Results for consistency limits and initial lime demand

Gravel Cont. (%)	**Natural Admixtures – RGN (%)**						**Treated Admixtures - RGT %)**				
	LL	PL	PI	LS	*PM	ICL	LL	PL	PI	LS	*PM
0	74.3	44.2	30.1	15.7	2472	4.60	59.1	47.0	12.0	8.2	986
20	67.3	42.2	25.1	12.4	1669	3.91	54.6	44.0	10.6	7.1	704
40	60.2	38.2	22.0	11.3	1122	3.31	50.4	41.2	9.2	6.2	469
60	53.5	33.8	19.7	10.2	697	2.90	46.4	38.2	8.1	5.1	287
80	46.8	28.8	18.0	9.5	358	2.50	41.5	34.3	7.2	4.1	143
100	41.8	24.2	17.6	8.6	76	2.38	37.9	31.3	6.6	3.3	28

Key: RGN = Natural red soil and gravel admixtures

RGT = Lime treated red soil and gravel admixtures

PM = Plastic Modulus = Plasticity Index (PI) × Percent passing 0.425 mm sieve (estimated*)

Consistency of a soil is defined as the degree of its firmness and is therefore a measure of its resistance to mechanical deformation or flow (Murthy, 2012). The consistency limits provide a valuable tool for evaluating, characterizing or comparing and classifying residual soils, and also for indicating the amount and type of clay mineral present (O'Flaherty, 2002; Wignall et al., 1999). The plasticity index (PI), in particular, is widely used as a quality-measuring tool for pavement materials.

i) Liquid Limit

The liquid limit is the percentage water content at which the soil flows under its own weight like a viscous mud, with very little strength (Budhu, 2011; Das, 2011). As shown in Table 4.1, the liquid limit values decreased with gravel content and lime ranging from 74.3-41.8% for the RGN soil admixtures and from 59.1-37.9% for the RGT soil admixtures. The liquid limit for red soil of 74.3% was within the typical range of 40-100% and above given by Murthy (2012) and O'Flaherty (2002) for clay soils of volcanic origin. Similar soils within Kenya's central highlands had recorded liquid limit in the range of 65-105% as reported by Sherwood (1967), Coleman et al. (1964), Newill (1961) and Terzaghi (1958). They found the index properties of red clay soil to be a function of mixing time, and increase with prolonged mixing due to break down of all aggregations, and to the consequent increase in clay content.

ii) Plastic Limit

Plastic limit is the moisture content at which a soil becomes too dry to exhibit plastic behavior – the ability to be rolled and molded without breaking apart – when it becomes friable and crumbly (Budhu, 2011). The respective plastic limit values for the RGN and RGT soil admixtures shown in Table 4.1 ranged from 44.2-24.2% and from 47.0-31.3%. The plastic limit for red soil at 44.2% fell within the 35-70% range for similar red clay soils worked to mixing effect for the liquid limit (Sherwood, 1967; Coleman et al., 1964; Terzaghi, 1958). However, the red soil used in this study did not pose any difficulty in determination of the plastic limit as reported by Newill (1961) for the Sasumua red soil where the thread of soil first crumbled at diameters of about 12 mm but finally crumbled at diameters of 3 mm with continued rolling of the crumbled portions. The plastic limits reduced progressively and the various soil admixtures gradually converted into the more workable, stronger and crumbly silt soil but without reaching the non-plastic state as also found by Jawad et al. (2014) and Morin et al. (1971).

iii) Plasticity Index

Plasticity index is defined as the range of water content over which the soil exhibits plastic behavior and deforms plastically; it is equal to the numerical difference in liquid limit and plastic limit (Craig, 2012; Budhu, 2011). The derived PI values for the RGN ranged from 30.1-17.6% as shown in Table 4.1 and also illustrated in Figure 4.3. The plasticity index for RGT soil admixtures ranged from 12.0-6.6% and was linearly related to the amount of clay fraction present as reported by Skempton (1953). The PI for natural and treated red soil of 30.1% and 12% compared well with values of 18-56% and 10%, respectively, reported by Sherwood (1967), Newill (1961) and Terzaghi (1958) for similar clay soils.

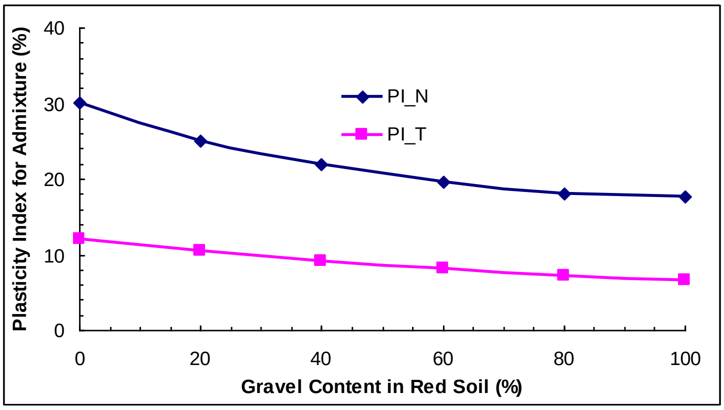

Figure 4.3: Variation in PI with gravel and lime stabilizers

Key: PI_N = Plasticity Index of natural red soil and gravel admixtures

PI_T = Plasticity Index of lime treated red soil and gravel admixtures

Plasticity of a soil is the property that allows it to undergo rapid deformation without rupture, volume change or elastic rebound. It is notable that existing design standards address the consistency of a soil in terms of its PI and not the liquid and plastic limits. The addition of gravel and lime to red soil effectively reduce the PI since this reduced the liquid limit but increased the empirically established plastic limit (Little, 1995). It was evident then that the PI decreased progressively with increasing gravel content and added lime. The PI also dropped significantly upon the addition of lime to the soil-gravel admixtures, as it was expected, but this change reduced with increasing gravel content. In general, the change in PI was more pronounced in soil-gravel admixtures, and smaller and more regular in lime-treated admixtures. The results also demonstrated that the hydrated lime was much more effective than natural gravel in stabilizing the red soil for plasticity.

PI is an indirect measure of clay content and type of clay mineral present in a soil; it is therefore used to classify clays, to roughly approximate clay content, and to estimate the strength of a cohesive soil expressed as its CBR (O'Flaherty, 2002; Sherwood, 1967). This is one of the most common and widely used soil indices, especially in quality control of road construction materials, to predict the behavior of a fine soil. According MoRPW (1986) the maximum PI for clayey and silty sands as well as natural gravel used for subbase in wet areas (with mean annual rainfall exceeding 500 mm) should be 5-12% and 15%, respectively. Thus, only the RGT admixtures with PI of less than 15% satisfied these plasticity requirements.

iv) Linear shrinkage

Linear shrinkage is the percentage decrease in the length of a bar of soil dried in an oven from the liquid limit (Hazelton & Murphy, 2007). From Table 4.1, the linear shrinkage values for the RGN and RGT admixtures ranged from 15.7-8.6% and from 8.2-3.3% respectively. The RGN admixtures values fell outside the normal range of 2-8% proposed by Budhu (2011) whereas the RGT admixtures were within the range. The results also demonstrated that linear shrinkage behaved in a manner similar to that of the PI upon the addition of the two red soil stabilizers. He also reported that the shrinkage limit is useful for the determination of the swelling and shrinking capacity of soils. In this regard, the potential volume change of soils can be deduced from its PI and linear shrinkage as presented in Table A7.8 of Appendix IV. As a result, majority of the blended soil-gravel admixtures and natural gravel classified as materials of 'low' volume change potential. Notably, red clay soil classified as a soil of medium volume change potential and therefore requires cautious use as a foundation for small structures including road pavements.

4.1.2.5 Clay Activity and Free Swell Index

Skempton (1953) used the term 'activity' to quantify the plasticity of the clay fraction of a fine grained soil. He indicated that activity is a function of plasticity index and the quantity of colloidal clay particles present in a soil. To avoid possible effect of lime on colloidal activity (A_c), the parameter was determined for the RGN admixtures only. Results for the soil admixtures ranged between 0.37 and 2.1 for the natural red soil and gravel, respectively, as illustrated in Figure 4.4 and tabulated in Table A7.8 of Appendix IV. Thus, there was a marked increase in activity of clay fraction beyond 40% gravel content in RGN admixtures.

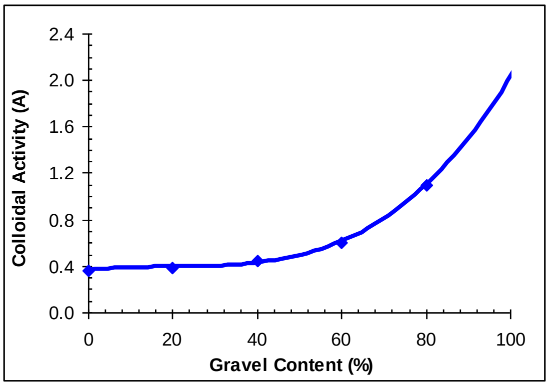

Figure 4.4: Variation in colloidal activity of natural soil and gravel admixtures

Clay activity is useful in description and identification of clay minerals, and as a good indicator of potential shrink-swell problems associated with expansive clays (Das, 2011). In this respect, the clay fraction in RGN admixtures with up to 70%, 70-85% and over 85% gravel content was described as 'inactive', 'normal' and 'active', respectively, as proposed by Skempton (1953) and outlined in Table 4.2.

