

**SUITABILITY OF USING MIXES OF HEATED BLACK
COTTON SOIL, MOLASSES AND LIME IN
STABILIZATION OF EXPANSIVE CLAY SOIL**

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**Suitability of using mixes of heated black cotton soil, molasses and lime
in stabilization of expansive clay soil**

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Construction Engineering and Management of Jomo Kenyatta
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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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DEDICATION

This work is dedicated to my daughter Mercy Muthoni Kabubo, My father, Joseph Kabubo (deceased), my mother Rose Mary Muthoni as well as my brothers and sisters for their prayers, support and encouragement

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My greatest gratitude goes to the Lord God Almighty, the Father of our Lord and Saviour Jesus Christ who has been my guide and inspiration through out the research, for apart from Him, I can do nothing. I also wish to express my sincere thanks to my supervisors, Prof SK Mwea, Dr SO Abuodha and Prof A Gariy because of their good guidance, advice, encouragement and constructive criticism which were of great help in carrying out the research. My thanks also go to the staff in JKUAT, ICRAF and the State Department of Infrastructure soils laboratories who were of great assistance. God bless you all.

ABSTRACT

Expansive clays, in moisture content fluctuation situations, cause deformations which result in heavy damage to pavements and structures built on them causing losses which run into billions of dollars worldwide. Most improvement methods use lime or lime-activated materials to treat black cotton soil, or cut to spoil followed by replacement with suitable materials. The former methods have challenges in regions of either high moisture and or in flood-prone areas. Though lime-activated Ground Granulated Blast Furnace Slag (GGBS) stabilized expansive clays have been found to withstand severe flooding conditions, GGBS is not locally available in Kenya. On the other hand, cut to spoil followed by replacement with suitable materials has high cost and environmental implications. This study undertook to stabilize black cotton soil in-place to withstand flooding conditions or high moisture situations using heated black soil in-place and mixes of lime, molasses and heated black cotton soil including evaluating their economic viability. Swelling characteristics and strength properties were conducted on the soil heated at various temperatures. Unconfined Compressive Strength (UCS) tests on various mixes of heated soil, lime and molasses established an optimal mix of 14% heated black cotton soil at 400°C, 6% lime and 0% molasses as the binders with 80% black cotton soil. Properties of this mix subjected to moist curing periods of 7, 14, 28, 56 and 90 days followed by soaking them in water for 4 and 10 days after each curing period to simulate flash and severe flooding respectively were investigated. The durability reduction indices for the each flooding scenario were computed. A model for a 2000x2000 mm building was constructed with a ground floor slab on a 1000 mm fill comprising the optimal mix, from which the heave was measured for moist curing periods of 7, 14, 28, 56 and 90 days and after 10 days side flooding at the end of 90 days. Costs were then determined for filling road embankments and under ground floor slabs with the heated soil and the optimal mix. The heated soil yielded a material suitable for fill at a temperature of 700°C when the soil became non-plastic with a soaked California Bearing Ratio (CBR) value of 16%. The optimal stabilized mix had a UCS value of 394 kN/m², and soaked CBR value of 39.2% with CBR swell of 0.22%; it was found durable and able to withstand flooding

conditions. The ground floor slab attained a maximum settlement of 5 mm, with no heave. The cost per m³ for filling with black cotton soil heated to 400°C activated with lime was Kshs 3900 and Kshs 3850 for roads and ground floor slabs respectively whereas the respective costs for the heated soil was Kshs 11,360 and Kshs 10,340. The heated soil and lime activated heated soil are therefore effective stabilizers for both roads and ground floors slabs. However, since the cost of using heated soil is very high it is recommended that lime-activated heated black cotton soil be used to stabilize black cotton soils in Kenya in areas subject to flooding.

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LIST OF ABBREVIATIONS

AASHTO T99-	American Association of State Highway and Transport Official Light Compaction Method.
ASTM -	American Society for Testing and Materials
BS 1377 –	British Standards Institution 1377
BSCS-	British Soil Classification System
CAH –	Calcium Aluminate Hydrates
CASH –	Calcium Aluminum Silicate Hydrates
CBR –	California Bearing Ratio
CEC –	Cation Exchange Capacity
CEC-	Cat Ion Exchange Capacity
CM –	Cubic Metere
CSH –	Calcium Silicate Hydrates
GGBS –	Ground Granulated Blast Furnace Slag
HBC –	Heated black cotton soil
ICCD -	International Centre for Diffraction Data
Ip –	Plasticity index
JKUAT –	Jomo Kenyatta University and Technology
Km –	Kilometer
MDD –	Maximum Dry Density
MOTC, 1986 -	Standard Specification for Road and Bridge Construction, Roads Department, Ministry of Transport and Communication, Kenya.
MOTC, 1987 -	Road Design Manual Part III, Roads Department, Ministry of Transport and Communication, Kenya.

MTIH&UD -	Ministry of Transport, Infrastructure, Housing and Urban Development
NBC –	Neat black cotton soil
NL –	Natural Lime
OMC –	Optimum Moisture Content
SL –	Shrinkage Limit
UCS -	Unconfined Compressive Strength
UK –	United Kingdom
W _L –	Liquid Limit
W _p –	Plastic Limit
XRD –	X-ray diffraction
XRF -	X-Ray fluorescence
USA-	United States of America
SRI -	Strength Reduction Index

CHAPTER ONE

INTRODUCTION

1.1 Background of the study

Infrastructural facilities such as structures and road pavements constructed in areas where the predominant soil is expansive is very expensive because these soils have to be improved or replaced. According to Nelson and Miller (1992), expansive soils are known to exhibit dual characteristics of excessive swelling and shrinkage under different moisture conditions, which cause deformations that are significantly greater than elastic deformations. The movements caused are usually of an uneven pattern and of such magnitude as to cause extensive damage to the structures and pavements resting on them. According to Alhassan and Olaniyi (2013) these characteristics cause considerable challenges in utilization of expansive clays for infrastructural development.

The presence of expansive soils greatly affects the construction activities in many parts of South-Western United States, South America, Canada, Africa, Australia, Europe, India, China and Middle East (Chen, 1975). Black cotton soil, an expansive clay occupies about 3% of the world area, about 340 million hectares (Bhavsar & Patel, 2014).

Various methods that have been employed to improve expansive clays include mechanical compaction, surcharge loading, pre-wetting, lime stabilization, cement stabilization, fly ash stabilization, chemical stabilization and organic compounds treatment (Nelson & Miller, 1992). Other methods include cut to spoil followed by importing good material for replacement (Mohktari & Deghani, 2012; Kalantari, 2012; Ministry of Transport and Communications (MOTC), 1987) and thermal treatment (Li, Wu, & Hou, 2014; Wang, Jao & Ghazal, 2008; Terzaghi, Peck & Mesri, 1996; Bell, 1993 O'Flaherty, 1974).

Cut to spoil followed by replacement with good materials and lime stabilization are among the more frequently used methods of dealing with black cotton soils (Mohktari & Deghani, 2012; Kalantari, 2012; Yang and Zheng, 2006; MOTC, 1987).

But excavating and importing good material for replacement has huge cost implications due to increase in carbon footage resulting from burning fossil fuel or cost due to damage to the environment resulting in increase in greenhouse gases, responsible for global warming (Prusinski & Bhattacharja, 1999; Yang & Zheng, 2006). Furthermore, good replacement materials put pressure on agricultural land since they are located in arable land which has led to legislation and imposition of levies in some countries for their excavation and transportation making them more and more expensive. On the other hand, lime is not durable in environments with high moisture and flood prone areas (Ministry of Transport and Communications, 1987; Obuzor, Kinuthia & Robinson, 2012).

A number of industrial based bi-products and agricultural waste materials have been recently developed as stabilizers, and the current emphasis is to amend these aluminosilicates with alkali activators such as lime in one-part geopolymer reactions to enhance their pozzolanic and cementitious properties (Luukkonen, Abdollahnejad, Yliniemi, Kinnunen & Illikainen, 2018; Sharma & Sivapullaiah, 2016). The precursors that have been used include ground granulated blast furnace slag (GGBS) fly ash, silica fume, bagasse ash and rice husk ash with lime as the alkali activator (Kavak & Bilgen, 2016; Obuzor et al., 2012; Obuzor et al., 2011; Dang, Hassan, Fatahi, Jones & Khabbaz, 2016; Wubshet & Tabese, 2014; Satyanarayayana, Vijaya, Mohan & Jaya, 2016; Karatai, Kaluli, Kabubo & Thiongo, 2016). Only lime-activated GGBS has been found to be effective in withstanding prolonged curing and harsh flooding conditions (Obuzor et al., 2012). GGBS, is not available in Kenya. Heated black cotton soil, a pozzolanic low calcium aluminosilicate can be activated by lime and be used as a locally available stabilizer

Molasses has been found to be effective as a stabilizer for expansive clays but it has limitations in wet conditions since the sugar obtained is water soluble (M’Ndegwa, 2011; O’Flaherty, 1974). Thermal stabilization by heating or freezing has been used, but use of heated black cotton soil as an alkali-activated pozzolanic low calcium aluminosilicate precursor for stabilizing black cotton soil in-place has not been done to date. (Li, Wu, & Hou , 2014; Wang et al., 2008; Bell, 1993; O’Flaherty, 1974). Stabilizing black cotton soil in-place and obtaining a material which can withstand prolonged curing and endure harsh environmental challenges such as prolonged

flooding conditions by using lime-activated heated black cotton soil mixed with cementitious molasses can be beneficial. In-place improvement methods can result in natural resource conservation, reduction in energy usage, reduction in carbon dioxide emission and increased cost efficiency (Lee & Karunaratne, 2007; Phanikumar et al., 2008).

In Kenya, black cotton soil covers large areas, and the solution has been to remove and replace them with suitable material. Structures and roads constructed in areas where black cotton soil is not removed crack extensively as the moisture levels change, with failures experienced depending on how extensive black cotton soil is. Thus, obtaining an effective local method of improving black cotton soil in-place will reduce the negative effects of black cotton soil and optimize on cost of construction. According to Provis (2017), effectiveness of use of geopolymers is dependent on local availability of binders.

1.2 Problem Statement

Expansive soils exhibit dual characteristics of excessive swelling and shrinkage under different moisture conditions which cause deformations of an uneven pattern and of such magnitude as to cause extensive damage to the structures and pavements resting on them (Nelson & Miller, 1992). The characteristics of these soils cause great challenges in their utilization for infrastructural development. The cost of the damages is \$15 billion and £400 million in the USA and UK respectively and many billions of dollars worldwide (Bhavsar & Patel, 2014; Dang et al., 2016). Structures and pavements built on these soils are characterized by extensive cracks due to the alternate swelling and shrinkage as the moisture level changes, which leads to high rehabilitation costs or failure and reduction of their useful life with a consequence of high life cycle costs. To avoid such destruction, methods of dealing with expansive soils have been devised which include stabilizing the soils with lime, molasses, industrial and agricultural wastes, alkali-activated alluminosilicates, thermal stabilization, cutting and replacing with suitable material, and be re-alignment to avoid the expansive soils. These methods have limitations. Lime stabilized black cotton soil is not durable in flood prone areas (Obuzor et al., 2012; MOTC, 1987) while molasses stabilized soils are unstable in levels of high moisture because the

resulting product is water soluble (M’Ndegwa, 2011; O’Flaherty, 1974). Cutting and replacement has high cost and environmental implications due to pressure on borrow and spoil areas which are getting further and further from locations of construction sites, the use of arable soil as borrow material, increase in carbon footage resulting from burning fossil fuel or cost due to damage to the environment resulting in an increase in greenhouse gases, responsible for global warming (Yang & Zheng, 2006; Prusinski & Bhattacharja, 1999). Cutting and replacement has also meant the use of high fills for roads in flood plains so as to ensure that the main pavement structure is well above the flood levels, resulting in high costs of construction. Re-alignment of roads in urban areas and densely populated areas is very costly, since it would mean expensive land acquisition and compensation to those who own the land where good alignment soils may be located. These methods are therefore not sustainable. For these reasons this study undertook to look into local suitable methods of stabilizing expansive soils to withstand flooding conditions.

Stabilizing low bearing capacity flood plain clays using lime activated ground granulated blast furnace slag (GGBS) can give a strong and durable soil which can withstand long term curing and flooding. (Obuzor, Kinuthia & Robinson, 2011; Obuzor et al., 2012). Use of GGBS and lime cannot be cost effective in Kenya because there is no local smelting of iron ore. Thermal treatment of expansive clays to date has not given specific heating temperatures or strength achieved. The heating times that have been given by some researchers are too long. Some researchers have focused on commercial clays. This research therefore studied the effectiveness of use of lime-activated low calcium pozzolanic heated soil on one part and lime-activated heated soil mixed with molasses including effectiveness of heated soil alone as a stabilizer, to provide cost-effective local in-place methods of treating naturally occurring black cotton soil.

1.3 Objectives

1.3.1 Main Objective

To determine the suitability of use of blends of heated expansive clay soil, lime, and molasses for soil stabilization of expansive clay soils as fill material in construction in Kenya.

1.3.2 Specific Objectives

1. To determine the effect of heat on expansive clay soils.
2. To determine the effectiveness of different proportions of heated expansive clay soil, lime and molasses as stabilizers on the physical, mechanical and electro-chemical properties of expansive clay soils for use as fill material in flood prone environments
3. To evaluate the economic viability of using heated expansive clay soil, lime and molasses as stabilizers for expansive clay soils.

1.4 Scope and Limitations

1.4.1 Scope

This study covered stabilization of black cotton soil located in Mwhoko (Kiambu County), about 25 Northwest of Nairobi, with heat and various proportions of heated expansive clay, lime and molasses. Since black cotton soils vary from one location to another, laboratory tests were conducted to establish the properties of the black cotton soil under study, and thus the need for its improvement. The ideal heating temperature was obtained by analyzing the soil heated at various temperatures. An optimal stabilized mix for the soil to be used as a fill material in road construction was established. The effect of long term curing and prolonged flooding on the optimum mix was then established on laboratory samples. In order to further test the optimal mix established for road fills, a model building with the ground floor slab on a stabilized fill was constructed and the settlement and heave measured over a long term curing period. The economic viability of stabilization was also be determined.