Table 4.2: Clay activity and relationship with clay minerals and free swell

Clay Descriptioı	Activity, A_c	Clay Minerals	Activity, A_c	Free Swell (%)
Inactive	<0.75	Kaolinite	0.35-0.5	5-60
Normal	0.75-1.25	Illite	0.5-1.3	15-120
Active	1.25-2	Ca-Smectite	0.5-2.0	45-145
Very Active	>6	Na-Smectite	4-7	1400-1600

(Source: Budhu, 2011)

Skempton (1953) held that clay activity remains constant when clay of a particular mineralogy is mixed with coarser material, like the natural gravel. Moreover, he quoted an activity of 0.33-0.46 for kaolinite whose presence was suggested by the low activity of 0.37 for the ‘inactive or inert’ red clay soil. According to West and Dumbleton (1970) and Wesley (2009), this is common in residual soils derived from igneous rocks like basalt where the soils are likely to be fairly coarse-grained with a small clay fraction as was the case reported in section 4.2.2.3. It is also associated with the presence of poorly ordered kaolinite along with clay-sized but non-clay minerals of gibbsite and goethite in the clay fraction. Conversely, natural gravel had an ‘active’ clay fraction which gave an activity of about 2.

The results for free swell index, also known as differential free swell index, for the RGN and RGT soil admixtures ranged from 0-2.4% and from 4-12.7%, respectively. The free swell index for red soil and natural gravel were 0% and 2.4% respectively. The free swell index results are presented in Figure 4.5 and tabulated in Table A7.8 of Appendix IV. The results show that the values for RGN admixtures were moderate and fairly low. However, the values for RGT admixtures increased tremedously upon the addtion of lime with a peak of 12.7% at about 67% gravel content. This effect of lime suggested a reduction in density of soil admixtures which was also reflected as a drop in the specific gravity of lime-treated admixtures presented in Figure 4.1 under section 4.2.2.2.

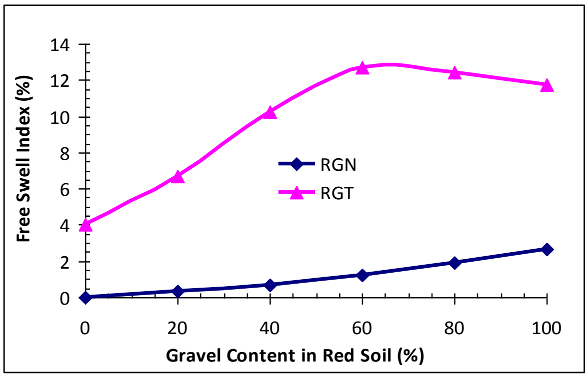

Figure 4.5: Variation in free swell index with gravel content and lime

Free swell index is an indicator of possible volume change of a soil during shrinking or swelling. According to Skempton (1953), the significant volume change of a soil is a function of colloidal activity and depends on amount and type of clay present. The 0% free swell index for red soil compared well with its free swell of 0.1-1.7%

reported by Waweru et al. (1998). Holtz and Gibbs (1956) reported that soils with a free swell value below 50% seldom exhibit appreciable volume change even under light loadings like road pavements. Thus, all the soil-gravel admixtures were considered to be non-expansive road construction materials.

4.1.2.6 Soil Composition and Mineralogy

According to Jawad et al. (2014) and Little (2009), mineralogical and chemical composition of a soil is used to assess its CEC and reactivity with the selected stabilizer. This creates cementitious materials that effectively improve workability, compressibility, strength and durability of the soil. However, the presence of organic matter in a soil, even in small proportions, is likely to inhibit the soil reactivity required for gain in strength by binding a substantial amount of water into the soil.

i) Chemical Composition

The mean chemical composition results obtained by means of XRF method are tabulated in Table 4.3 for the red clay soil, natural gravel and hydrated lime. The full chemical analyses results are also shown in Table A7.4 of Appendix II.

Table 4.3: Chemical composition of the red soil, gravel and hydrated lime

Determination	Mean Percentage (%)		
	Red Soil	Natural Gravel	Hydrated Lime
Silica, SiO_2	**40.672**	**50.748**	**-**
Alumina, Al_2O_3	26.649	13.413	1.858
Iron, Fe_2O_3	25.396	13.727	0.492
Titanium, TiO_2	4.018	3.526	0.058
Sulphide, SO_2	0.713	0.379	0.654
Phosphite, P_2O_2	0.693	1.382	1.180
Manganese, Mn_2O_4	0.451	0.216	0.023
Calcium, $Ca(OH)_2$	0.429	13.622	**95.371**
Potassium, K_2O	0.226	2.494	-
Zirconium, Zr	0.200	0.070	0.028
Cerium, Ce	0.146	-	-
Chloride, Cl	0.078	-	0.110
Total	99.671	99.578	99.774
Total Carbon	0.347	-	-

(Courtesy: Department of Mines and Geology, Nairobi)

Both red soil and natural gravel consisted of silica as the principal compound at 40.7% and 50.7%, respectively, together with other oxides. The chemical composition of red soil generally compared well with that of Nyeri clay as presented under section 2.3.1. That of the hydrated lime was simply 95.4% calcium hydroxide and 4.6% of other oxides. The calcium hydroxide was equivalent to 72.5% calcium oxide and it was also noted that the natural gravel and red soil had registered 13.6% and a mere 0.4% of additional calcium oxide respectively. A trace of organic carbon was also found in the red soil at 0.35% just like the content reported by Coleman et al. (1964), Newill (1961) and Terzaghi (1958) for the Nyeri and Sasumua red clays. The geochemical index or S-S ratio for red soil was 0.78 (<1.33) and 1.87 (range 1.33-2.0) for natural gravel. Thus, the red clay soil classified as a highly weathered laterite soil after Rossiter (2004) criteria, and in agreement with Nyeri red clay investigated by Coleman et al. (1964). Similarly, the natural gravel classified as partially weathered lateritic gravel. Non-lateritic soil has an S-S ratio above 2.0.

According to Robinson and Thagesen (2004), and based on silica content, the red soil existed predominantly as quartz whose specific gravity is 2.65. With silica content of less than 55%, the natural gravel classified as 'basic' rock which was extrusive and fine-grained basalt of the Laikipian Series. Sherwood (1967) pointed out that free iron oxide acts as a cementing agent while carbonates and organic matter may also bind the soil particles together. However, there were no carbonates in red soil and organic carbon content was very low. Thus, high iron oxide content was responsible for aggregation of the red clay soil. Organic carbon is also responsible for high plasticity, shrinkage, compressibility, deformation and low strength of soils and can inhibit the reaction between calcium and the clay mineral surface (Little, 1995). The low organic carbon content at 0.35% was strikingly similar to that of Sasumua clay reported by Newill (1961). It was well below 3% for subgrade, 2% for subbase and 1% for base specified by MoTC (1987) and MoRPW (1986). It was also below the critical level of 1-2% proposed by O'Flaherty (2002) and Little (1995); 1% maximum is recommended in lime stabilization (Little, 2009). According to specialist interpretation, the low organic content meant that the red soil lacked any form of lime treatment – agricultural or otherwise – in the recent past.

ii) Mineralogy

Soil mineralogy is a particularly decisive factor that influences the moisture characteristics and mechanical behavior of soils (Smoltczyk, 2003; Coleman et al., 1964). According to Little (1995), the XRD spectrum that is used to identify clay minerals is a function of the angle of reflection of an x-ray beam. In effect, the XRD analysis of the red soil was represented by the diffractogram shown in Figure A7.2 of Appendix II whose interpretation gave the percentage mineral composition of the red soil as detailed in Table 4.4. As already indicated by colloidal activity of the red soil, the results again confirmed kaolinite the predominant clay mineral in the soil. This also enhanced report by local community that the soil had a history of pottery-making which highly depend on kaolinitic clay. The feldspars microcline (or orthoclase) and albite, as well as quartz and hematite followed closely in that order.

Table 4.4: XRD results for the red soil

Mineral	Amount (%)	Specific Gravity*	Remark*
Kaolinite	47.9	2.61-2.66	Silicate clay mineral of low activity
Microcline	18.4	2.54-2.57	Feldspars, the second most common silicate clay minerals
Albite	12.6	2.62-2.76	
Quartz	11.6	2.65	Most common silicate clay mineral
Hematite	9.5	5.2-5.3	Cause of reddish-brown color in soil
TOTAL	100	-	-

(Courtesy: International Centre for Research in Agro-Forestry [ICRAF], Nairobi; *Day, 2010)

Rollings et al. (2002) reported that mineralogical analysis of tropical red soils often finds a high content of allophane or halloysite clay minerals but these were missing in the red soil under study. However, its mineralogical composition was largely in agreement with findings by Bruggemann and Gosden (2004), Dixon and Robertson (1970) and Terzaghi (1958) that red clay soils consist principally of halloysite, silica as ordinary quartz, ferric oxide as hematite and also small quantities of kaolinite, gibbsite, goethite, and certain feldspars. As observed by Northmore et al. (1992b), Coleman et al. (1964) and Newill (1961), the XRD plot lacked a peak definition of minerals – a phenomenon common with red soils found in Kenya. This also infers a poorly-ordered (or non-crystalline) structure indicative of kaolinite or halloysite, the two similar clay minerals expected to weather from feldspar minerals. Nonetheless, the mineralogical results for red soil agreed with the findings by Terzaghi (1958) that the soil consisted of 70-100% clay minerals.

Jawad et al. (2014) and O'Flaherty (2002) reported that kaolinite is a stable but relatively inactive clay mineral. Kaolinitic soils are therefore characterized by low expansion potential, CEC, and low pozzolanic strength development irrespective of the lime type, content, or length of curing. Such soils are also sensitive to remolding and drying, and also difficult to compact in the field (Kamtchueng et al., 2015; Gidigasu, 1974). Microcline and albite are alkaline feldspars of potassium and sodium, respectively. Feldspars are a large family of aluminosilicate minerals very common in extrusive igneous rocks like basalt and granite and make up about 80% of the Earth's crust. Albite is the pivot mineral of two series of feldspars known as anorthoclase and plagioclase, as illustrated in Figure A7.3 of Appendix II. Microcline and albite are fairly hard and are widely used in manufacture of glass and ceramics and as fillers in paints, plastics and rubber.

According to Mitchell and Soga (2005), prior knowledge of what minerals are in a soil provides intuitive insight as to its behavior. They added that mineralogy is the primary factor controlling the size, shape, and properties of soil particles that, in turn, determine the possible ranges of physical and chemical properties of any given soil. Moreover, clay and organic matter in a soil usually influence properties in a manner far greater than their abundance. In this respect, the consistency, chemical reaction, cementation, workability and bearing strength of soil-gravel admixtures would all be affected. In particular, the CEC of kaolinite would determine the reactivity of clay and more so the pozzolanic reaction upon addition of hydrated lime. The presence of deleterious matter in the form of organic or sulphate contents would retard hardening and reduce the strength gain in lime-stabilized admixtures as reported by many authors like Jawad et al. (2014), Guyer (2011), Little (2009) and O'Flaherty (2002). This was not expected to happen as organic matter was very low in content or absent.

4.1.3 Soil Classification

A soil classification system is a universal language based on soil grading and plasticity that all geotechnical engineers understand and use to eliminate the ambiguity in communicating and comparing the characteristics of a soil (Budhu, 2011; Craig, 2004). According to Das (2011), there are many soil classification systems but the USCS is used worldwide in geotechnical engineering, and the AASHTO system mostly in road pavements. These two systems are presented as Tables A7.1 and A7.2 in Appendix I. The USCS classified the red soil as sandy SILT of high plasticity (MH) and the natural gravel as well-graded gravel (GW). Similarly, the AASHTO system classified the red soil as clayey soil (A-7-5), and the natural gravel as clayey sand and GRAVEL (A-2-7). The AASHTO system also rated the red soil and gravel as ‘poor’ and ‘good’ subgrade material respectively.

Fine cohesive soils like the red soil are also classified using a plasticity chart as shown in Figure 4.6. The A-line represents the boundary between clays (C) and silts (M), and the U-line represents the uppermost limit that provides a good check for erroneous test data (Day, 2010). Thus, the consistency of fines in all soil-gravel admixtures plotted normally under the U-line as illustrated in Figure 4.7. Nonetheless, the RGN admixtures plotted at a higher level and a wider scatter than the RGT admixtures. This difference reflected on the effectiveness of gravel and lime treatment on red clay soil. The red soil with a liquid limit of 74.3% and a PI of 30.1% also plotted below the A-line and further to the right as SILT of high plasticity (MH). This corresponded to findings by Foss (1973) and Day (2010) for other kaolin clays and indicated a silt-like character of red clay soil.

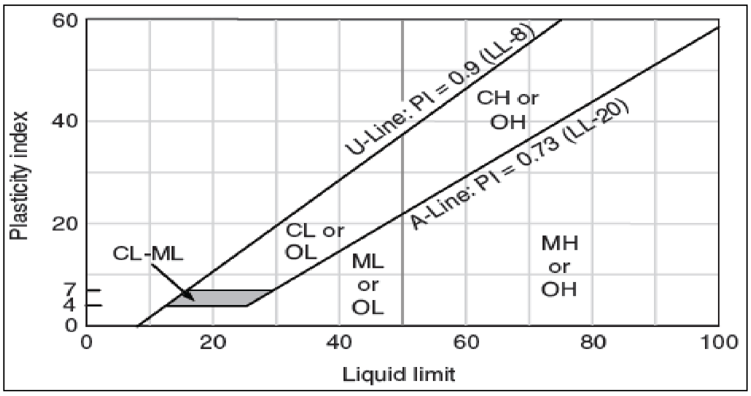

Figure 4.6: Typical plasticity chart

(Source: Das, 2011)

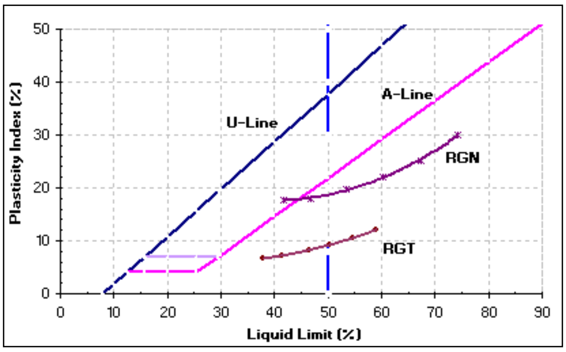

Figure 4.7: Plotting of soil admixtures on the plasticity chart

(Source: Author)

The red soil is often described as a clay soil but plotted below the A-line which is definitely abnormal but not strange for many fine tropical soils developed from igneous rocks like basalt (Bruggemann & Gosden, 2004; Rollings et al., 2002; Waweru et al., 1998; West & Dumbleton, 1970; Terzaghi, 1958). In this respect, Wesley (2009) asserted that many residual soils behave like silty clays for engineering purposes, and rightly fall into the category of silty clay on plasticity chart. Additionally, the AASHTO classification system rated the red clay soils as poor subgrade material. Nevertheless, Little (1995) opined that MH and CH soils are among those that are potentially capable of being stabilized with lime.

A modified plasticity chart may also be used to quickly and inexpensively identify mineralogy of a clay soil as illustrated in Figure 4.8. The red soil plotted below the A-line, and was confirmed as predominantly kaolinite for the third time.

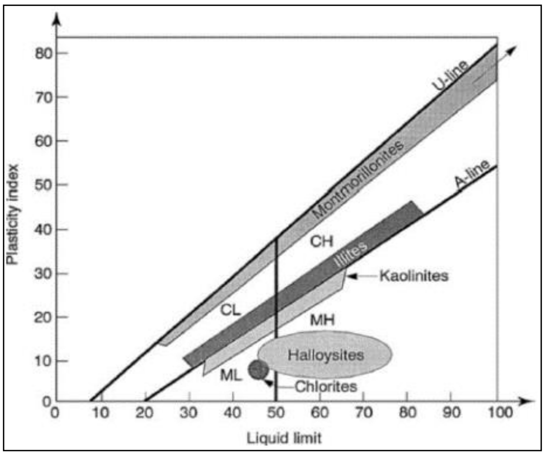

Figure 4.8: Location of common clay minerals on plasticity chart

(Source: Day, 2010; Holtz & Kovacs, 1981)

4.2 Compaction Characteristics of Red Soil-Gravel-Lime Admixtures

Compaction, as outlined in section 3.4.2, is the mechanical process of increasing the density of a soil, by packing the solid particles closer together to reduce volume of air in the soil, in order to improve essential engineering properties such as strength, stability and permeability (Craig, 2004; O'Flaherty, 2002). According to Budhu (2011), two related laboratory compaction procedures are used to investigate the maximum dry density and optimum moisture content of a soil. These are the Proctor or standard compaction, and the modified compaction.

4.2.1 Standard and Modified Compaction

The average results for the standard compaction of natural red soil was a MDD of 1255 kg/m^3 and an OMC of 39.3%. For modified compaction of the natural soil-gravel (RGN) admixtures, the MDD ranged from 1325-2016 kg/m^3 whereas the OMC was in the range of 35.9-14.6%. The corresponding results for the soil-gravel-lime (RGT) admixtures, at design lime content, were a MDD of 1272-2006 kg/m^3 and OMC of 37.4-15.1%. Figure 4.9 illustrates the variation in the MDD and OMC for the RGN and RGT admixtures. The same results are also tabulated in Table A7.8 of the Appendix IV.