1.4.2 Limitation

The expansive clay tested was that located in Mwihiko, Kiambu County. Soils from other parts of the country were not tested since clay properties vary from one location to another. However, the level of treatment for other expansive clays would be expected to be of the same as established by this research since behavior of expansive clays is similar.

1.5 Significance of the Study

In Kenya and the world over, treatment of black cotton soil with lime is the primary method, other than replacement with suitable materials and realignment of the road. In buildings, the tradition has been replacement of the black cotton soil with suitable material. However, as stated in section 1.2 above, lime stabilized materials are ineffective in areas subjected to flooding while replacement is expensive and has heavy negative environmental implications. M’Ndegwa, (2011) has demonstrated that molasses can be used to treat expansive soils. However, treatment with molasses also has challenges in wet areas (M’Ndegwa, 2011; O’Flaherty, 1974).

Most roads will experience wet conditions and some degree of flooding of the sides during the rainy season. Some roads may experience flooding and wet conditions due to heavy rains or failure of drainage systems. It is common to have ground water level rising during the wet seasons. Therefore, the fill material in roads and buildings will always experience some flooding and wet conditions during the rainy seasons in areas where the expansive clays are prevalent due the poor drainage properties of these soils. This will also be experienced in areas where the level of the water table is high. The implication of this is that fills stabilized with lime and molasses will fail after some time meaning that the roads and the buildings constructed using these fills will need to either be reconstructed or be maintained at high costs resulting in high life cycle costs.

Where the method of construction adopted is avoidance by excavation and replacement with suitable materials or realignment of the road, the cost of construction is usually very high. The excavation and replacement also results in considerable damage to the environment on the borrow and disposal areas.

The implications of these construction methods namely, treatment with lime or molasses and avoidance by replacement or alignment is that fewer roads and buildings would be constructed for any amount of allocated funds. Moreover, as stated above, the damage to the environment is very high where the method of construction is avoidance of the expansive clay by excavation and replacement. Therefore these methods are not sustainable.

Successful use of heated black cotton soil in-place and stabilization of black cotton soil using mixes of lime and heated soil which this study demonstrated as possible will have a great positive impact on the construction of roads and buildings in areas where there is black cotton both in terms of cost and on the environment. More durable roads can be constructed, more jobs created and the black cotton soil which is normally a waste used effectively. There would be a reduction in the high costs associated with avoidance of expansive clays by excavating and replacement in construction of building and roads or re-alignment in construction of roads, a reduction in the negative impact on the environment caused by excavation and spoiling expansive clay materials in dump sites; there would be a reduction in burning fossil fuel hence improvement in carbon footprint; there would also be relief on arable land used for borrows sites.

This will clearly have a major impact in Kenya in road construction, contribute significantly to achievement of vision 2030 (economic pillar on efficient infrasture) as well as contribute to UN agenda on Sustainable Development Goals, particularly goals 9 on resilient infrastructure and goal 11 on resilient and sustainable cities and human settlements, as well as converting waste to riches in using black cotton soil, a waste for useful purposes.

1.6 Organization of the thesis

This thesis comprises of five chapters. Chapter one covers the background information on the research, the statement of the problem, the research objectives, the scope and limitations and the significance of the study.

Chapter two provides the literature review which outlines the properties which make expansive clays a problematic soil, the current methods used to improve the

properties of these soils and their limitations, and the need to research into other methods of improving these soils. The method of evaluating the cost of the product of the research is also discussed.

Chapter three covers the research methodology adopted for the study which describes in detail the materials used, the experimental set up, data collection and analysis methods. The economic viability of using the stabilized soil as fill material is also included.

Chapter four covers the results obtained and a critical discussion of these results including the practical implication of the results. The conclusions drawn from the research are presented in Chapter five including the recommendations and suggestions for further research work .

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

This chapter outlines the challenges encountered when construction is done where black cotton soils are prevalent. The minerals which contribute to the behaviour of black cotton soils are discussed, including their origin and formation. The properties which make black cotton soil a problematic soil are brought out. The methods which are currently used to improve the properties of these soils are discussed together with their limitations, and hence the need to research into other methods of improving these soils. The current research trends focusing on enhancing stabilizers with alkali-activators such lime in one-part geopolymer reactions using aluminosilicate precursors is brought out, which is the focus of this study, in addition to thermal stabilization. The method of evaluating the cost of the product of the research is also discussed.

2.2 Expansive Soils

Expansive soils are soils that change in volume as the water content changes. They exhibit significant swell and shrinkage potentials. They contain expansive clay minerals such as smectite (Jones & Jefferson, 2012). According to Mokhtari and Dehghani (2012) expansive clays contain the clay mineral the montmorillonite with claystones, shales sedimentary and residual soils and are capable of absorbing great amounts of water and expand. They also shrink when they dry out, causing fissures which help water to penetrate when water is present causing a cycle of shrinkage and swelling making the soil undergo great amount of volume changes. The swelling and shrinkage are not fully reversible processes, and thus the cracks formed during swelling do not close up perfectly in re-wetting, leaving spaces which enhance access to water for the swelling process (Holtz & Kovacs, 1981).

According to Sridharan and Prakash (2000) expansive clays are characterized by the presence of a large proportion of highly active clay minerals of the montmorillonite group which are responsible for the pronounced volume change capability of the

soils when levels of moisture change. The montmorillonite subgroup is the most common of the smectite group of clay minerals (Mitchell & Soga, 2005). Nontronite which is a smectite and vermiculate also impart expansive characteristics. In general, the larger the amount of the minerals which impart expansive characteristics, the greater the expansive potential of the soil. However the presence of other non-swelling minerals such as quartz and carbonate may reduce the expansive effects (Kemp, Merriman & Bouch, 2005). The type and amount of clay minerals in a soil, the water content, the internal structure, the vertical stresses and the void ratio are the main factors which affect the volume change potential of the soil (Estabragh, Parsael & Javadi, 2015). The mineral composition of a soil depends on its origin and formation (Knappet & Craig, 2012). Black cotton soil is an expansive clay (Mamatha & Dinesh, 2017; Satyanarayana et al., 2016; Mehta et al., 2014).

2.3 Origin and Formation of Clay Minerals.

Soil is formed from rock either by physical or chemical processes. Physical processes may be erosion by the action of wind, water or glaciers or disintegration caused by alternative freezing and thawing in cracks in rocks. The resultant soil particles retain the same composition as that of the parent rock. Chemical weathering processes on rock particles are due to the action of water, oxygen and carbon dioxide causing changes in the parent rock, resulting in formation of groups of crystalline particles of colloidal size (less than 0.002 mm), known as clay minerals (Knappet & Craig, 2012). The systematic process of clay mineral synthesis is schematically shown in Figure 2.1.

The clay minerals are hydrates of aluminium, iron or magnesium silicate generally combined in such a manner as to create sheet-like structures only a few molecules thick. These sheets are built from two basic units, the tetrahedral unit of silica (Figures 2.1) and the octahedral unit of the hydroxide of aluminium, iron or magnesium (Figures 2.1). In clay mineral structures, an octahedral sheet of aluminium is called a gibbsite while a magnesium one is called a brucite (Knappet & Craig, 2012; Smith & Smith, 1998; Mitchell & Soga, 2005). Many of tetrahedral and the octahedral sheets piled together form a single clay particle. Since each of the

sheets has a definite thickness without limitations in the directions at right angles to the thickness, the clay minerals exhibit a flat or curved terraced surface. The behaviour of the clay particles when they interact with water and the various materials dissolved in water defines their engineering properties (Terzaghi et al., 1996).

2.4 Types of Clay Minerals

The formation mechanism shown in Figure 2.1 results in three main groups of clay minerals, depending on how the sheet-like structures are stacked together. These groups are the kaolinite, the mica-like and the smectite group. The illite is the most common of the mica-like group whereas the montmorillonite subgroup is the most common of the smectite group. Each group comprises various subgroups, with members of the same group generally having similar engineering properties (Mitchell & Soga, 2005; Smith & Smith, 1998). However, most soils usually consist of more than one type of clay mineral, and are thus called mixed-layer clays (Mitchell & Soga, 2005). This research looked into the improvement of expansive clays. These are identified by the presence of smectite minerals, particularly the montmorillonite subgroup or albites together with montmorillonite

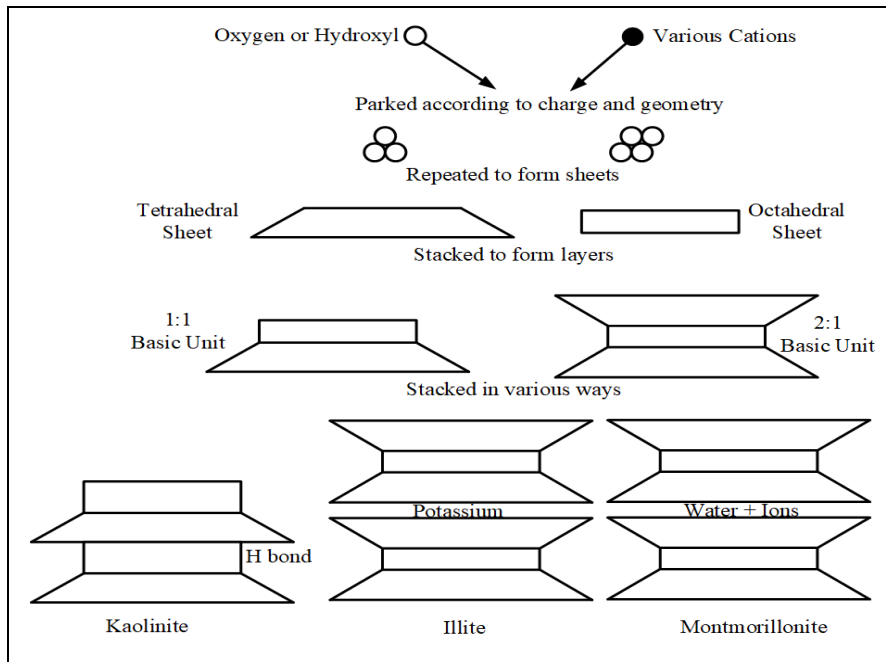


Figure 2.1: Schematic drawing showing clay mineral formation.

(Source: Chittoori, 2008)

2.4.1 Kaolinite Minerals

Kaolinite formation occurs when alumina is abundant and silica is scarce (Mitchell and Soga, 2005), and consists of a single sheet of silica tetrahedrons combined with a single sheet of alumina octahedrons, forming a two layer structure and thus called 1:1 minerals (Figures 2.1 and 2.2). The basal spacing is 7.2 \AA (0.72 nm). The silica:alumina sheets are held tightly together by hydrogen bonding; a kaolinite particle may consist of over one hundred stacks (Knappet & Craig, 2012; Mitchell & Soga, 2005). Kaolinites are very stable with a strong structure, absorb very little water and have low swelling and shrinkage characteristics (Smith & Smith, 1998). The structural formula is $(\text{OH})_8\text{Si}_4\text{Al}_4\text{O}_{11}$ (Mitchell & Soga, 2005).

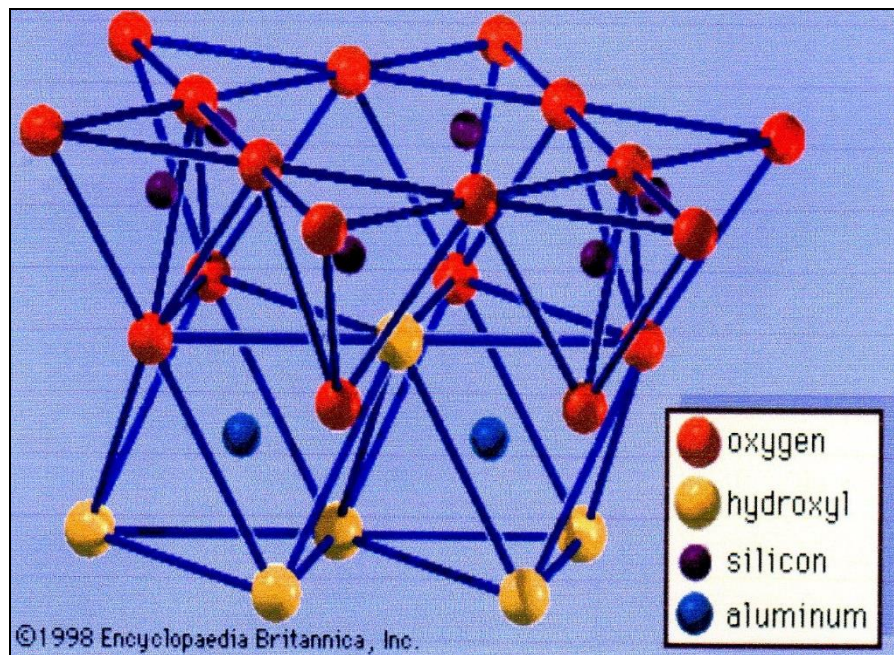


Figure 2.2: Diagrammatic sketch of the kaolinite structure

(Source: <http://media-2.web.brittanica.com/eb-media/76/2676-004-3893834B.gif>)

2.4.2 Micalike Clay Minerals

Micalike clay minerals mainly comprise muscovites, illites and vermiculates. They are sometimes referred to as illite because illite is the most commonly found clay mineral in soils encountered in engineering practice (Mitchell & Soga, 2005). The basic structure of illite and muscovite consists of a sheet of alumina octahedrons sandwiched between two tetrahedral sheets of silica and thus referred to as 2:1 mineral (Figure 2.1 and Figure 2.3).

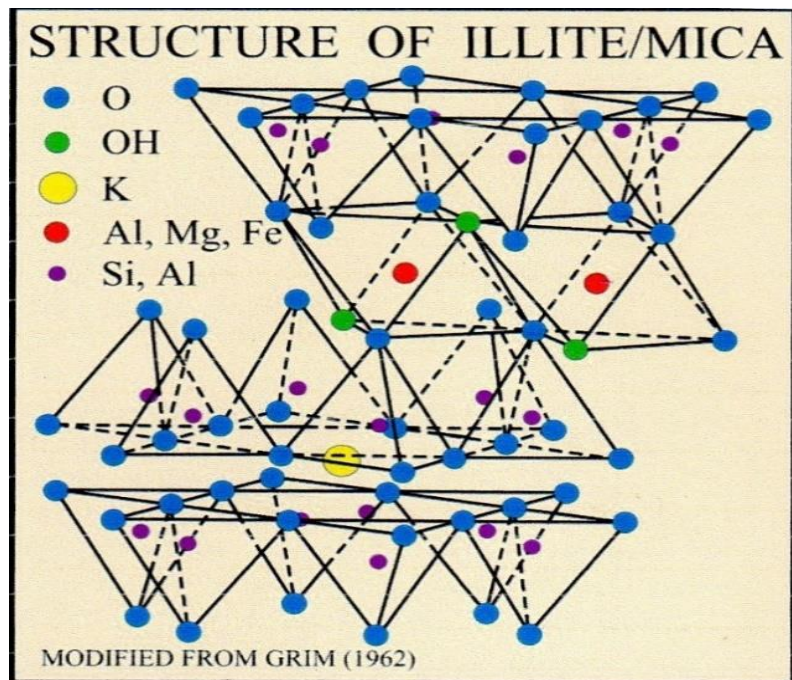


Figure 2.3: Structure of illite/Mica clay mineral

(Source: <http://pubs.usgs.gov/of/2001/of01-041/htmldocs/images/illstrc.jpg>)

The basal spacing for muscovites and illites is 10 Å. Some of the aluminium is partially replaced by iron and magnesium in the octahedral sheets, while in the tetrahedral sheets, there is partial replacement of silicon by aluminium. The structural formula is $(K,H_2O)_2(Si)_8(Al,Mg,Fe)_{4,6}O_{20}(OH)_4$. The sheets are combined together by fairly weak bonding due to non-exchangeable potassium ions held between them. Since the potassium ions are positively charged, the negative charge to balance them comes from the substitution of aluminium for some silicon in the tetrahedral sheets, a process called isomorphous substitution where one element is substituted for another without the changing the crystalline form. Illites have higher swelling and shrinkage characteristics than kaolinites, and tend to absorb more water. (Knappet & Craig, 2012; Das, 2008; Mitchell & Soga, 2005; Smith & Smith, 1998). Vermiculate differs slightly from illite in that the octahedral sheets are mainly magnesium, with the basic units combined together by exchangeable magnesium or calcium which form a double water layer which gives a basal spacing 14 Å The interlayer bond is weak. The structural

formula for vermiculate is $(\text{OH})_4(\text{MgCa})_x(\text{Si}_{8-x}\text{Al}_x)(\text{MgFe})_6\text{O}_{20} \cdot y\text{H}_2\text{O}$ where $x \approx 1$ to 1.4, and $y \approx 8$ (Mitchell & Soga, 2005).

2.4.3 Smectite Minerals

Smectites are 2:1 minerals consisting of a sheet of alumina octahedrons sandwiched between two tetrahedral sheets of silica just like the illites. The basal spacing varies from 9.6 Å to complete separation. In the absence of isomorphous substitution, the composition is $(\text{OH})_4\text{Si}_8\text{Al}_4\text{O}_{20} \cdot n(\text{interlayer})\text{H}_2\text{O}$. Montmorillonite subgroup (Figures 2.1 and 2.4) is the most common of the smectite group.

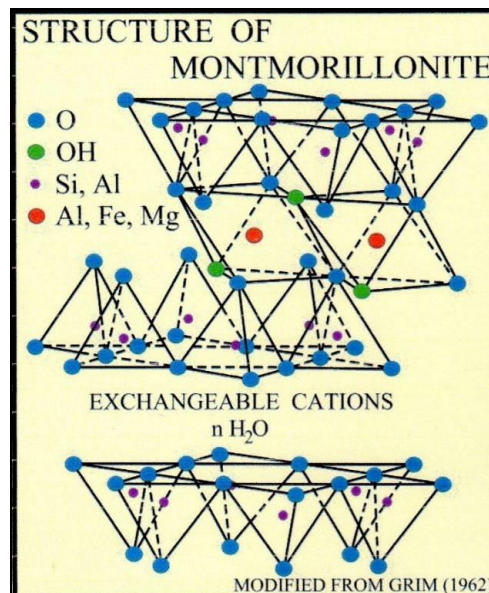


Figure 2.4: Diagrammatic sketch of the montmorillonite structure

(Source: <http://pubs.usgs.gov/of/2001/of01-041/html/docs/images/monstru.jpg>)

In montmorillonite, some of the silicon is replaced by iron, magnesium and aluminium in the tetrahedral sheets. The spaces between the combined sheets is occupied by water molecules and exchangeable cations other than potassium. These ions make the bond between the combined sheets very weak. Additional water can be adsorbed between the combined sheets giving montmorillonite extremely high water absorption, swelling and shrinkage characteristics (Smith & Smith, 1998; Mitchell & Soga, 2005; Knappet & Craig, 2012). The percentage of smectite minerals in a soil especially montmorillonite

largely contributes to its swelling characteristics (Mokhtari, & Dehghani, 2012; Jones & Jefferson, 2012).

2.4.4 Chlorites

The structure of the chlorite is like that of the vermiculate except that an octahedral sheet replaces the double water layer between the various layers, with the basal spacing having a value of 14 Å. Chlorites always occur in mixture with other clay minerals (Mitchell & Soga, 2005).

2.4.5 Mixed-Layer Clays

Most soils usually consist of more than one type of clay mineral. Interstratifications of two or more layer types often occurs within a single clay particle because of the great similarity in crystal structure among the different minerals. The interstratification may be regular, with a definite repetition of the different layers in sequence, or it may be random. The most common mixed-layer clay is montmorillonite-illite; others are chlorite-vermiculate and chlorite-montmorillonite (Mitchell & Soga, 2005). Weaver and Pollard (1973) as cited in Mitchell and Soga (2005) state that randomly interstratified clay minerals are only second to illite in abundance. The properties of mixed-layer clay minerals are representative of the constituent minerals (Terzaghi et al., 1996).

2.5 Isomorphous Substitution and Cation Exchange in Clay Minerals

The combination of the octahedral or tetrahedral sheets as discussed in 2.3 above to form clay minerals is only ideal; some of the tetrahedral and octahedral spaces are occupied by cations other than those in the ideal structure, such as aluminum in place of silicon, magnesium instead of aluminum, and ferrous iron (Fe^{2+}) for magnesium. There is no change in crystal structure and thus the process is called isomorphous substitution. The actual tetrahedral and octahedral cation distributions may develop during initial formation or subsequent alteration of the mineral (Mitchell & Soga, 2005).

Mainly as a result of isomorphous substitution of aluminium or silicon atoms by atoms of lower electrovalence in all of the clay minerals with the possible exception of those

in the kaolinite group and also due to disassociation of hydroxyl ions, clay particles have a net negative charge. Broken bonds at the edges of the particles also give rise to unsatisfied charges. Cations present from salts in the soils pore water are therefore attracted and held between the layers and on the surfaces and edges of the particles so as to preserve electrical neutrality. Many of these cations are exchangeable because they may be replaced by cations of another type since they are weakly held. The cations can be arranged in a series in terms of their affinity for attraction generally in order of valency, as follows: $Al^{3+} > Ca^{2+} > Mg^{2+} > NH_4^+ > K^+ > H^+ > Na^+ > Li^+$. Thus, Al^{3+} ions can replace Ca^{2+} and K^+ can replace H^+ and so on. The quantity of exchangeable cations is termed the cation exchange capacity (CEC) which is a measure of the net negative charge and is usually expressed as milliequivalents (meq) per 100 g of dry clay (Mitchell & Soga, 2005; Das, 2008; Knappett & Craig, 2012). Table 2.1 gives cation exchange capacity of common clay minerals, showing that montmorillonite has the highest cation exchange capacity. Generally, because of the large amount of unbalanced substitution in the smectite minerals, they have very high cation exchange capacities (Mitchell & Soga, 2005; Terzaghi et al., 1996).

Table 2.1: Cation Exchange Capacity of Principal Clay Minerals

Mineral	Cation Exchange Capacity (meq/100g)
Kaolinite	3-10
Illite	20-30
Chlorite	20-35
Hydrated halloysite (member of the Kaolinite group)	40-50
Montmorillonite	80-120

(Source: Terzaghi et al., 1996)

2.6 Effect of water in clay.

Exchangeable cations in clays are discussed in 2.5 above. When water surrounds a clay particle, the exchangeable cations are not attached to it. The negative electrical

charge on the particle tends to attract cations as counter ions that are not uniformly distributed throughout the pore fluid but are highly concentrated close to the surface of the clay particle forming a small but definite volume of pore fluid which is different from the extended liquid. These counter ions are not rigidly held, but tend to diffuse into the pore liquid phase for a layer thick enough to balance their charge and that on the particle surface. The distributed charges in the solution phase and the negatively charged surface are together termed the diffuse double layer as illustrated in Figure 2.5. The water surrounding the clay particles is called double-layer water which gives the soil its plastic properties (Figures 2.6). The double layer water comprises the excess of cations and their associated anions. The concentration of the ions in the double-layer water decreases inversely with the distance from the clay particle surface (Figure 2.5).

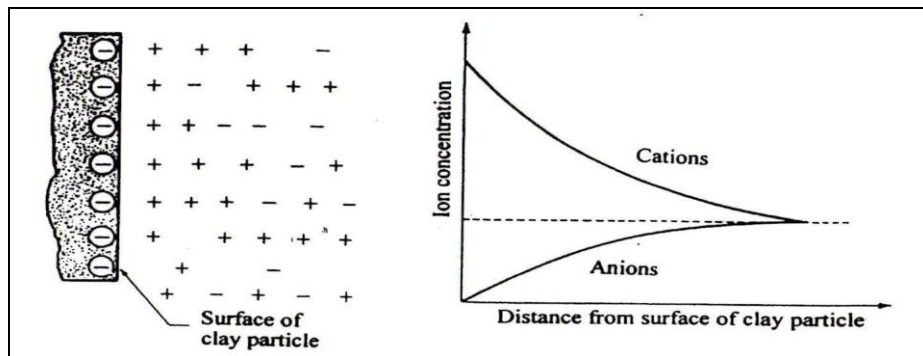


Figure 2.5: Diffuse double layer

(Source: Das, 2008).

As shown in Figure 2.6(b), the thickness of the double layer in montmorillonite is very high compared to the thickness of the crystal resulting in a very weak bond between the particles which explains the high swelling behaviour of montmorillonite in water. The innermost layer of the double-layer is called adsorbed water, and is very strongly held by clay particles. Adsorbed water molecules can move relatively freely parallel to the particle surface but in a restricted manner perpendicular to the surface. Water in clay will therefore exist as chemically bound water (in form of hydroxyl ions - OH, which is integral, indivisible part of the chemistry of the clay

particle), double layer water and free water (Li, Wu & Hou, 2014; Knappet & Craig, 2012; Das, 2008; Mitchell & Soga, 2005; Terzaghi, et al., 1996).

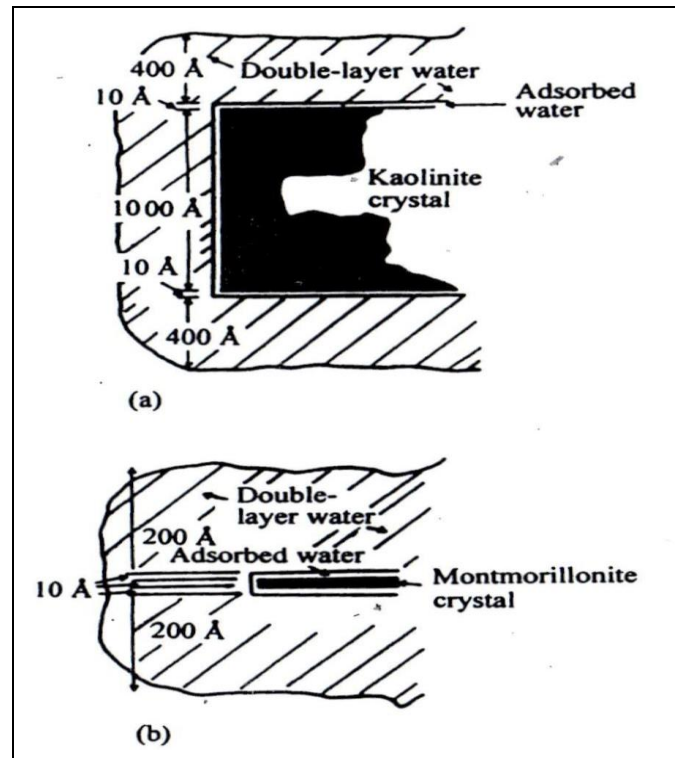


Figure 2.6: Clay water (a) typical kaolinite particle (b) typical montmorillonite particle

(Source: Lambe, 1960)

The ion distribution close to a charged surface is described by the Gouy-Chapman theory (Das, 2008; Mitchell & Soga 2005). This theory assumes that the ions in the double layer can be treated as point charges with no interactions between them and that the charge on the particle surface is well distributed and that the surface of the clay particles is large compared to the double layer. From this theory the average electric potential (Φ) in the diffuse double layer varies inversely with distance x from a charged surface as shown in Figure 2.7. In clays, Φ is negative because of the negative surface charge. The potential at the surface is designated Φ_0 , and for a large enough distance $\Phi = 0$. The repulsive potential Φ_0 decreases with the increase of ionic concentration n_0 and ionic valence v for a given value of Φ_0 and x as shown in Figures 2.8 and 2.9 respectively.