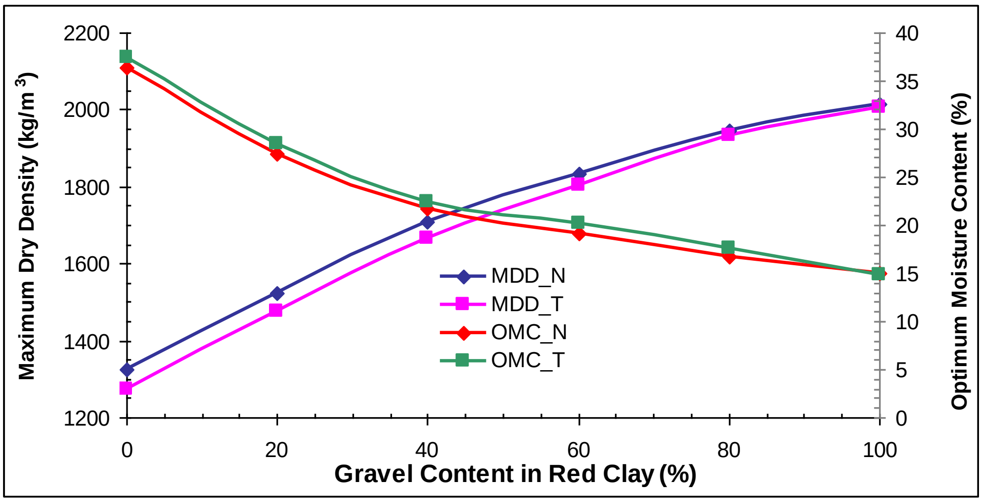

Figure 4.9: Variation in MDD and OMC of soil admixtures with gravel and lime

Key: MDD_N, OMC_N = MDD and OMC for Natural red soil and gravel admixtures

MDD_T, OMC_T = MDD and OMC for lime-Treated red soil and gravel admixtures

Coleman et al (1964) attributed the low compactibility of red soil to its nature represented by high clay content, a large specific surface area, high moisture holding capacity, and a porous surface of hematite. He added that loss of water by drying of soil also gives rise to abnormally high moisture content and low dry density of the compacted red soil. The MDD and OMC results obtained for natural red soil, upon its standard compaction, lay within the respective ranges of 1153-1363 kg/m^3 and 50-32% reported on red clay soils by Gichaga et al. (1987), and Dixon and Robertson (1970). The OMC was also less than its plastic limit of 44.2%, as observed by Terzaghi (1958). Upon modified compaction, the MDD for red soil at standard compaction actually increased by about 6% from 1255 to 1325 kg/m^3 while the corresponding OMC dropped by about 9% from 39.3 to 35.9%. This agreed with Rijn (2005) findings that MDD of a soil generally increases by 5-10% when 4.5 times the standard effort becomes the applied compactive energy. According to Budhu (2011) and Craig (2004), this is the general effect whenever the compactive effort is raised, and vice versa.

Figure 4.9 further demonstrated that the modified MDD increased, and OMC decreased, as the RGN admixtures became more granular and less plastic. These results were generally within the values for clayey and silty soils, and also granular soils. The MDD was in the range of 1550-2060 kg/m^3 and the OMC in the range of 35-15% as reported by Johannessen (2008) and Wignall et al. (1999). It was also evident that the RGT admixtures attained a slightly lower MDD, and a slightly higher OMC, than the RGN admixtures for the same compactive effort. Jawad et al. (2014), Rijn (2005) and O'Flaherty (2002) attributed this phenomenon to an increase in void ratio due to the resultant flocculation and textural change of soil admixture when treated with lime or cement. According to Powrie (2004), Craig (2004) and Cook et al. (2001), the addition of lime also makes the soil more stiff and difficult to compact and this raises the water content and lowers the dry density.

Soil compaction is extensively employed in construction of embankments and road pavements. The laboratory compaction results are relevant in control and evaluation of such field compaction of earthworks where the MDD and OMC represent 100% of field compaction and suitable moisture content for a given soil, respectively (Budhu, 2011; O’Flaherty, 2007). Many road agencies specify relative compactions of at least 95% the laboratory MDD for subbases and 98% for bases while the compacting water content should be within 80-105% of the OMC (Wambura et al., 1991; MoTC, 1987). According to Craig (2004) and Little (1995), higher relative compactions are advantageous to obtain and hence other laboratory test specimens were prepared at 100% MDD. Such test specimens were used to determine optimal or design lime content and bearing strength of different soil admixtures.

4.2.2 Lime Demand

The initial consumption of lime, based on the PI and percent passing 0.425 mm of RGN admixtures, ranged between 4.6% and 2.4% of hydrated lime as it was shown in Table 4.1. This amount included a small construction tolerance of 0.5% as proposed by O’Flaherty (2002) and AustStab (2002). The ICL so obtained was then adjusted for the maximum possible MDD by means of compaction at three points about this lime content for the RGT admixtures. As a result, the design lime content for the admixtures ranged from 4.7-2.2% of hydrated lime. Nevertheless, the ICL for 100% natural gravel was approximated from the chart also shown in Figure 2.14 and whose application is limited to at least 3% PI and 10% fines in an admixture.

According to, addition of 2-5% of hydrated lime by weight, and. The variation in ICL and design lime content with gravel content in the red soil is shown in Figure 4.10. This demonstrates that both the ICL and design lime content reduced with increasing gravel content in an admixture. The design lime content obtained was within 2-5% hydrated lime by weight reported by Matalucci (1962) to have, almost without exception, an increase in soil strength and a marked decrease in PI. Further, design lime content was generally adequate for stabilization of a kaolinitic clay that require 1-3% by mass of lime to achieve full flocculation potential of the clay as reported by O’Flaherty (2002), AustStab (2002) and Hudson (1997). The results for hydrated lime content in natural materials were roughly within the ranges recommended by MoTC (1987) and MoRPW (1986) of 2-4% for subbase and 4-6% for roadbase. It was apparent that the design lime contents were generally applicable in subbase but could only be determined in collaboration with CBR strength test, as summarized in Table 4.6 under section 4.4.2.2.

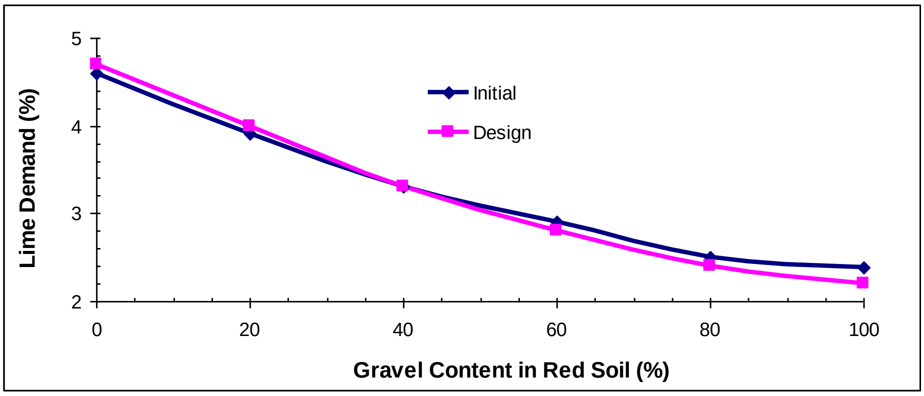

Figure 4.10: Variation in demand for hydrated lime with gravel content

4.3 Strength of Red Soil-Gravel-Lime Admixtures

4.3.1 Aggregate Crushing and Impact Values

The determination of the mechanical strength of natural gravel was outlined in section 3.5.2. Both the ACV and AIV provide a relative measure of aggregate toughness. The difference between individual test results were established to be within the respective allowable tolerances of 0.07 and 0.15 times the mean value for ACV and AIV specified by BS 812 (1990). The average results obtained for the two aggregate properties are presented in Table 4.5 and Table A7.9 of Appendix IV.

Table 4.5: Mechanical properties of natural gravel

Property	Mean Value (%)	Limiting Value* (%)	Remark*
ACV	24	30	For pavement wearing surface
		45	For other use
AIV	20	30	For pavement wearing surface
		45	For other use

(Source*: BS 882, 1992)

Cocks et al. (2015) posited that the performance of a material as a subbase or base course is largely dependent upon its strength and stiffness which may be reasonably inferred from the simple index tests for particle size distribution and PI. These come mainly from mechanical interlock and toughness of the constituent particles. Both the ACV and AIV of the natural gravel fell below the limiting value of 30% for pavement wearing course and the gravel satisified the requirements of stone Class A

(MoTC, 1987; MoRPW, 1986). This meant that the natural gravel was generally safe against fragmentation and its quality superseded the one required for either subbase or base materials. The high quality of gravel also removed the general caution placed by MoTC (1987) over use of natural gravels derived from weathered igneous rocks and perceived to be of poor quality, particularly basalt and phonolite. This also confirmed the finding by Cocks et al. (2015) and O'Flaherty (2002) that conventional criteria based on classification tests often have the disadvantage of excluding some materials capable of giving satisfactory performance.

According to BS 812-111 (1990), the AIV can be used to estimate more conveniently the alternative and tedious Ten Percent Fines (TPF) value of an aggregate. The TPF, in turn, is also related to the Los Angeles Abrasion (LAA) test which is often the reference test for the determination of resistance to fragmentation of pavement aggregates but it was not performed for lack of the equipment.

4.3.2 California Bearing Ratio

O'Flaherty (2002) indicated that the CBR of a soil is an empirical or arbitrary index of its shear strength. The CBR test was based on either standard or modified method of compaction with its applicable MDD and OMC as outlined under section 3.5.2. As further discussed in section 2.5.3.2, the CBR so obtained is used universally in classification of subgrade soil or in thickness (that is, structural) design of road pavements, respectively.