Thus if the exchangeable cations are of low valence such as Na^+ , thick double layers develop in free pore water of low electrolyte concentration such as in freshwater lakes and rivers, whereas the double layer is suppressed if the exchangeable cations are of high valence such as Ca^{2+} and in electrolyte concentration such as in a marine environment. Generally, a soil particle will be covered by a 1 nm layer of adsorbed water, surrounded by 1 to 50 nm of double-layer water, which is in turn enveloped by free water. Double-layer water is most significant in sodium montmorillonite since it has very small and filmy particles (Terzaghi et al., 1996). Figure 2.6(a) shows the typical case for montmorillonite

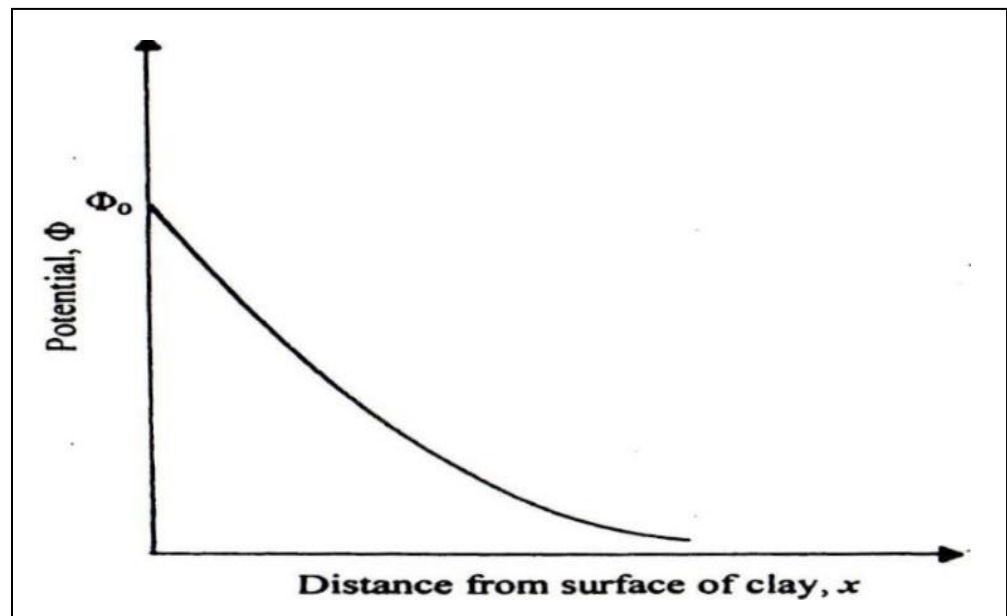


Figure 2.7: Variation of electric potential with distance from the clay surface

(Source: Das, 2008).

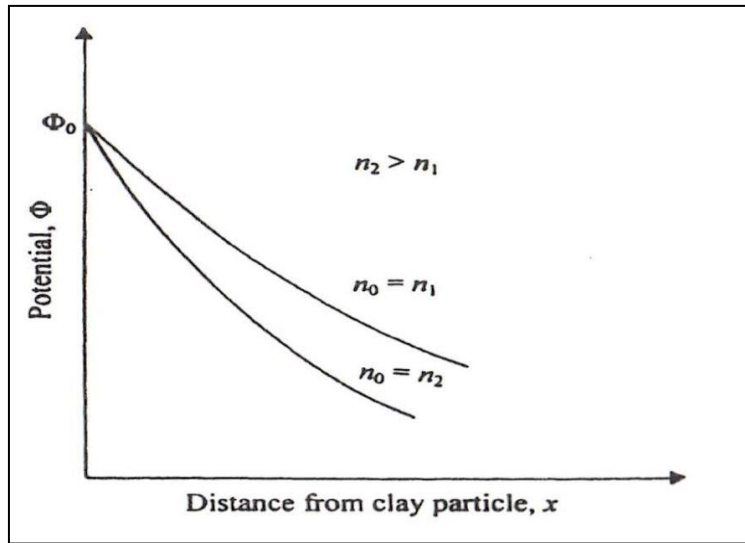


Figure 2.8: Effect of cation concentration on the repulsive potential

(Source: Das, 2008).

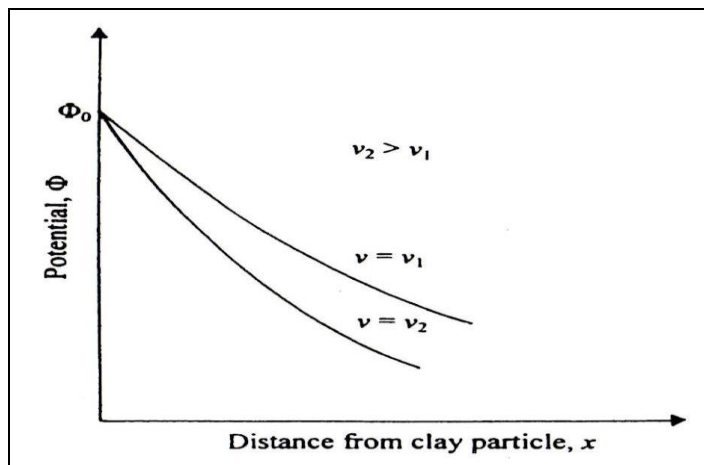


Figure 2.9: Effect of ionic valence on the repulsive potential

(Source: Das, 2008).

In the double layers, repulsion forces occur between like charges, depending on the characteristics of the layers. There is also attraction between the particles as a result of Van der Waal's forces which for two parallel plates vary inversely as $1/x^3$ to $1/x^4$ where x is the distance between the plates, and is independent of the constitution of

the water separating the clay particles. The interaction of the forces is illustrated in Figure 2.10. If the net force is repulsive, the particles settle individually and the soil is said to be in a dispersed state. On the other hand, if the net force is attractive, flocs are formed, and the clay is referred to as a flocculated clay. However, in the natural state the clay forms are more complex than this since the clay contains significant proportions of larger, bulky particles.

The presence of expansive clay minerals in a soil usually exerts influence on the properties of the soil far greater than their percentage by weight (Knappet & Craig, 2012; Das, 2008). Zhao, Ge, Petry and Sun (2014) found that 13% of montmorillonite caused a clay to be expansive while Skippervik (2014) established that albite imparted swelling characteristics on a soil in the presence of montmorillonite and water. In this research, the type of clay minerals present in the soil under test were determined to establish their influence on the behaviour of the soil.

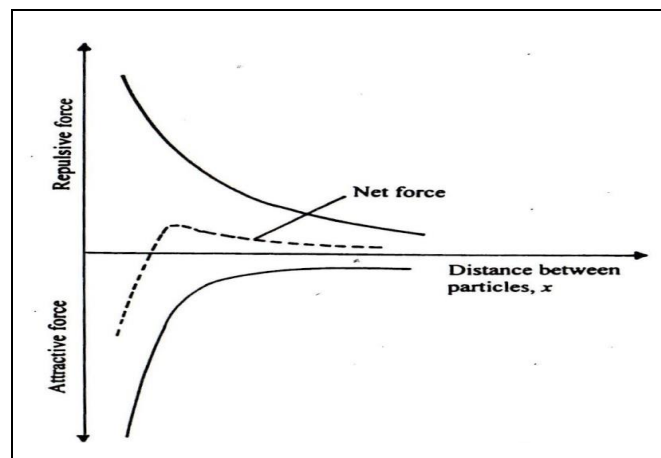


Figure 2.10: Dispersion and flocculation of clay in a suspension.

(Source: Das, 2008)

2.7 Mineralogy of soils using X-ray Diffraction (XRD) Analysis

According to Mitchell and Soga (2005) each crystalline material has its own characteristic atomic structure which diffracts X-rays in a characteristic pattern. Thus, comparing the diffraction pattern of a material under test with a spectrum for a

known material helps identify minerals in the former material. In this study, the measurement was used to identify the minerals present and especially the montmorillonite which is the main minerals in expansive clays (Murthy, 2007; Mitchell & Soga, 2005; West, 1995). The analysis included the relative quantities of the minerals for the neat, heated and stabilized soils. The analysis is based on Bragg's Law (Mitchell & Soga, 2005), given in Equation (2.1) and based on Figure 2.11.

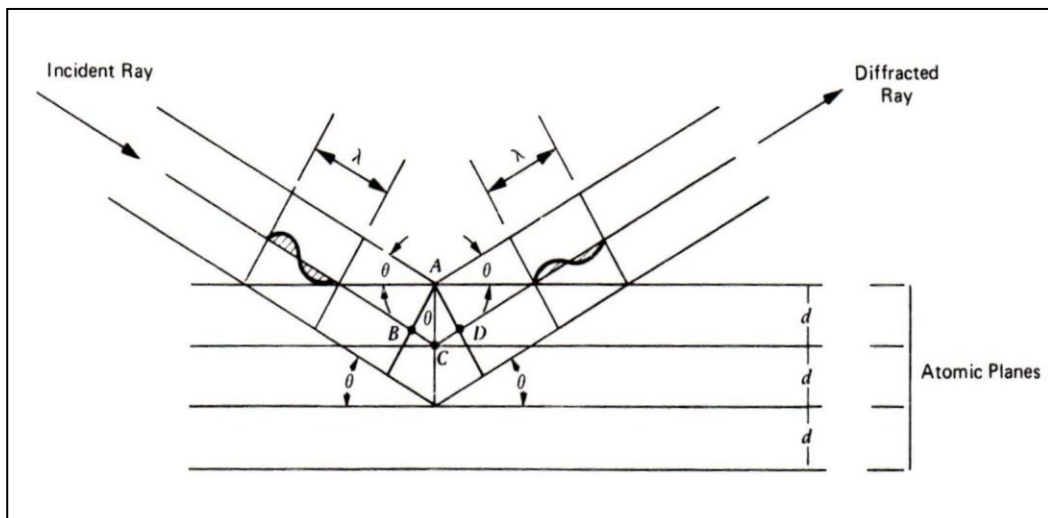


Figure 2.11: Geometrical conditions for X-ray diffraction according to Bragg's law

Source: (Mitchell & Soga, 2005)

Bragg's Law is given as:

$$n\lambda=2d\sin\theta \quad (2.1)$$

Where:

n is an integer, representing the order of reflection

d is the spacing between the atomic planes (basal spacing)

λ is the X-ray wavelength

θ is the angle of the ray incident angle on a fixed crystal plane.

Bragg's law relates the wavelength of electronic radiation to the diffraction angle θ and the crystal plane spacing d . An X-ray beam incident on a pair of parallel planes at an angle θ with a spacing d will give a diffracted beam of maximum intensity if the waves represented by the diffracted rays are in phase, which represents the point at which Bragg's Law is obeyed. A crystalline sample in powdered form is scanned through a range of 2θ angles to attain all possible diffraction directions since the powdered material is randomly oriented. The diffraction peaks are converted to d -spacing to allow identification of the mineral since each mineral has a set of unique d -spacing. Each value of $\frac{d}{n}$ will give a different value of λ , and therefore several values of 2θ will be obtained giving different intensities for the same mineral. These values have been established for the common minerals and are available in standard databases, such as the one maintained by the International Center for Diffraction Data (Li et al., 2014) for inorganic and organic materials.

Various investigators have used XRD analysis to determine the mineral composition of soils. Li et al. (2014) investigated the main clay minerals in an expansive clay soil at various temperatures varying from room temperature to 140°C using XRD analysis and established that in the main components were montmorillonite, illite, kaolin, quartz and feldspar in percentages of 13, 39, 31, 9 and 8 respectively. These ratios did not change within the range of temperature used. Ravichandran, Krishan, Javani, Annadurai and Gunturi (2015) used XRD analyses to establish the minerals present in both neat and stabilized soils, while Prandel, Dias, Saab, Brinatti, Giarola and Pires (2017) used XRD analysis together with other methods to characterize kaolinite in a hard setting Brazilian soil, obtaining the percentage of kaolinite among other parameters.

In this study, XRD analysis was used to establish the minerals present, especially the expansive montmorillonite and albites in the soil under study, together with their relative quantities for the neat, heated and stabilized soils.

2.8 Soil classification

Soil classification places soils within a limited number of behavioural soil groups on the basis of the grading and plasticity characteristics determined from soil tests that are carried out on disturbed soil samples. Grading and plasticity provide very useful guides as to how a disturbed soil will behave when used as a construction material in earthworks, under various conditions of moisture content. The basic soil types are boulders, cobbles, gravel, sand, silt and clay which are defined in terms particle size ranges as shown in Figure 2.12. Others are organic clay, silt or sand and peat. Composite types are mixtures of the basic soil types (O’Flaherty, 2002; Knappett & Craig, 2012). The particle size of a soil is based on its mineralogy (Mitchell & Soga, 2005). Expansive clays are fine and highly plastic.

Clay	Silt			Sand			Gravel			Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse		
	0.002	0.006	0.02	0.06	0.2	0.6	2	6	20	60	200
	0.001	0.01		0.1		1		10		100	
Particle size (mm)											

Figure 2.12: Particle Size Ranges.

(Source: Knappett & Craig, 2012)

The soil engineering classification systems that are in widest use today internationally are based on the Casagrande system, devised in 1947 by Casagrande (O’Flaherty, 2002; Knappett & Craig, 2012). The British Soil Classification System (BSCS) is superior to the Casagrande system (BS1377 Part 2:1990). It was therefore adopted for this study.

Under the BSCS, the soil is divided into two main categories: coarse and fine. Soils having at least 35 percent by mass of material finer than 0.06 mm (passing the 63 micron test sieve) are fine while those having less than 35% passing the 63 micron test sieve are coarse. The other subgroups and are as illustrated on .Table 2.2, which is a summary of the BSCS.

Fine soils are classified using their liquid and plastic limits, by making use of the plasticity chart associated with the BSCS shown in Figure 2.13. The diagonal line known as the A-line represented by Equation 2.2 empirically classifies the fine soils as either clays (C) if above the line or silts and organic silts (M) if below the line. The chart is divided into five ranges of liquid limit and classification letters allocated to the soil depending on the plasticity. However the A-line is not a rigid boundary between clay and silt for purposes of soil description (Knappet & Craig, 2012).