4.3.2.1 Subgrade Classification

According to Ministry of Transport and Infrastructure [MoT] (2013), subgrade strength in wet areas like Nyeri is determined based on CBR at 100% MDD of standard compaction and 4 days soak. The average soaked CBR for the red soil at standard compaction was 10.6% with a swell factor of 0.89%. These values were within the values specified by MoTC (1987) of 5-30% CBR and a swell factor less than 2% for any soil to qualify as a direct subgrade material. The CBR of red soil was also within the range of 2-15% reported by Wambura et al. (2003) for most Kenyan soils. The red soil therefore classified as subgrade Class S3 which did not require removal or overlying with a a higher quality material since it was adequately strong for direct support of a road pavement. Nevertheless, Holt (2010) and MoTC

(1987) suggested that subgrade soils Class S2 and S3 may receive an improved subgrade (or capping layer) of subgrade material classes S2 to S5, relative to cost.

Contrary to the rating of the red clay soil by the AASHTO classification system as a poor subgrade material, its subgrade class proved otherwise. Its qualification as a good natural subgrade also agreed with the findings by Elsharief et al. (2013), Wesley (2010), Bruggemann and Gosden (2004), Rolt (1979) and Smart (1973) on similar soils. They found that the red soil is remarkably stronger and surprisingly favorable as a subgrade material than it is normally considered, which is beneficial in terms of increased bearing strength, reduced overlying layer thickness, and minimized surface levels at certain critical locations (AustStab, 2002). This further proved many authors right in their common view that uniform criteria for classification of soils do not exist, and that this also fails to reconcile with properties of tropical soils because of local variations and characteristics (Cocks et al., 2015; Venkatramaiah, 2006; O'Flaherty, 2002; Verrujit, 2001; Northmore et al., 1992a).

4.3.2.2 Design Bearing Strength

Test specimens were prepared and tested as outlined in section 3.5.2.3. The soaking period was either 4 days or 7 days after specifications for natural or treated test specimens, respectively (MoTC, 1987; MoRPW, 1986). Soaked CBR values ranged from 16.8-112.6% for RGN admixtures and from 31.5-138.5% for RGT admixtures. These test results are presented in Figure 4.11, and in Table A7.8 of Appendix IV.

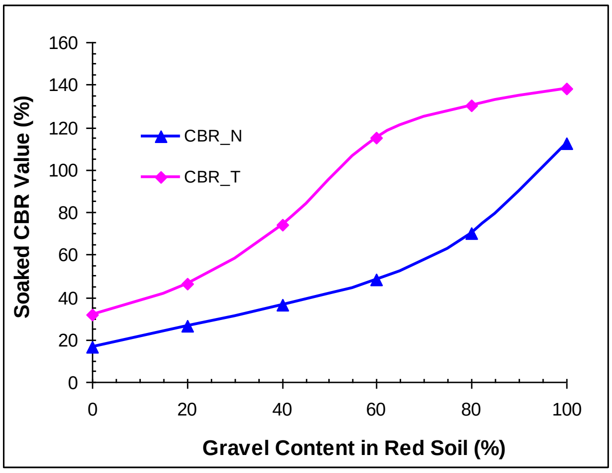

Figure 4.11: CBR values for neat and lime-treated soil admixtures

Key: CBR_N = CBR for natural (N) red soil and gravel admixtures

CBR_T = CBR for lime treated (T) red soil and gravel admixtures

The corresponding swell factors, as at the end of soaking period, ranged from 0.84-0.31% for RGN admixtures and from 0.79-0.29% for RGT admixtures. Figure 4.12 is a representation of these swelling results.

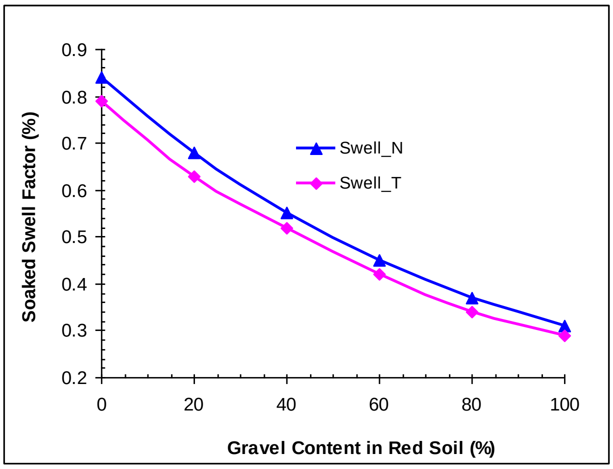

Figure 4.12: Swelling factor for different soil admixtures

It was evident from Figures 4.11 and 4.12 that the soaked CBR and swell factor values differed in variation with increasing gravel content in red soil. Thus, the CBR increased progressively, while the swell factor decreased, with increasing gravel content in the soil. Addition of hydrated lime to the soil-gravel admixtures had a similar effect with increasing gravel content but was accompanied by a further increase in CBR but a reduced swell factor. Additionally, there was a significant change in CBR at about 60% gravel content for both RGN and RGT admixtures. After this point, and as the admixtures became more gravelly in composition, the CBR for RGN admixtures increased steadily while that of RGT admixtures increased marginally. This point probably marked the limit of influence of both the fine red soil and hydrated lime in the admixtures. According to Cook et al. (2001) and Hudson (1997), fine soils may contribute towards improved internal friction of an admixture while some natural gravel may exhibit considerable strength gain with lime treatment. They explained that the latter case is due to formation of a natural cement in a pozzolanic reaction that occurs between lime and the available silica and probably some alumina in the clay. The trend of both CBR curves appears to corroborate the two different effects of clay and lime in gravel.

The soaked CBR and swell factors presented here are applicable in wet areas like Nyeri County with a mean annual rainfall over 500 mm, after MoTC (1987) and MoRPW (1986) specifications. These two values are highly influenced by the type of material, method of compaction and the compacted dry density obtained (Jawad et al., 2014; MoTC, 1987; Morin et al., 1971). For instance, the CBR of the natural red soil rose from 11.6% to 16.8% with a mere change in method of compaction from standard to modified type. This was also prove that compaction translates into instantaneous strength gains and reduced permeability for the natural subgrade soil as held by Jawad et al. (2014). This outcome suggested that red soil could be improved to subgrade Class 4 by simply raising the compactive effort to that of modified method.

Both MoTC (1987) and MoRPW (1986) further specified that all natural materials with at least 30 and 80% soaked CBR at 95% MDD qualify for use as subbase and base course materials, respectively, of a standard road pavement. Nevertheless, the subbase layer may not be used in rare cases when the subgrade strength is 30% CBR or class S6. Cement or lime treated natural materials with at least 60 and 160% soaked CBR at 95% MDD also qualify for similar application, respectively. It was observed from the soaking phase that the soil-gravel admixtures compacted into stable and dense mass with minimum swell potential and water ingress. The MoTC (1987) and MoRPW (1986) design standards are silent on limits of the swell factors but this was expected to be less than 2% specified for a subgrade, and below which all the soaked swell factors fell.

4.3.2.3 Application in Road Pavement Design

Roadway design standards usually rely on strength test results to approve construction materials. Performance of pavement layers depend on subgrade strength and an improved subgrade is often beneficial in increasing bearing strength at reduced cost and thinner pavement (Holt, 2010; Vorobieff & Murphy, 2003; O'Flaherty, 2002; MoTC, 1987). All admixtures with a CBR of 15-30% qualified for use as improved subgrade classes S5 and S6. Suitable mix proportions for for use as subbase and base course materials were derived from Figures 4.10 and 4.11. This was based on MoTC (1987) and MoRPW (1986) criteria that qualify both natural and treated materials for such application. Accordingly, Table 4.6 summarizes the diferent mix proportions that qualified for use as subgrade, improved subgrade, subbase and even base material. This demonstrated that some of the soil-gravel admixtures were not only useful in LVRs but also in high volume roads.

Table 4.6: Material mix proportions for different road pavement layers

Pavement Layer	CBR Range (%)	Gravel Content (%)	Red Soil Content (%)	Lime Content (%)
Natural Materials (RGN)				
Normal Subgrade	5-15	0	100	-
Improved Subgrade	15-30	0-31	100-69	-
Subbase	30-80	31-87.4	69-12.6	-
Base	>80	87.4-100	12.6-0	-
Lime-Treated Materials (RGT)				
Normal Subgrade	10-30		Not Applicable	
Improved Subgrade	30-60	0-32.2	95.3-64.2	4.7-3.58
Subbase	60-160	32.2-97.8	64.2-0	3.58-2.2
Base	>160		Not Achieved	

Results in Table 4.6 shows that none of the soil admixtures had a soaked CBR of less than 15%. Therefore, all the admixtures were of high quality and applicable at least as improved subgrade material. Further, the RGN admixtures performed so well that the highest soaked CBR in the range was beyond 80% and qualified the material as basecourse, which was outside the scope of this study. Incidentally, this was a feat that the RGT admixtures could not realize since its highest CBR was below the minium 160% required. Assuming the average values to represent a range of applicable mixes, then the improved subgrade would be constituted of either 84.5% red soil and 15.5% natural gravel or consist of 79.8% red soil, 16.1% natural gravel and 4.1% hydrated lime. Similarly, the subbase would be constituted of either 40.8% red soil and 59.2% natural gravel or consist of 32.1% red soil, 65.0% natural gravel and 2.9% hydrated lime. Finally, the base would be constituted of only 6.3% red soil and 93.7% natural gravel since the lime-treated natural admixtures failed to meet the strength requirements of the base. Thus, incorporation of the red soil in various soil admixtures reduced progressively, as expected, as the strength requirements for a pavement layer increased.