Table 2.2: Summary of the BSCS for Engineering Purposes (after BS 5930:1999)

Soil groups		Subgroups and laboratory identification				
GRAVEL and SAND may be qualified Sandy GRAVEL and Gravelly SAND etc. where appropriate		Group symbol	Subgroup symbol	Fines (% less than 0.06 mm)	Liquid limit %	Name
COARSE SOILS less than 35 % of the material is finer than 0.06 mm	GRAVELS More than 50 % of coarse material is of gravel size (coarser than 2 mm)	G	GW	GW	0 to 5	Well graded GRAVEL
			GP	GPu GPg		Poorly graded/Uniform/Gap graded GRAVEL
		G-F	G-M	GWM GPM	5 to 15	Well graded/Poorly graded silty GRAVEL
			G-C	GWC GPC		Well graded/Poorly graded clayey GRAVEL
		GF	GM	GML, etc	15 to 35	Very silty GRAVEL; subdivide as for GC
			GC	GCL GCI GCH GCV GCE		Very clayey GRAVEL (clay of low, intermediate, high, very high, extremely high plasticity)
	SANDS More than 50 % of coarse material is of sand size (finer than 2 mm)	S	SW	SW	0 to 5	Well graded SAND
			SP	SPu SPg		Poorly graded/Uniform/Gap graded SAND
		S-F	S-M	SWM SPM	5 to 15	Well graded/Poorly graded silty SAND
			S-C	SWC SPC		Well graded/Poorly graded clayey SAND
SF		SM	SML, etc	15 to 35	Very silty SAND; subdivided as for SC	
	SC	SCL SCI SCH SCV SCE		Very clayey SAND (clay of low, intermediate, high, very high, extremely high plasticity)		
FINE SOILS more than 35 % of the material is finer than 0.06 mm	Gravelly or sandy SILTS and CLAYS 35 % to 65 % fines	FG	MG	MLG, etc		Gravelly SILT; subdivide as for CG
			CG	CLG CIG CHG CVG CEG	< 35 35 to 50 50 to 70 70 to 90 > 90	Gravelly CLAY of low plasticity of intermediate plasticity of high plasticity of very high plasticity of extremely high plasticity
	SILTS and CLAYS 65 % to 100 % fines	FS	MS	MLS, etc		Sandy SILT; subdivide as for CG
			CS	CLS, etc		Sandy CLAY; subdivide as for CG
		F	M	ML, etc		SILT; subdivide as for C
			C	CL CI CH CV CE	< 35 35 to 50 50 to 70 70 to 90 > 90	CLAY of low plasticity of intermediate plasticity of high plasticity of very high plasticity of extremely high plasticity
ORGANIC SOILS	Descriptive letter 'O' suffixed to any group or sub-group symbol.		Organic matter suspected to be a significant constituent. Example MHO: Organic SILT of high plasticity.			
PEAT	Pt	Peat soils consist predominantly of plant remains which may be fibrous or amorphous.				

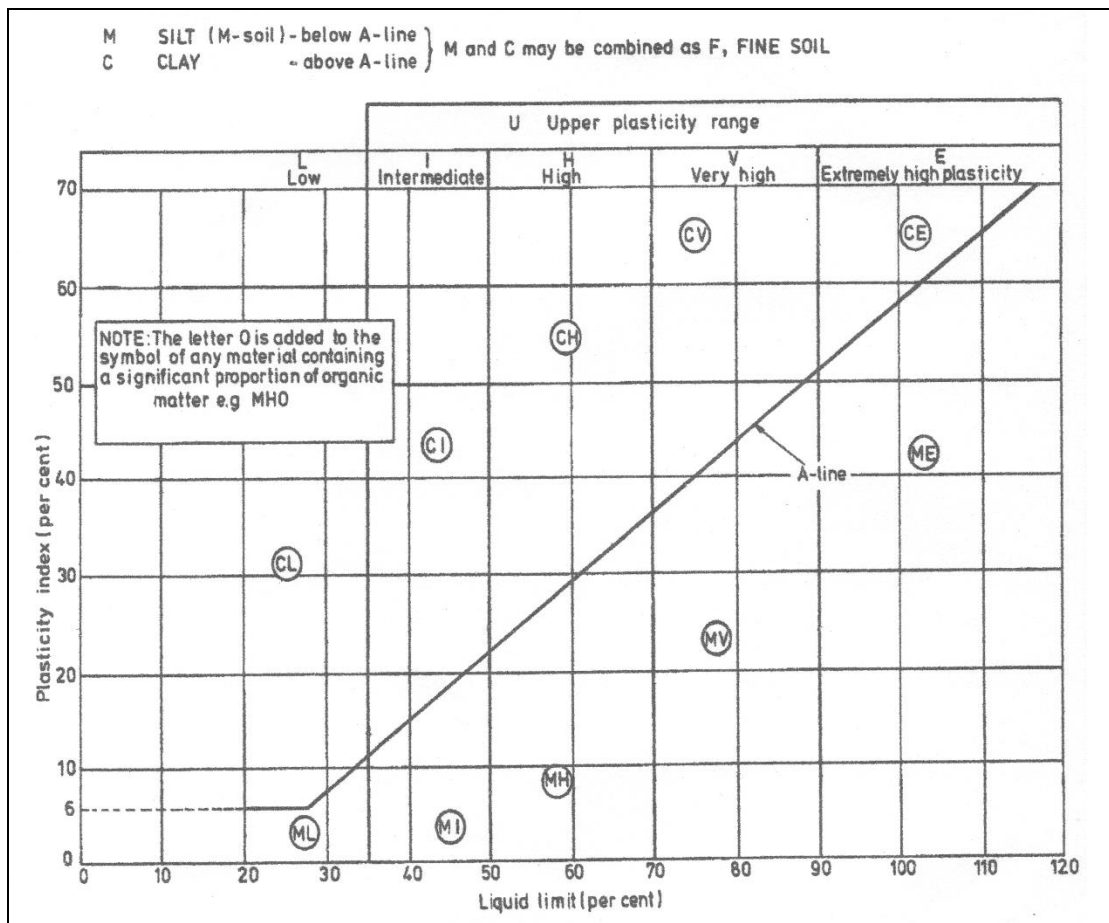


Figure 2.13: Plasticity Chart for fine soils and finer part of coarse soils.

(Source: BS5930, 1999)

$$I_P = 0.73(W_L - 20) \quad (2.2)$$

Where

I_P is the plasticity index

W_L is the liquid limit

Particle size distribution curves also indicate whether the soil is well or poorly graded, and if poorly graded whether uniform or gap-graded, with symbols in capital as W, P, Pu and Pg respectively. The symbol O is added to soil if it contains a significant amount of organic matter. When describing a soil, symbols are used from a combination of the ranges of characteristics, starting with the letter designating the

main soil type, followed by other letters further describing the soil. Thus CHO denotes organic CLAY of high plasticity; CIS denotes sandy CLAY of intermediate plasticity while SP_uF denotes uniform clayey sand, and so on. (O’Flaherty, 2002; Knappett & Craig, 2012; Smith & Smith, 1988). Plasticity and grading which are required before a soil can be fully classified, are discussed in sections 2.8.1 and 2.8.2 respectively.

2.8.1 Plasticity of soils

Expansive soils are highly plastic. Plasticity is the ability of a soil to undergo deformation without cracking or crumbling. It is only important in fine-grained soils, and is enabled by the presence of a significant content of clay minerals which allow these soils to be remoulded in the presence of some moisture without crumbling. A soil may exist either in liquid, plastic, semi-solid or solid states depending on the water content. The water contents at which these states exist differ from soil to soil.

Atterberg (1911) as cited in Smith and Smith (1988) and in Das (2008) evolved methods of determining the various values of water content at which fine-grained soils change their state, and are thus called Atterberg limits. They include the liquid limit, the plastic limit and the shrinkage limit. The liquid limit is the water content as a percentage at which the soil changes from a liquid state to a plastic state. The water content at which the soil changes from a plastic to a semisolid state is called the plastic limit, and the water content at which the soil changes from a semisolid to a solid state is called the shrinkage limit. The plastic index is defined as the range of water content within which a soil is plastic, and is computed as difference between the liquid limit and the plastic limit. In this research, these tests were determined for the neat, heated and treated soils.

2.8.2 Particle Size Distribution/Grading.

Expansive soils are in the fines category comprising clays and silts, usually with a high clay content. The grain size distribution of soils is determined by sieve analysis up-to 0.063 mm, below which the distribution is obtained by sedimentation (BS 1377 Part 2:1990; Knappett & Craig, 2012; Smith & Smith, 1988). A plot on a semi-logarithmic chart (referred to as a grading chart) is done of the percentage of material

passing each sieve against the sieve sizes to give the grading (see Figure 2.14). From this distribution, other soil parameters can be determined. These include the effective size (given the symbol D_{10}), the coefficient of uniformity (C_u) and coefficient of curvature (C_c) defined as follows:

$$C_u = \frac{D_{60}}{D_{10}} \quad (2.3)$$

$$C_c = \frac{D_{30}^2}{D_{10} \times D_{60}} \quad (2.4)$$

Where

D_{10} is the particle diameter at which 10% by mass is finer

D_{30} is the particle diameter at which 30% by mass is finer

D_{60} is the particle diameter at which 60% by mass is finer

From the distribution, it is also possible to establish the grading pattern of the soil. According to O'Flaherty (2002), a soil is either well graded, uniform graded or poorly graded. It is well graded if the coefficient of uniformity C_u is 5 or more and the coefficient of curvature C_c is between 1 and 3. A uniform soil has a small range of sizes with a steep gradation curve, and is poorly graded if C_u is equal to or less than 2. A soil which may have a near-horizontal "hump" or "step" in its gradation is poorly graded indicating that it is missing some intermediate sizes; it is called gap graded. Well graded soils compact to a dense mass.

A typical BSCS classification chart including group symbols for selected soils is illustrated in Figure 2.14, which also illustrates D_{10} , D_{30} and D_{60} .

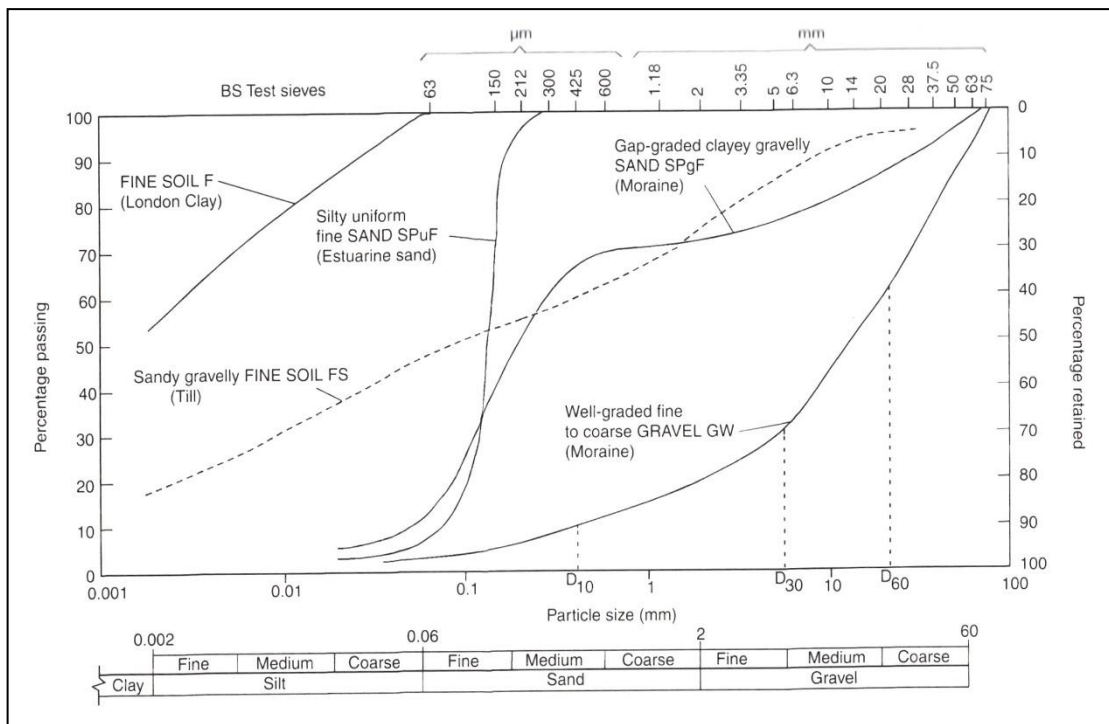


Figure 2.14: BSCS grading chart for soils, with grading curves of selected soil types.

(Source: BS5930, 1999)

2.9 Properties of Expansive Soils

Properties of expansive soils vary from soils located in one location to another. There are however properties which distinguish expansive soils from other soils, thus assisting in their identification. The properties discussed in this section are that identify a soil as expansive. These are the clay mineralogy and the engineering properties. The latter are the physical, mechanical and chemical properties. The treated soil was assessed against the same properties to establish the degree of improvement.

2.9.1 Mineralogy

As discussed in section 2.3, the mineralogy of a soil depends on its origin and formation. Usually majority of soils comprise crystalline minerals which are mainly non-clay, meaning that the percentage of crystalline clay minerals in a particular soil may be fairly

low. In spite of this, the influence of these minerals on the properties of the soil exceeds their proportion by far (Knappet & Craig, 2012). The mineralogy of a soil controls its size, shape, physical and chemical properties. (Mitchell & Soga, 2005). Thus the mineralogy highly influences the expansive behaviour of a clay. Soils containing a considerable amount of smectites clay minerals especially the montmorillonite sub group exhibit high swelling and shrinkage characteristics (Murthy, 2007). Clay minerals such as vermiculites and some mixed clays also impart expansive characteristics. Illites and kaolinites only cause volume changes when the particles are extremely fine (Mokhtari & Dehghani, 2012).

Albite is a tectosilicate consisting of SiO_4 -tetrahedrons in a three dimensional framework with a 1:2 ratio between silicon and oxygen elements. This structure of the tectosilicates enable for a volume increase due to the interlayer spacing between the unit layers. Provided other factors influencing on the swelling such as presence of water and montmorillonite are fulfilled, the swelling potential is mobilized (Skippervik, 2014).

2.9.2 Engineering Properties

As discussed in section 2.2 and 2.9.1, the main challenge associated with expansive clay is the shrink-swell behaviour. The deformations so caused are significantly greater than those can be predicted using classical elastic and plastic theory, meaning that different approaches to predict and treat expansive soils have been developed (Jones & Jefferson, 2012). Further, expansiveness has no direct measure and therefore comparisons under known conditions are made to express expansive behaviour (Gourley, Newill & Schreiner, 1994). This section discusses the methods used to classify expansive clays which help to predict their behaviour and thus help devise appropriate treatment methods. Sabuni (2015) outlined test results from a number of researchers as shown in Table 2.3. The results show that the tested parameters vary from one location to another. Sabuni (2015) also tabulated results for the soils they tested from several locations in western Kenya, as shown in Table 2.4. These results also reveal that the tested parameters vary from one location to another. It is thus important to test the neat soil before treatment to establish the properties before improvement so that the degree of improvement can be established.

The same tests were done on the heated soil to establish the changes caused by heating.

Table 2.3: Engineering Properties of Black Cotton Soils from Several Researches

Sample No.	W _L	W _P	I _P	LS	MDD (kN/m ³)	OMC (%)	CBR (%)	UCS (kN/m ²)	Ref.
1	170	50	120	38	1400	26	1.54		Bidula, 2012
2	85.23	33.1	52.12		1550	24.7	2.02		Radhakrishman et al., 2014
3	66	28.2	37.8		1520	1.15		112.8	Udayashankar & Puranik 2010
4	62	25					3.8		Gandhi, 2013
5	66.67	38.89	27.47		1580	21.79	2.08		Jadhav & Kulkaini, 2014
6	65	41	24		1570	17.2	1.2	93	Chavan et al., 2014
7	57	31.4	26.5		1610		1.5	166.2	Mudgal et al., 2014
8	74	31	44		1730	19.2			Hassan, 2012
9	77.14	38.55			1590	20.5	3.09		Aripita, 2011
10	61	31	30		1610	20.7	1.92	58	Sabat, 2014
11	47	30	17		1860	11.9	4		Yadu et al., 2011
12	54	13.6	4	19	1860	23.5	4.05	116.3	Hakari et al., 2012
13	66	30	36		1310	17	3		Tu'Unji, 2015
14	69	31	38		1580	25		71.35	Mehrotra & Siddique, 2014

Key: W_L – Liquid , W_P – Plastic limit, I_P – Plasticity index, LS - Linear shrinkage, MDD – Maximum dry density, OMC – Optimum moisture content, UCS – Unconfined compression strength.

(Source: Sabuni, 2015)

Table 2.4: Engineering Properties of Black Cotton from Some Locations in Western Kenya

Location	W _L	W _P	I _P	LS(%)	MDD (kN/m ³)	OMC (%)
Bunyala	61	28	33	17	1324	30
Ndhiwa	54.8	27.18	27.82	12.5	1242	23.4
Dominion	56.8	28.2	25.6	12	1330	27.2
Ahero	60	27	33	14	1400	33

(Source: Sabuni, 2015)

2.9.2.1 Physical, mechanical and chemical / electrochemical properties.

The physical are properties of expansive soils which can be measured include particle size distribution, specific gravity, atterberg limits, free swell (FS). Mechanical properties include compaction characteristics, California Bearing Ratio (CBR) and Unconfined Compression (UCS). The chemical properties are oxides content, organic matter content, loss on ignition, sulphates, pH and chloride content. Cat ion exchange (CEC) which is used to establish the mineralogy has already been considered. These properties vary from one location to another

(a) Physical Properties.

(i) Particle Size Distribution.

As discussed in section 2.8.2, the grain size distribution of soils is determined by sieve analysis up-to 0.063 mm, below which the distribution is obtained by sedimentation (BS 1377 Part 2:1990; Knappett & Craig, 2012; Smith & Smith, 1988). A plot on a grading chart is done of the percentage of material passing each sieve against the sieve sizes to give the grading (see Figure 2.14). From this distribution, the effective size (D_{10}), the coefficient of uniformity (Cu) and coefficient of curvature (Cc) and the grading pattern of the soil can be established. The range of soil sizes is shown in Figure 2.12. Expansive soils are fine, with a large proportion of clay since most silts are non-swelling.

(ii) Specific gravity

This is a measure of how much heavier a soil is than water. High specific gravity will also result in high density. The clay particles will have closer spacing which may be an indicator of greater repulsive forces between particles resulting in larger swelling potential (Mokhtari & Deghani, 2012).

(iii) Atterberg limits

Atterberg limits are the water contents at which fine grained soils change their state (Das, 2008). They are the liquid limit (W_L), the plastic limit (W_P) the plasticity index (I_P) and the linear shrinkage (LS). They are a measure of soil plasticity which is an indicator of swell potential. Soils that show plastic behaviour over wide ranges of moisture content and also have high liquid limits have greater potential for swelling and shrinkage (Mokhtari & Deghani, 2012). Their values vary from region to region. According to Bhavar and Patel (2014) black cotton soils in India have values for liquid limit (W_L), plasticity index (I_P), and linear shrinkage (LS) in the range 50 to 100%, 20 to 86% and 9 to 14% respectively. Atterberg limits are widely used to classify soils since they give a good indication on the soils swell potential. This is discussed in section 2.9.2.2.

(iv) Free Swell.

The free swell is a measure of the volume change of soil due to moisture changes. It was proposed by Holtz & Gibbs (1956).

The free swell increases with plasticity index (Murthy, 2007). Holtz & Gibbs (1956) suggested that soils having free swell values of 100 can cause considerable damage to lightly loaded structures and that soils having a free swell below 50 seldom exhibit appreciable volume change even under light loadings.

(b) Mechanical Properties

(i) Compaction

Compaction is a process whereby the density of a soil is increased by packing the particles more closely together through a reduction in the amount of air, without a

significant change in the volume of water in the soil. Generally, the higher the degree of compaction, the higher is the shear strength and the lower is the compressibility of the soil. By compacting soil under controlled conditions, the air voids in well-graded soils can be almost eliminated and the soil can be brought to a condition where there are fewer tendencies for subsequent changes in moisture to take place (Knappett & Craig, 2012; O’Flaherty, 1974). Typical compaction curves are shown on Figure 2.15.

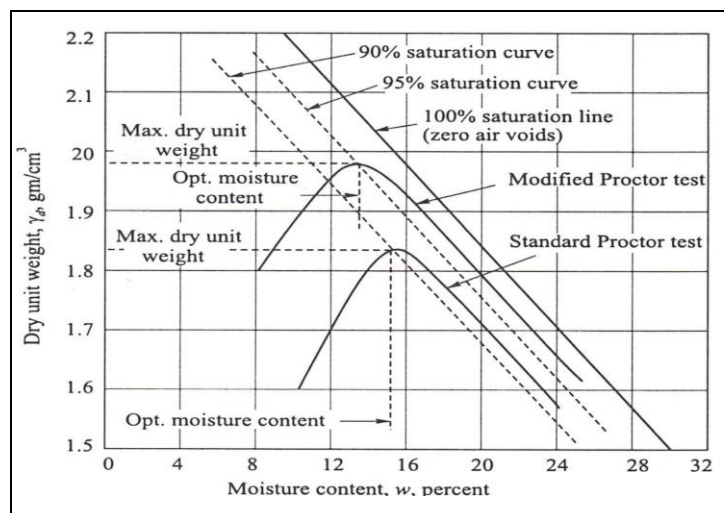


Figure 2.15: Typical Compaction Curves

(Source: Murthy, 2007)

The degree of compaction is measured in terms of the dry density of soil, which is the mass of solids per unit volume of soil. The moisture content of the soil is the mass of water it contains expressed as a percentage of the mass of the dry soil. The dry density of a soil depends on the moisture content and the compactive effort (Knappett & Craig, 2012; O’Flaherty, 1974). Thus, the soil will have a particular dry density at particular moisture content. If water is added gradually as the soil is compacted at the same compactive effort, the dry density will also increase gradually, because the water acts as a lubricant between the soil particles causing them to rearrange into a denser state as compaction continues, resulting in reduction in the air content as the air voids are replaced with water. A maximum dry density

(MDD) is obtained at an “optimum moisture content (OMC)”, beyond which further addition of water will result in reduction of the dry density due to the increase in water which is lighter than soil, since no further air can be expelled. At this optimal point the compaction curve approaches the zero air voids line which is a theoretical curve relating dry density with moisture content for a soil with no air voids (Knappett & Craig, 2012; Das, 2011; O’Flaherty, 1974).

Compaction characteristics of soils are investigated using standard laboratory tests, where the soil is compacted in layers. In accordance with BS 1377:part 4 (1990), the tests are the light compaction test using a 2.5 kg rammer (standard proctor, T99) for soils with particles larger than 20 mm removed, the heavy compaction (modified proctor) test using a 4.5 kg rammer also with particles larger than 20 mm removed and the vibrating hammer method for soils with particles larger than 37.5 mm removed. As shown in Figure 2.15, the curve for the heavy compaction is higher than for the light compaction, with a higher maximum dry density is obtained at a lower moisture content. With the same compactive effort, different soils exhibit different dry density-moisture content relationships as shown in Figure 2.16. Therefore, compaction is not a property of a soil, but depends on the compactive effort. In this research, the light compaction method was used which is the preferred method for soils containing particles up to medium-gravel size (Knappett & Craig, 2012; Smith & Smith, 1998; BS 1377, Part 4: 1990).

When expansive soils are compacted to the levels higher than the optimum moisture content (OMC) by say 3-4% above the OMC, the heave decreases substantially. This can be safe for a slab-on-grade where the probable heave is less than 38mm (Das, 2016).

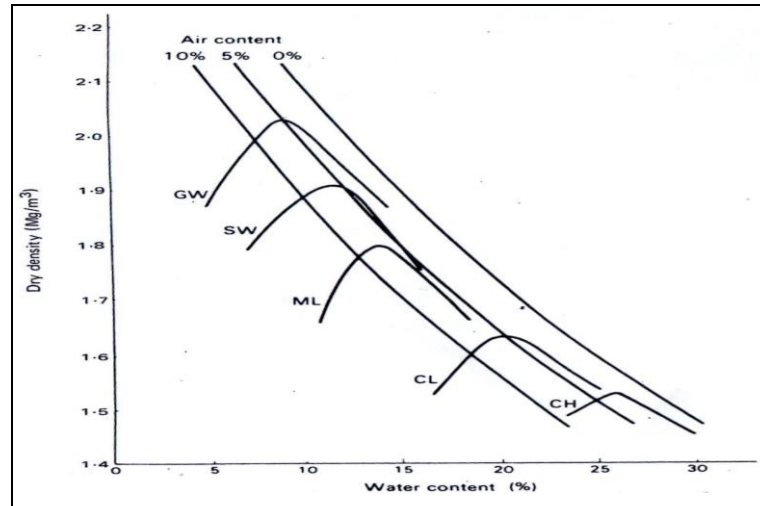


Figure 2.16: Compaction Curves for a range of soil types

Source: Knappett & Craig, 2012)

(ii) California Bearing Ratio (CBR)

The California Bearing Ratio test is a penetration test for evaluation of the mechanical strength of soils. According to BS 1377, Part 4: 1990, the CBR value gives the relationship between force and penetration when a cylindrical plunger of a standard cross-section is made to penetrate the soil at a given rate. The CBR then is defined as the ratio of the applied force to a standard force at certain values of penetration, expressed as a percentage. For untreated soils the test is done in three stages which are compaction, soaking for 4 days and testing. For treated soils, the standard test is done after 7 days cure followed by 7 days of soaking. Black cotton soils are characterized with low CBR values. In Kenya, black cotton soils are classified as subgrade class S1 with CBR ranging between 2% and 5% (MOTC, 1987). CBR values reported by Sabuni (2015) on Table 2.3 are within this range. According to MOTC (1987) measurement of CBR values less than 2% is very uncertain and of little significance. The CBR swell is limited to a maximum of 2% or 3% for a material to be used for fill or subgrade respectively.

(iii) Unconfined Compression Strength Test (UCS)

The Unconfined Compression Strength Test (UCS) measures the compressive strength of a cylindrical soil specimen to which no lateral support is offered. In the test, a cylindrical cohesive specimen is subjected to an axial load which is increased steadily until failure occurs, giving the compressive strength of the soil. Since the minor principal total stress is zero, the shear strength is calculated as half the strength at failure. It is carried out within a short enough time to ensure that no drainage of water is permitted out of the specimen. The test is used for fine grained soils, but is unsuitable for cohesive soils (Knappet & Craig, 2012; Das, 2011; BS 1377, Part 4: 1990; Smith & Smith, 1988). There is currently a trend to use UCS as an alternative to the CBR method for evaluation of soil strength for the purpose of road works (Thomas, 2001). According to Bandara, Mampearachchi and Sampath (2017) the UCS method is better than the CBR method in testing the strength of stabilized soils because the CBR method being an empirical penetration based test can only measure the strength of a soil that does not take tension under loading. However stabilized pavement layers take tension loads. The UCS method was therefore used as the main test to assess the soil strength in this research. Nonetheless, the CBR test was done for the purposes of comparing the strengths of the soil with standards which are in terms of CBR

(c) Chemical properties

Chemical properties considered were mineralogy content as discussed in section 2.8.1, the oxides content obtained from the X-ray Fluorescence (XRF), loss on ignition, organic matter, sulphates and chlorides contents and the soil pH.

(i) Oxides Content

The oxides content gives the main oxides in a soil. The oxides content for clays fall within certain ranges. According to Newman (1987), soils which contain montmorillonite as the main mineral have silicon (SiO_2), aluminium oxide (Al_2O_3), and iron oxide (Fe_2O_3) or magnesium oxide (MgO) as the main oxides. The percentages SiO_2 , Al_2O_3 , Fe_2O_3 , MgO and calcium oxide (CaO) in clays are 48-56, 11-22, >5.0, 4-9.0, and 0.8-3.3 respectively (Bennet & Need, 1969).

(ii) Organic Matter

Organic matter in soils comes from dispersed vegetable matter (Knappet & Craig, 2012). The organic matter may be responsible for high plasticity, high shrinkage, high compressibility, low hydraulic conductivity and low strength. Since organic particles can strongly adsorb on mineral surfaces, they modify the properties of minerals once the adsorption takes place. Soils with high organic content plot below the A-line in Figure 2.13 and are characterized with high liquid limit. An increase in organic content significantly decreases the compacted density and the soil strength but causes an increase in the optimum moisture content (Knappet & Craig, 2012; Mitchell & Soga, 2005). MOTC (1987) limits the organic matter content to 5% for fill material and 3% for subgrades.