Even though most of the RGN admixtures qualified in terms of strength for use as subbase and base materials, they were eliminated on the basis of plasticity requirements. The plasticity of these admixtures generally fell beyond the maximum plasticity index of 15% allowed for wet areas by the MoTC (1987) and MoRPW (1986). This outcome justified the use of hydrated lime in lowering the high plasticity of neat soil-gravel admixtures. The optimal mixes therefore consisted of 79.8% red soil, 16.1% natural gravel and 4.1% hydrated lime for an improved subgrade, and 32.1% red soil, 65.0% natural gravel and 2.9% hydrated lime for the subbase.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

In view of the laboratory results and their discussion, the following conclusions were made.

1) Material Properties

The red soil was identified as highly-weathered 'laterite' with kaolinite as predominant clay mineral. It was described as 'clayey silty SAND' which classified as MH and as poor subgrade material A-7-5, after USCS AASHTO soil classification systems. Similarly, the grayish natural gravel was identified as partially-weathered 'lateritic gravel'. It was described as 'sandy GRAVEL' which classified as GW and good subgrade material A-2-7 according to the two respective classification systems. The hydrated lime in a fine white powder form consisted mainly of 72.5% calcium oxide by weight, equivalent to 95.4% calcium hydroxide, and other minor constituents.

2) Compaction Characteristics

The red soil compacted easily into a dense mass with a MDD of 1255 kg/m^3 and an OMC of 39.3% after standard compaction. The MDD for soil-gravel admixtures increased from 1325-2016 kg/m^3 while OMC decreased from 35.9-14.6% with increasing gravel content. The treated soil-gravel-lime admixtures had a slight reduction in MDD which ranged from 1272-2006 kg/m^3, and an increase in OMC that ranged from 37.4-15.1%. The amount of hydrated lime required was based on soil plasticity and decreased with increasing gravel content from 4.7-2.2% by weight. Addition of gravel and hydrated lime to red clay soil also had an effect of greatly increasing the MDD and decreasing OMC of admixtures. Moreover, most admixtures compacted with relative ease and to higher densities and reduced moisture though it became more difficult as gravel content increased.

3) Material Strength

Based on standard compaction, the red soil was strong enough to support a road pavement after classifying as subgrade soil class S3 with a soaked CBR of 11%. The soil also had a soaked CBR of 17% upon modified compaction and this was equivalent to subgrade soil class S4 that simply eliminated the need of any improved subgrade. The bearing strength of the all admixtures generally increased with increasing gravel content and with the addition of lime. The corresponding soaked CBR ranged from 16.8-112.6% and from 31.5-138.5%, respectively. Similarly, all swell factors generally reduced and ranged from 0.84-0.31% and from 0.79-0.29%, respectively. These values were well below the maximum 2% allowed for in design.

4) General Conclusion

The lateritic gravel is effective in stabilizing red clay soil by lowering its plasticity and increasing its strength. Moreover, these parameters are significantly improved by the addition of hydrated lime to meet requirements for use as subgrade and subbase of LVRs. According to MoTC (1987) specifications, the natural gravel qualified as Stone Class A, with an ACV value of 24%. The natural and treated admixtures with 30-80% and 60-160% CBR, respectively, qualified as both subbase and base materials. However, all the natural admixtures were eliminated from this application based on the plasticity index criteria. Thus, the optimal mixes for soil-gravel-lime admixtures consisted of 79.8% red soil, 16.1% natural gravel and 4.1% hydrated lime for an improved subgrade, and 32.1% red soil, 65.0% natural gravel and 2.9% hydrated lime for the subbase. This demonstrated the success of double stabilization of red clay soil using natural gravel and hydrated lime to produce a fairly wide range of satisfactory materials for improved subgrade and subbase of LVSRs.

5.2 RECOMMENDATIONS

The following recommendations were arrived at from the findings of this study:

a) For Application

1) Natural gravel and hydrated lime have different capacities – lime is more effective – in lowering the high plasticity of red soil and result in non-rigid or plastic soil not susceptible to cracking.

2) Higher compacted densities can be realized by adding natural gravel and hydrated lime to red soil but lime could be beneficial in reduction of self weight though at a higher water demand.

3) The stability and bearing strength of red soil are substantially increased by adding natural gravel and hydrated lime with the latter giving even better results regardless of reduced density.

b) For Further Research

1) The products of this study may be tried as surfaces to footpaths, tracks and roads in remote and steep rural areas prone to soil erosion.

2) Additional research may be conducted on the soil-gravel admixtures using a higher amount of lime than determined here, or Portland cement to realize much stronger stabilized admixtures that qualify as road base material.

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APPENDICES

Appendix I: Soil Description and Classification

Table A7.1: Unified Soil Classification System (USCS)

Unified Soil Classification System (USCS) - from ASTM D 2487				
Major Divisions			**Group Symbol**	**Typical Names**
Course-Grained Soils More than 50% retained on the 0.075 mm (No. 200) sieve	**Gravels** 50% or more of course fraction retained on the 4.75 mm (No. 4) sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravels with Fines	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands 50% or more of course fraction passes the 4.75 (No. 4) sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
Fine-Grained Soils More than 50% passes the 0.075 mm (No. 200) sieve	**Silts and Clays** Liquid Limit 50% or less		ML	Inorganic silts, very fine sands, rock four, silty or clayey fine sands
			CL	Inorganic clays of low to medium plasticity, gravelly/sandy/silty/lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays Liquid Limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
			CH	Inorganic clays or high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, muck, and other highly organic soils

Prefix: G = Gravel, S = Sand, M = Silt, C = Clay, O = Organic
Suffix: W = Well Graded, P = Poorly Graded, M = Silty, L = Clay, LL < 50%, H = Clay, LL > 50%

Table A7.2: AASHTO Soil Classification System

AASHTO Soil Classification System - from AASHTO M 145												
General Classification	**Granular Materials** 35% or less passing the 0.075 mm sieve								**Silt-Clay Materials** >35% passing the 0.075 mm sieve			
Group Classification	**A-1**		**A-3**	**A-2**					**A-4**	**A-5**	**A-6**	**A-7**
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7					A-7-5 A-7-6
Sieve Analysis, % passing												
2.00 mm (No. 10)	50 max	---	---	---	---	---	---		---	---	---	---
0.425 (No. 40)	30 max	50 max	51 max	---	---	---	---		---	---	---	---
0.075 (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max		36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No. 40)												
Liquid limit	---		---	40 max	41 min	40 max	41 min		40 max	41 min	40 max	41 min
Plasticity index	6 max		N.P.	10 max	10 max	11 min	11 min		10 max	10 max	11 min	11 min[a]
Usual types of significant constituent materials	stone fragments, gravel and sand		fine sand	silty or clayey gravel and sand					silty soils		clayey soils	
General rating as a subgrade	excellent to good								fair to poor			

[a]Plasticity index of A-7-5 subgroup is equal to or less than the LL - 30. Plasticity index of A-7-6 subgroup is greater than LL - 30

Basic Characteristics of Soils

The description and classification of red soil and natural gravel, based on particle size distribution results, are summarized in Table A7.3.

Table A7.3: Basic characteristics of red soil and natural gravel

Property/Material	Red Soil	Natural Gravel
Color	Reddish Brown	Grey
Texture	Firm, Homogeneous	Hard, Jointed
Weathering	Full	Partial
Parent Rock	Volcanic, Basalt	Volcanic, Basalt
Particle Size – a) Clay (%)	34	0
- b) Silt (%)	20	1
- c) Sand (%)	44	10
- d) Gravel (%)	2	89
Soil Description	Clayey silty SAND	Sandy GRAVEL
Classification - USCS	MH	GW
- AASHTO	A-7-5	A-2-7
AASHTO Rating as Subgrade	Poor	Good
Kenyan Subgrade Strength Class	S3	S6

Figure A7.1, however, represents the evaluation of natural gravel used in subbase and base for possible fragmentation after its placement and compaction, as stipulated by GoK (1986) and presented in section 2.3.2. The natural gravel, in its uncompacted condition, is denoted as RGN 100. Resistance of the gravel to fragmentation may also be deduced from the low ACV and AIV values presented in Table A7.9.

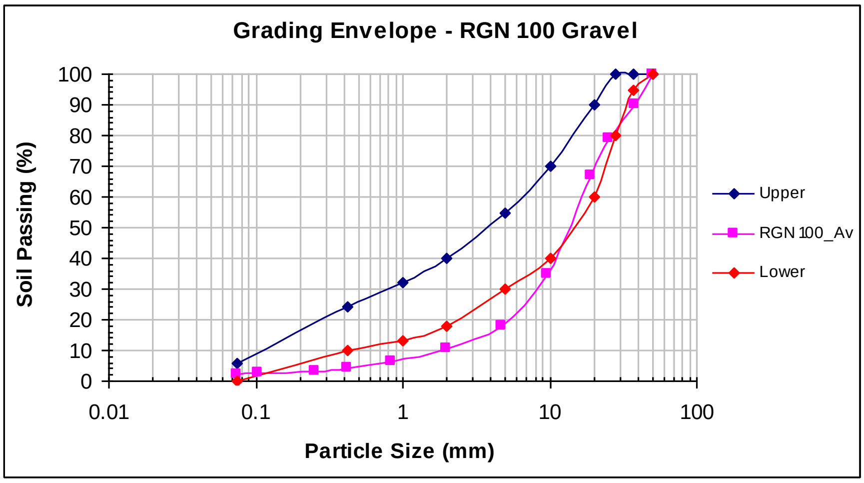

Figure A7.1: Plotting of natural gravel in grading envelopes for aggregates

Appendix II: Chemical and Mineralogical Composition

Table A7.4: Chemical composition of research materials

Department of Mines and Geology, Nairobi			
Material Type	**Red Soil**	**Natural Gravel**	**Lime**
Element Nature	**%**	**%**	**%**
Silica, SiO_2	40.672	50.748	-
Iron, Fe	25.396	13.727	0.492
Aluminium, Al_2O_3	26.649	13.413	1.858
Titanium, Ti	4.018	3.526	0.058
Sulphur, S	0.713	0.379	0.654
Phosphorous, P_2O_2	0.693	1.382	1.180
Manganese, Mn	0.451	0.216	0.023
Calcium, CaO, $*(OH)_2$	0.429	13.622	*95.371
Potassium, K_2O	0.226	2.494	-
Zirconium, Zr	0.200	0.070	0.028
Cerium, Ce	0.146	-	-
Chloride, Cl	0.078	0.001	0.110
Niobium, Nb	0.062	0.015	-
Stronium, Sr	0.058	0.207	0.268
Barium, Ba	0.058	0.129	-
Zinc, Zn	0.021	0.014	0.001
Copper, Cu	0.012	0.020	0.003
Nickel, Ni	0.007	0.008	-
Yttrium, Y	0.006	0.008	-
SUM	99.895	99.989	100.046

(Courtesy: Department of Mines and Geology, Nairobi)

Table A7.5: Complete mineralogical analysis data on red soil

ICRAF, Nairobi			
SSN	icr177385	**Site**	Othaya
Element			**%**
Kaolinite			47.9
Microcline			18.4
Albite			12.6
Quartz			11.6
Hematite			9.5
Total Carbon			0.347
Acidified Carbon			0.327
Total Nitrogen			0.036
Acidified Nitrogen			0.035

(Courtesy: ICRAF, Nairobi)

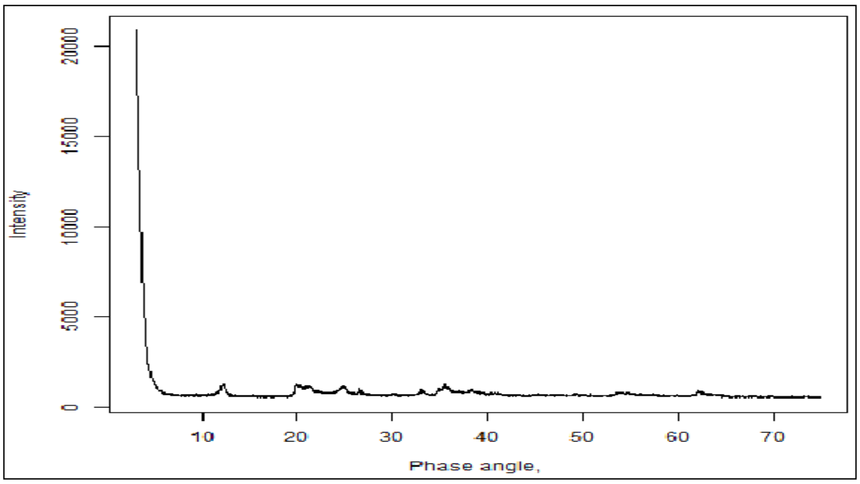

Figure A7.2: The XRD diffractogram for the red soil

(Courtesy: ICRAF, Nairobi)

Schematic illustration of different types of feldspars

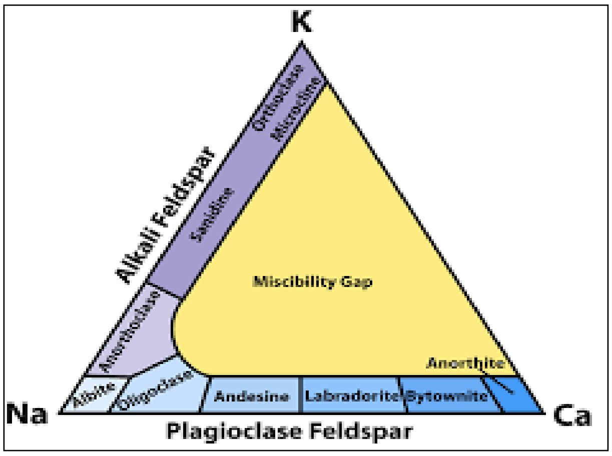

Figure A7.3: Feldspar composition diagram

There are no feldspars within the miscibility gap shown. The type of feldspar is based on its constituent molecular percentage.

Appendix III: Established Material Properties

Table A7.6: Potential volume change of soils by plasticity and climate

Potential Volume Change	Humid Climate		Arid & Semi-Arid Climate	
	PI %	LS %	PI %	LS %
Low	0-30	0-12	0-15	0-5
Medium	30-50	12-18	15-30	5-12
High	>50	>18	>30	>12

(Source: Hazelton & Murphy, 2007)

Table A7.7: Subgrade Beaing Strength Classes and Elastic modulus

Soil Class	CBR		Elastic Modulus	
	Range (%)	Median (%)	Kg/cm^2	kN/m^2 ($\times 10^3$)
S1	2-5	3.5	150	15
S2	5-10	7.5	500	50
S3	7-13	10	650	65
S4	10-18	14	900	90
S5	15-30	22.5	1250	125
S6	≥30	-	2500	250

(Source: MoTC, 1987)

Appendix IV: Summarized Laboratory Results and Quality of Materials

Table A7.8: Summary of laboratory results for soil admixtures

Parameter		**Soil-Gravel Admixture (RGN)**					
Gravel Content	%	**0**	**20**	**40**	**60**	**80**	**100**
Specific Gravity		2.79	2.79	2.78	2.77	2.75	2.73
Estimated Fines (<425 μm)	%	82.1	66.5	51.0	35.4	19.9	4.30
Colloidal Activity	%	0.37	0.38	0.45	0.60	1.10	2.10
Free Swell Index	%	0	0.36	0.71	1.24	1.94	2.63
Liquid Limit (Cone)	%	74.3	67.3	60.2	53.5	46.8	41.8
Plastic Limit	%	44.2	42.2	38.2	33.9	28.8	24.2
Linear Shrinkage	%	15.7	12.4	11.3	10.2	9.5	8.6
Plasticity Index	%	30.1	25.1	22/0	19.7	18.0	17.6
Initial Lime	%	4.60	3.91	3.31	2.90	2.50	2.38
Maximum Dry Density	kg/m^3	1325	1523	1707	1835	1945	2016
Optimum Moisture Content	%	35.9	27.1	21.5	19.3	17.2	14.6
California Bearing Ratio CBR	%	16.8	26.3	36.2	48.1	70.2	112.6
CBR Swell Factor	%	0.84	0.68	0.55	0.45	0.37	0.31
Parameter		**Soil-Gravel-Lime Admixture (RGT)**					
Gravel Content	%	**0**	**20**	**40**	**60**	**80**	**100**
Specific Gravity		2.77	2.76	2.76	2.75	2.73	2.71
Estimated Fines (<425 μm)	%	-	-	-	-	-	-
Colloidal Activity	%	-	-	-	-	-	-
Free Swell Index	%	4.00	6.69	10.27	12.73	12.45	11.76
Liquid Limit (Cone)	%	59.1	54.6	50.4	46.4	41.5	37.9
Plastic Limit	%	47.0	44.0	41.2	38.3	34.3	31.3
Linear Shrinkage	%	8.2	7.1	6.2	5.1	4.1	3.3
Plasticity Index	%	12.0	10.6	9.2	8.1	7.2	6.6
Design Lime	%	4.7	4.0	3.3	2.8	2.4	2.2
Maximum Dry Density	kg/m^3	1272	1476	1667	1805	1931	2006
Optimum Moisture Content	%	37.4	28.5	22.5	20.2	17.9	15.1
California Bearing Ratio CBR	%	31.5	46.5	73.8	115.0	130.2	138.5
CBR Swell Factor	%	0.79	0.66	0.52	0.42	0.34	0.29