(iii) Loss on Ignition

The loss on ignition is a measure of the percentage by mass that is lost from a soil by ignition at 1000°C in an electric muffle furnace to constant mass. In expansive clays the combustible material, the free water, the adsorbed water and the chemically held water are lost.

(iv) Sulphates Content

Sulphates may occur in fine grained soils in form of sodium sulphate, Na_2SO_4 and gypsum, CaSO_4 . High levels of sulphates react with CaO in lime stabilized soils to form ettringite $[\text{Ca}_6\text{Al}_2(\text{OH})_{12}(\text{SO}_4)_3 \cdot 27\text{H}_2\text{O}]$ or thaumasite $[\text{CaSiO}_3 \cdot \text{CaCO}_3 \cdot \text{CaSO}_4 \cdot 14.5\text{H}_2\text{O}]$ which are very expansive materials causing distress in the structures built on the soil, which may lead to failure (Kavak & Bilgen, 2016; Beetham et al., 2015; Bell, 2007; Mitchell & Soga, 2005; Rajasekaran & Rao, 2005). It is therefore important to establish the sulphates in a soil to be treated with lime. Sulphates level in lime stabilized clays should not exceed 2.5% to avoid formation of ettringite or thaumasite (Beetham et al., 2015).

(v) Chlorides

The chloride level has no adverse effect on expansive clays because chlorides are used to stabilize expansive clays; Davoudi and Kabir (2011) used a mix of lime and sodium chloride to stabilize low plasticity fine grain soils, and established that 1% and 2% sodium chloride caused an increase in UCS of 28% and 18% respectively at 28 days. There was no improvement at 3%, with a decrease of UCS at higher percentages. They also established that there is an increase in UCS for soil stabilized with lime in the range 2-8% and sodium chloride in the range 1-5% regardless of the percentage of lime, with the effect becoming negligible after 5% concentration.

(vi) Soil pH

High levels of pH are desirable in lime stabilization; Davoudi and Kabir (2011) state that increase in pH caused by the presence of sodium hydroxide in lime-stabilized low plasticity soil mixed with sodium chloride caused increase in UCS. Kabir (2009) cited in Davoudi and Kabir (2011) states that the solubility of silicates and aluminates which enhances the pozzolanic reaction in lime stabilized soils is a function of the pH of the media. In lime stabilization, sufficient excess lime is added to ensure that the stabilized mixture maintains a pH value of at least 12.4 which is necessary to ensure the long term stability of the hydration products (O'Flaherty, 2002; Dang et al., 2016).

2.9.2.2 Classification of expansive soil based on index tests.

According to Chen (1988) soil property tests can be used for the evaluation of swelling properties of expansive soils. The problems that expansive soils cause in the construction of foundations (potential swell) give the basis for their classification (Das, 2016). The commonly used criteria for determining the swell potential are discussed in this section. Yilmaz (2006) summarized the various universally commonly used schemes for assessing the swelling potential index as shown in Figure 2.17 used in classifying expansive clays. The criteria is based on commonly used indices such as the activity, the percentage of the clay size, the plasticity index, the liquid limit, the soil suction and the soil water content. However they lack standard definitions of swell potential, and are therefore yet to be adopted (Jones & Jefferson, 2012).

(a) Classification based on plasticity and swelling indices

Das (2016) listed extracts from Abduljawad and Al-Sulaimani (1993) who summarized some criteria for identifying swell potential; they give limits for swell potential as shown in Tables 2.5 and 2.6. Table 2.5 gives the swell potential based on the plasticity index (I_p) and shrinkage index (SI) where the shrinkage index is the liquid limit less the shrinkage limit, according to Sowers & Sowers (1970), whereas Table 2.6 gives the swell potential based on the plasticity index (I_p), according to Chen (1988).

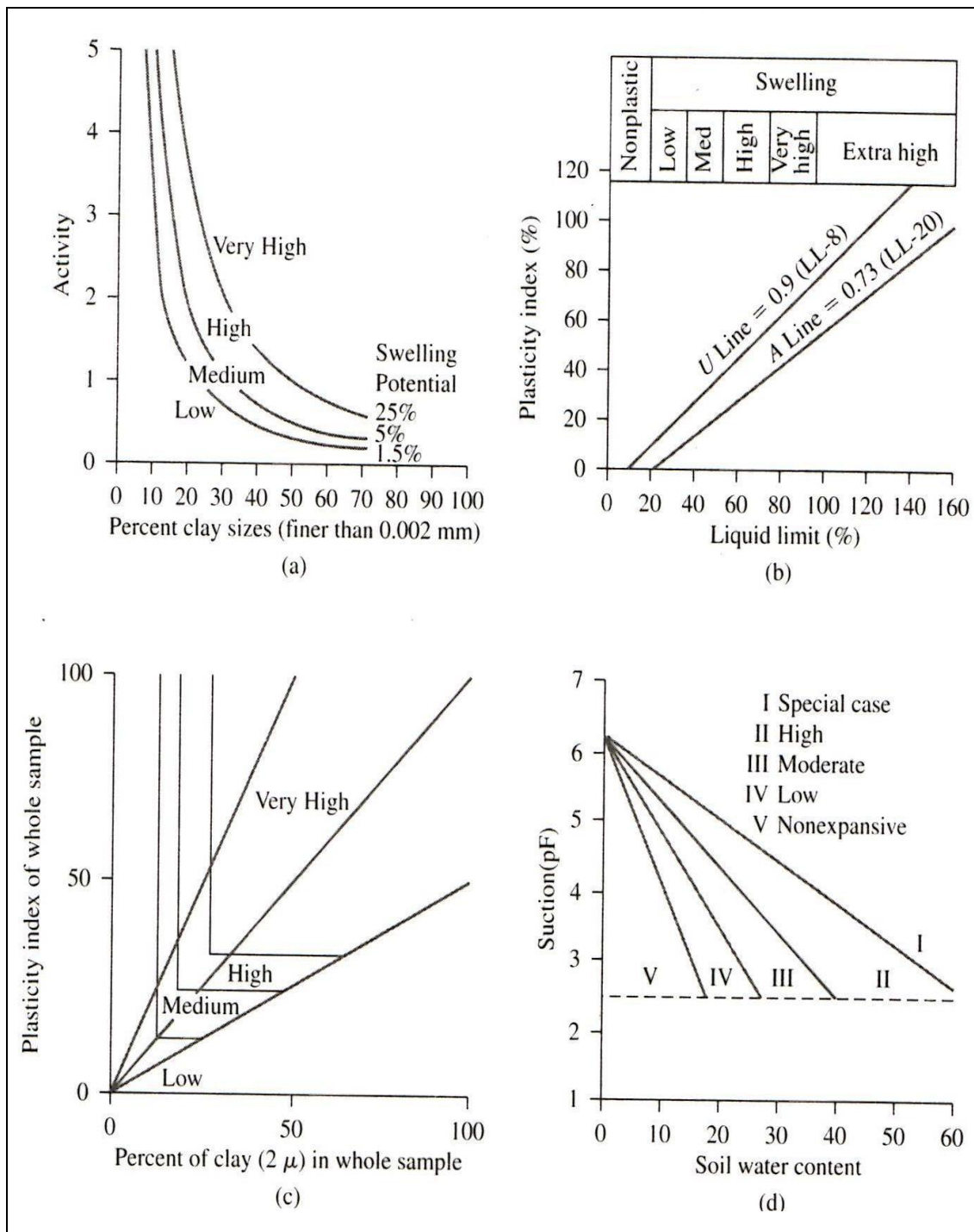


Figure 2.17: Commonly used criteria for determining swell potential from across the world

(Source: Yilmaz, 2006)

Table 2.5: Swell Potential based on Ip and SI

Index Property	Swell Potential
$I_p > 32$ and $SI > 40$	Very High
$23 \leq I_p \leq 32$ and $30 \leq SI \leq 40$	High
$12 \leq I_p \leq 23$ and $15 \leq SI \leq 30$	Medium
$I_p < 12$ and $SI < 15$	Low

Source: Sowers & Sowers (1970)

Table 2.6: Swell Potential based on Ip

Index Property	Swell Potential
$I_p \geq 55$	Very high
$20 \leq I_p \leq 55$	High
$10 \leq I_p \leq 35$	Medium
$I_p \leq 15$	Low

Source: Chen (1988)

Table 2.7 gives extracts from the summary done by O'Neill and Poormoayed (1980) of the classification system developed by the U.S. Army Waterways Experiment Station. This system is the most widely used in the United States (Das, 2016). This research considered these requirements against the local requirements by MOTC (1987) which only require that I_p be less than 50, without any qualifications on W_L .

Table 2.7: Expansive Soil Classification System.

Index Property	Potential	Potential	Swell
W _L	I _P	Swell (%)	Classification
< 50	< 25	< 0.5	Low
50-60	25-35	0.5-1.5	Marginal
> 60	> 35	> 1.5	High

Source: O'Neill & Poormoayed (1980)

(b) Classification based on soil activity

The activity of a clay is defined as the ratio of plasticity index to the clay size fraction as shown in Equation 2.5 (Skempton, 1953 as cited in Mitchell & Soga, 2005 and Bell, 2007).

$$\text{Activity} = \frac{\text{Plasticity Index}}{\text{Percentage by mass finer than 0.002mm}} \quad (2.5)$$

According to Bell (2007) active clays are associated with a relatively high water holding capacity, a high cation exchange capacity, have low permeability and low resistance to shear. Skempton (1953) as cited in Bell (2007) divided activity into three classes, namely, active, normal and active. He further subdivided these into five groups as shown, by splitting the inactive and active clays into two activity levels:

1. Inactive with activity < 0.5
2. Inactive with activity 0.5-0.75
3. Normal with activity 0.75-1.25
4. Active with activity 1.25-2
5. Active with activity >2

Montmorillonitic clays range from inactive to active while kaolinitic and illitic clays are inactive (Bell, 2007). Table 2.8 gives activity levels for various types of clays (Mitchell & Soga, 2005)

Table 2.8: Activities of Various Clay Minerals

Mineral	Activity
Smectites	1-7
Illite	0.5-1
Kaolinite	0.5

Source: Mitchell & Soga (2005)

2.10 Challenges Posed by Expansive Clays in Construction

According to Nelson and Miller (1992) expansive soils are known to exhibit dual characteristics of excessive swelling and shrinkage under different moisture conditions which cause deformations significantly greater than elastic deformations. These deformations cannot be predicted by classical elastic and plate theory. The movements caused are usually of an uneven pattern and of such magnitude as to cause extensive damage to the structures and pavements resting on them. Expansive soils are highly problematic because of their tendency to heave during wet seasons when they absorb water and shrink during dry seasons when water evaporates from them (Muthukumar & Phanikumar, 2015). They are characterized by excessive compressibility, collapse, low shear strength, low bearing capacity, and high swell potential (Rao & Thyagara, 2007). In drier seasons they can form deep cracks and expand dramatically when wet affecting their strength performance as a construction material (Tripathy, Rao & Fredlund, 2002).

Structures constructed on expansive clays undergo severe distress caused by alternate swelling and shrinkage of the soil due to seasonal moisture changes. (Muthukumar & Phanikumar, 2015). Light structures such as road pavements, residential buildings, buried pipes and canal linings are the most affected since they are very sensitive to differential movement (Wiseman, Komornik & Greenstein, 1985; Chen, 1988).

Expansive soil problems occur due to water content changes in the upper few metres of a soil (Nelson & Miller, 1992). Climatic and environmental factors significantly influence the water content of these layers, and is thus termed the active zone (Jones

& Jefferson, 2012; Murthy, 2007). Figure 2.18 shows the water content variation for expansive clays with respect to depth during wet and dry seasons. The active zone is also called the unstable zone, represented by the depth D_{us} in Figure 2.18. During the dry season, the natural moisture content is practically zero at the surface, and the volume of the soil is at the shrinkage limit. The water content increases with depth up-to the depth D_{us} beyond which it remains constant.

During the wet season, the water content is a maximum at the surface, decreasing to a constant value at a depth almost equal to D_{us} . Thus the soil lying within the depth D_{us} is subject to drying and wetting causing large movements in the soil. Structures built within the active zone are likely to move up and down depending on the season, causing damage if differential settlements are considerable (Murthy, 2007). The active zone depth varies from country to country. In the United Kingdom it is 1.5 to 2 m (Jones & Jefferson, 2012), while it can be as much as 15 m in some clays and shales in western USA (Das, 2016). In Kenya, MOTC(1987) requires that for road construction only 1.0 m of the expansive clays be removed and replaced with suitable material.

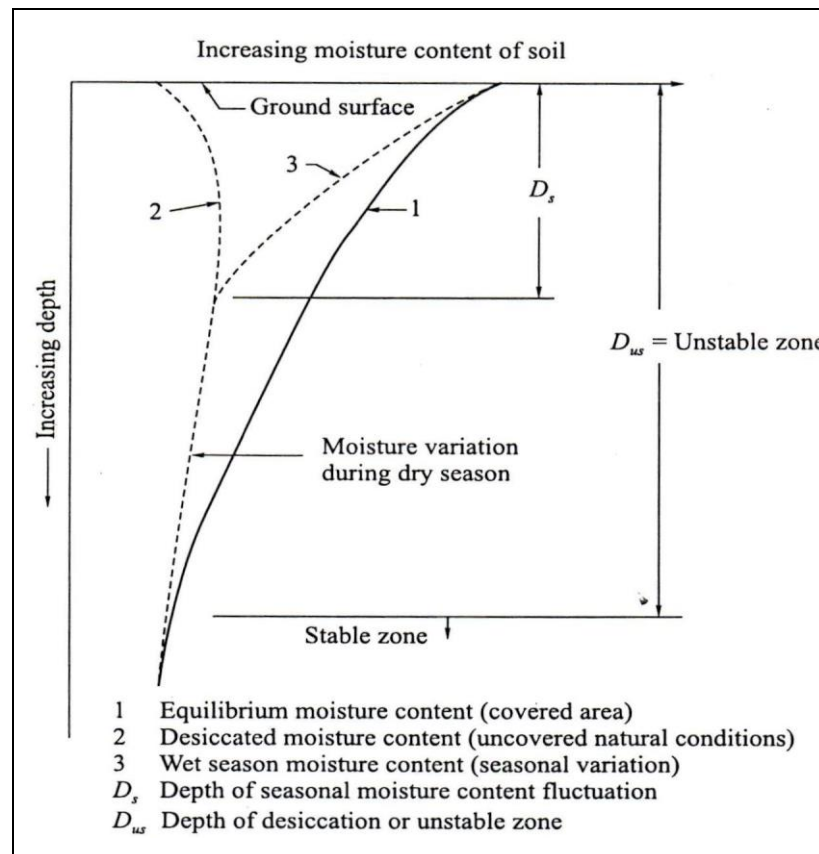


Figure 2.18: Moisture content variation with depth below ground surface

(Source: Chen, 1988)

Holtz & Gibbs (1956) suggested that soils having free swell values of 100 can cause considerable damage to lightly loaded structures and that soils having a free swell below 50 seldom exhibit appreciable volume change even under light loadings. Chen (1988) states that alternate swelling and shrinkage in expansive clays causes lightly loaded civil engineering structures such as residential buildings, pavements and founded on these soils to be severely distressed

According to Chen (1988) and Murphy (2007), expansive soils fall among the six major natural hazards which include earthquakes, landslides, hurricanes, tornados and floods. They tie with hurricane wind/storm surge for the second place as the most destructive natural hazards in the USA in terms of dollar losses to buildings (Behzard & Kalantari, 2012). The adverse effect of shrink-swell behaviour of expansive clays make it the most damaging geohazard in Britain (Jones & Jefferson, 2012). Figure 2.19 shows the distribution of expansive clays worldwide, and they fall

about 45° north and south of the equator, showing that the challenges associated with expansive clays are experienced in many countries.