Table A7.9: Density and bearing strength of natural materials

Parameter		**Limit ***	**Red Soil**	**Natural Gravel**
Proctor Maximum Dry Density	(kg/m^3)	-	1255	-
Optimum Moisture Content	(%)	-	39.3	-
Proctor California Bearing Ratio	(%)	≥ 2	10.6	-
Proctor Swelling Factor	(%)	< 2	0.89	-
Aggregate Crushing Value	(%)	< 45	-	23.97
Aggregate Impact Value	(%)	< 45	-	20.09

(*Source: MoTC, 1987; BS 882, 1992)

Table A7.10: Summarized Rating for Properties of Research Materials

Property		Natural Materials			Requirements		Suitability
		Red Soil	Gravel	Lime	Limit	Reference	
Color		Brown	Gray	White	None	Budhu (2011)	Suitable
Moisture Content	(%)	23.5	11.7	-	0-1200	Day (2010)	Suitable
Specific Gravity		2.79	2.73	2.35	2.55-2.75	Smith (2014); Day (2010)	Suitable
Gravel Fraction	(%)	2	89	-	None		Suitable
Sand Fraction	(%)	44	10	-	None		Suitable
Silt Fraction	(%)	20	1	-	None		Suitable
Clay Fraction	(%)	34	-	-	50-80	Newill (1961); Terzaghi (1958)	Unsuitable
Free Swell Index	(%)	0	2.4	-	50	Holtz & Gibbs (1956)	Suitable
Clay Activity		0.37	2	-	Inactive/Active	Skempton (1953)	Suitable
Organic Matter	(%)	0.35	-	-	3-1	MoTC (1987); MoRPW (1986)	Suitable
Clay Mineral, main		Kaolinite	-	-	None	ICRAF	Suitable
Chemical Compound, main		Quartz, SiO_2	Quartz, SiO_2	$Ca(OH)_2$	None	State Dept of Mines	Suitable
Free Lime as CaO	(%)	-	-	72.5	50_{min}	MoRPW (1986)	Suitable
Residue on 0.2 mm	(%)	-	-	0.83	1_{max}	MoRPW (1986)	Suitable
Residue on 0.075 mm	(%)	-	-	1.8	10_{max}	MoRPW (1986)	Suitable
Geological Description		Recent Deposit	Olivine Basalt	-	None	Fairburn (1966); Shackleton (1945)	Suitable
Textural Description		Silty SAND	Sandy GRAVEL	Powder	None	Craig (2004)	Suitable
USCS Classification		Silt (MH)	Gravel (GW)	-	None	ASTM D 2487	Suitable
AASHTO Classification		Clayey (A-7-5)	Gravel (A-2-7)	-	None	AASHTO M145	Suitable
AASHTO Rating as Subgrade		Poor*	Good	-	None	AASHTO M145	Soil Unsuitable

Table A7.11: Summary of Material Properties for Subbase

Property		Red Soil		Gravel		Materials Requirements		Suitability
		RGN	RGT	RGN	RGT	Limit	Reference	
Uniformity Coefficient	(No)	-	-	9	-	5_{min}	MoTC (1987); MoRPW (1986)	Suitable
Maximum Particle Size	(mm)	5	-	50	-	63_{max}; 10-50	MoRPW (1986)	Suitable
Passing 2 mm	(%)	98	-	11	-	95_{max}	MoTC (1987); MoRPW (1986)	Suitable
Passing 0.425 mm	(%)	82	-	4*	-	15_{min}	MoTC (1987); MoRPW (1986)	Grv Unsuitable
Passing 0.075 mm	(%)	54	-	2*	-	10-30; 50_{max}	MoTC (1987); MoRPW (1986)	Grv Unsuitable
Plasticity Index, wet area	(%)	30.1*	17.6*	12.0	6.6	5-12; 15_{max}	MoTC (1987); MoRPW (1986)	Neat Unsuitable
Plasticity Modulus	(%)	2472	986	75	28	2,500; 250_{max}	MoTC (1987); MoRPW (1986)	Suitable
Linear Shrinkage	(%)	15.7*	8.2	8.6	3.3	12_{max}	Hazelton & Murphy (2007)	Soil Unsuitable
Hydrated Lime Required	(%)	-	4.7	-	2.2	2-4	MoTC (1987); MoRPW (1986)	Suitable
Organic Matter	(%)	0.35	-	-	-	2_{max}	MoTC (1987); MoRPW (1986)	Suitable
MDD (Standard)	(kg/m^3)	1255	-	-	-	None		Suitable
OMC (Standard)	(%)	39.3	-	-	-	None		Suitable
Subgrade Soaked CBR	(%)	10.6	-	-	-	5-30	MoTC (1987); MoRPW (1986)	Suitable
- CBR Swell factor	(%)	0.89	-	-	-	2_{max}	MoTC (1987); MoRPW (1986)	Suitable
Subgrade Strength Class		S3	-	-	-	S2-S5	MoTC (1987); MoRPW (1986)	Suitable
MDD (Modified)	(kg/m^3)	1325	1272	2016	2006	None		Suitable
OMC (Modified)	(%)	35.9	37.4	14.6	15.1	None		Suitable
Subbase Soaked CBR	(%)	16.8*	31.5	112.6	138.5	30-80	MoTC (1987); MoRPW (1986)	Suitable
- CBR Swell factor	(%)	0.84	0.79	0.31	0.29	2_{max*}	MoTC (1987); MoRPW (1986)	Suitable
Aggr Crushing Value	(%)	-	-	24	-	35_{max}	MoTC (1987)	Suitable
Aggregate Impact Value	(%)	-	-	20	-	45_{max}	BS 882 (1992)	Suitable

Appendix V: Stabilization Effect on Soil Plasticity and CBR

Reduction in plasticity index of any soil-gravel admixture was based on the value for the neat red soil. The effect of natural gravel on PI was proportional to the gravel content as shown in Figure A7.4. However, the net effect of hydrated lime dropped with increasing gravel content. The combined effect of double stabilization stood at about 60% of the neat red soil and no admixture became non-plastic.

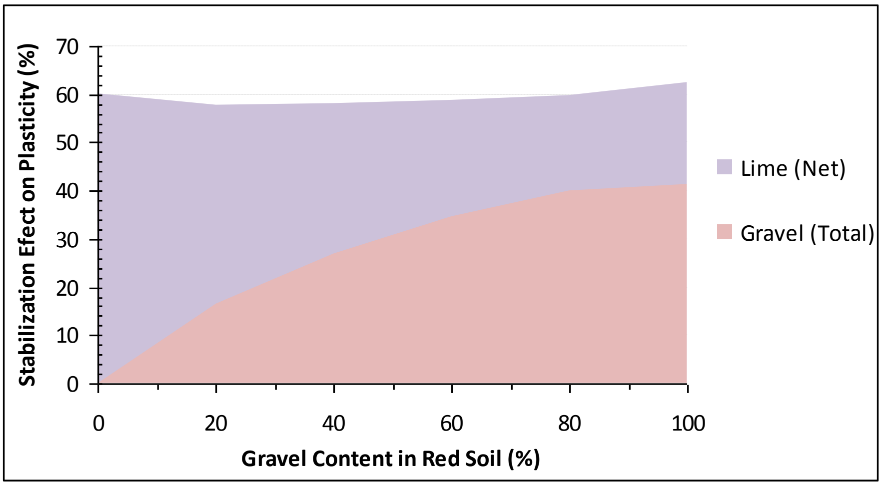

Figure A7.4: Improvement in soil plasticity with stabilization method

Based on the CBR for neat red soil, the bearing strength was highly dependent on the gravel content. Beyond about 25% gravel content, the net effect of hydrated lime was exceeded by that of gravel as presented in Figure A7.5.

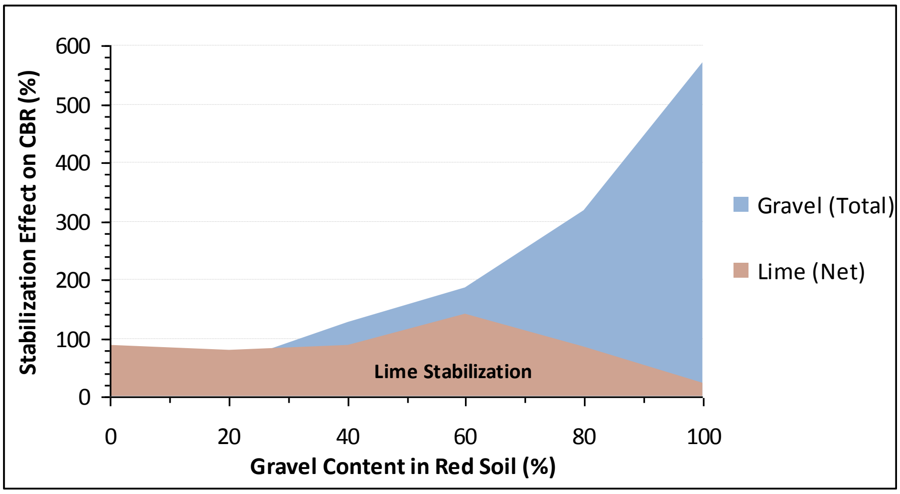

Figure A7.5: Gain in CBR with stabilization method