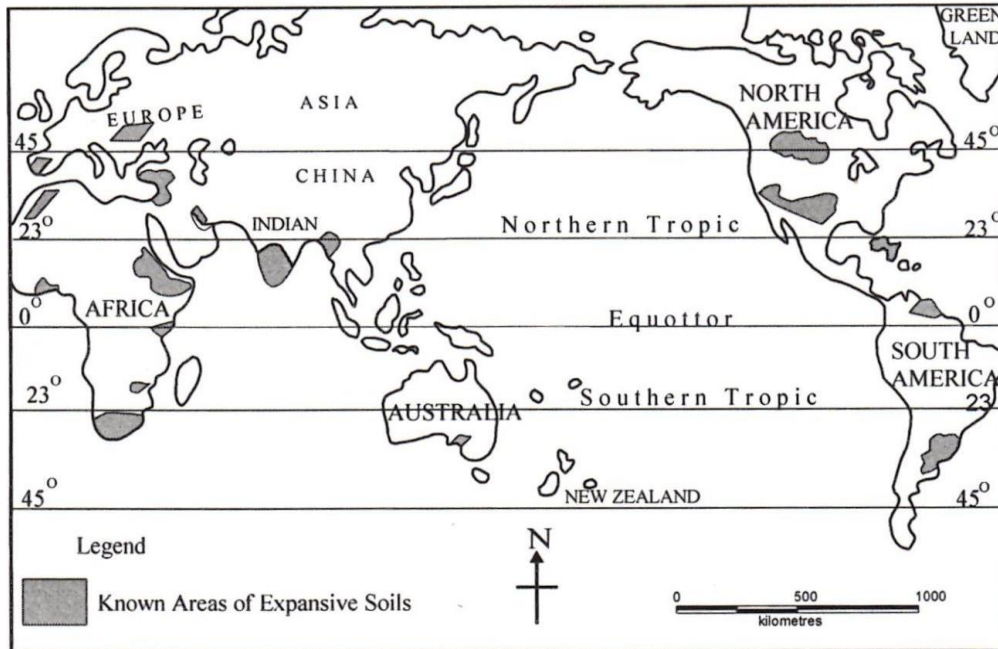


Figure 2.19: Distribution of Expansive Soils in the World

Source: Donaldson (1969)

According to Kalantri (2012) expansive clays are susceptible to cycles of drying and wetting inducing shrinkage and swelling behaviour. The severe damage caused by these soils is mainly dependent on the amount of monovalent cations adsorbed to the clays minerals.

Swelling caused by an increase in moisture content in an expansive clay can cause large uplift forces on structures built on them, inducing heaving, cracking, and breakup of both building foundations and slab-on-grade members. Some of the effects are shown in Figures 2.20 to 2.23. Figure 2.20 shows possible buckling of road pavements induced by swelling in the middle or on the edges of the road; Figure 2.21 shows a residential driveway damaged by swelling in the middle of the road, whereas Figure 2.22 shows cracks on exterior walls for a building built on black cotton soil, while Figure 2.23 shows extensive damage on a road built on black cotton soil, caused by surface run-off due to delayed maintenance. These damages

are typical of structures built on expansive clays. Remedial works and maintenance costs are very high. Some structures built on expansive clays experience failure

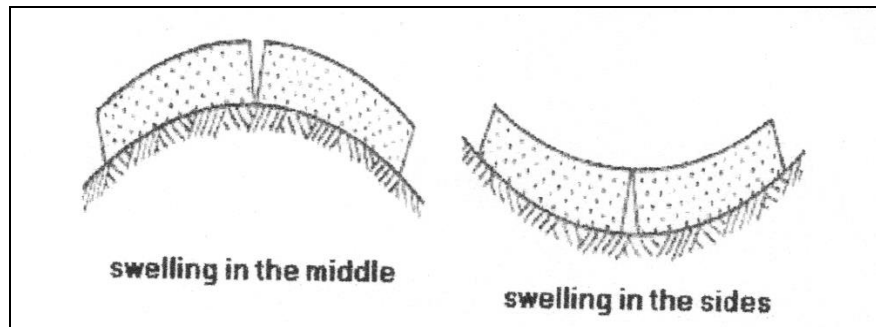


Figure 2.20: Possible Buckling of Pavements on Expansive Clays.

(Source: Kalantari, 2012)



Figure 2.21: Residential driveway damaged by expansive soil

Source: Mokhtari and Dehghani (2012)



Figure 2.22: Cracks in exterior walls for a building built on black cotton at Mavoko (Machakos County)



Figure 2.23: Extensive damage on a road on black cotton soil (Katumani-Vota-Konza Road, Machakos County)

Road pavements are particularly vulnerable to expansive soil damage. Approximately half of the costs of damage caused by expansive soil is on pavements (Chen, 1988). The vulnerability is caused by their relatively light weight which is extended over a large area. In roads, the damages appear in four major forms (Jones & Jefferson, 2012):

- Severe unevenness along significant lengths, where cracks may or may not be visible which is particularly important for airport runways
- Longitudinal cracking
- Lateral cracking which develop from significant localized deformations
- Localized pavement failure due to disintegration of the surface.

Although pavement design is similar to that for foundations for structures, different approaches are required because a pavement cannot be isolated from contact with soils. It is also impractical to make pavements strong to avoid differential settlement. It is thus more economical to treat pavement subgrades (Jones & Jefferson, 2012).

2.11 Kenyan Requirements for fill material for road construction.

According to MOTC (1987), materials to be used for fill in roads should not have organic matter content, CBR swell and plasticity index exceeding 5%, 3% and 50% respectively. The upper 300 mm of fill forms the subgrade, and it is thus desirable that as a minimum the fill material should meet subgrade requirements. MOTC (1987) lowest strength category has CBR in the range 2%-5%, referred to as soil class S1. Values of CBR less than 2% indicate low strength. Black cotton soil is classified in this category. Pavements should not be laid on soils of such poor bearing capacity. The measurement of the bearing strength of such soils is most uncertain and CBR values below 2% are of little significance. Class S1 materials should be avoided.

However, MOTC(1987) recommends that ideally, subgrades should have a CBR value exceeding 5% and a CBR swell less than 2% both at 100% standard compaction and 4 days soak. The organic matter content by weight should be less than 3%. Thus where there are challenges in getting good material, class S1 material with organic matter content, CBR swell and plasticity index not exceeding 3%, 3% and 50% respectively can be used as fill after which a minimum 275 mm layer of improved subgrade can be provided depending on the quality of the improved subgrade material and the required subgrade class. It is noted that MOTC(1986), which is earlier than MOTC (1987) is more stringent, requiring that the top 300 mm of subgrade to have a CBR not less than 8%, and a swell less than 1%. In both

manuals, the plasticity index should not exceed 50%. MOTC (1987) states that the black cotton soil may be stabilized with lime to reduce the plasticity index to less than 20%, the swell to negligible values, increase the CBR to a minimum of 10% after 7 days cure, and the grading to be similar to that of a silt. The stipulation is that a minimum thickness of 300 mm needs to be treated, and that the minimum pavement thickness including the subgrade should be 600 mm. However in this research, consideration was given to treating a layer of 1000 mm including the subgrade since the aim of the research was to ensure the stabilized soil can sustain a prolonged flood, prevalent in low lying areas where the road needs to be raised.

2.12 Treatment of Expansive Clays.

Expansive clays require treatment for them to be useful for use in engineering due to the problems posed by their potential swelling and shrinkage nature. For soils with low swell potential, there may be no need for any special treatment. However, special treatment is required for soils with medium and high swell potential (Das, 2016). According to Chittoori (2008), the most common reasons for treatment for road subgrades and bases, which can also apply to fills material are:

- Reduce shrink/swell of expansive clays
- Increase strength to provide long-term support for pavement structures
- Reduce pavement thickness
- Reduce moisture susceptibility and migration
- Utilize local materials

For roads the following are the options for dealing with expansive clays:

- (i) Avoiding expansive clays area by re-alignment (Jones & Jefferson, 2012; MOTC, 1987)
- (ii) Removing and replacing with suitable non-expansive material (Mohktari & Deghani, 2012; Kalantari, 2012; Jones & Jefferson, 2012; MOTC, 1987)
- (iii) Design for low strength and allow regular maintenance (Jones & Jefferson, 2012)
- (iv) Soil Stabilization. The methods of stabilization are:
 - Compaction

- Mechanical stabilization
 - Thermal treatment
 - Pre-wetting
 - Stabilization through chemical additives/stabilizing agents
- (v) For other structures such as buildings, the options according to Das (2016) are
- Strengthening structures to withstand heave.
 - Constructing structures flexible enough to withstand the differential soil changes
 - Putting up structures on isolated deep foundations below the active zone and suspending the ground floor slab
 - Moisture Control Barriers
 - Grouting/Injection.

2.12.1 Avoiding expansive clays area by re-alignment

Realignment is only possible if the areas covered with expansive clays are of limited extent (Jones & Jefferson, 2012; MOTC, 1987). This is a challenge in urban areas and where development has taken place since it would involve high land acquisition costs. It is thus imperative that a better way of dealing with expansive clay is obtained, which was established by this research.

2.12.2 Replacement with suitable material

Replacement with suitable material involves removing the original top expansive soil and replacing it with compacted non-expansive material within the active zone illustrated in Figure 2.18 (Mohktari & Deghani, 2012; Kalantari, 2012; Jones & Jefferson, 2012; MOTC, 1987). It is only economical for reasonable thickness of expansive soil. Chen (1988) suggests a minimum of 1-1.3m. (MOTC, 1987) recommends that in road construction, excavation of the expansive soil can be done to a depth of about 1 m and carting it to spoil, followed by replacement with a suitable material, of at least class S2 material with CBR of 5%-10%. Failure may still occur if there is ingress of water in the lower layers and an expensive pervious layer may need to be provided stop water ingress (Jones & Jefferson, 2012).

Where the thickness is high or the expansive soil is extensive, excavating and importing good material for replacement has huge cost implications due to increase in carbon footage resulting from burning fossil fuel or cost due to damage to the environment resulting in increase in greenhouse gases, responsible for global warming. Furthermore, good replacement materials put pressure on agricultural land, since they are located in arable land which has led to legislation and imposition of levies for their excavation and transportation making them more and more expensive (Prusinski & Bhattacharja, 1999; Yang & Zheng, 2006; Obuzor et al. 2012). For this reason, this research considered treating the soil in-place instead of removing and replacing with suitable material.

2.12.3 Design for low strength and allow regular maintenance

Roads can be designed to rest on expansive soils, provided they carry light traffic, and provisions are made for regular maintenance (Jones & Jefferson, 2012). Challenges encountered with this is that improvement of roads generates traffic which may exceed that which was expected. Further, in some cases funds provided for maintenance may be inadequate. This method is therefore inadequate. This research therefore recommends treatment of the soil from the results of the research.

2.12.4 Soil Stabilization

Soil stabilization is required when the soil available for construction is not suitable for the intended purpose (Murthy, 2007). It is defined as the alteration of any property of a soil to improve its engineering characteristics and performance (Bandara et al., 2017). The main properties that may be required to be altered by stabilization are below (Chittoori, 2008; Murthy, 2007,):

- Strength- stabilization increases the strength and thus stability and bearing capacity of the soil. The shear strength increases giving the soil ability to resist shear stresses developed as a result of traffic loading;
- Stability- its effect is to control the swell-shrinkage characteristics caused by moisture changes. The physical volume and mass are maintained when the soil is subjected to load or moisture;

- Durability- the process increases the resistance to erosion, weathering or deformation due to traffic usage;
- Permeability- The effect of this is to reduce permeability and hence the ingress and passage of water through the stabilized soil;
- Stabilization- Increases shear strength, reduces permeability and reduces compressibility;
- Resistance to moisture- Stabilization gives the soil ability to resist absorption of water enabling it to maintain shear strength, and modulus and decreasing volumetric swell;
- Modulus (stiffness)- The ability to respond elastically and minimize permanent deformation when subjected to traffic loading is increased.

The various stabilization methods noted in 2.12 (iv) are discussed below. They differ from one soil type to another (Murthy, 2007).

2.12.4.1 Stabilization by Compaction.

The process of compaction is discussed in section 2.4.2.1. Compaction increases the density thus reducing the voids making it possible to achieve a relatively impermeable fill thus minimizing water ingress. Less expansion is observed when soil is compacted at low densities above the optimum moisture content than below (Jones & Jefferson, 2012). The swelling potential and heave decrease substantially when expansive soils are compacted to levels higher than the optimum moisture content (OMC) by say 3%-4% above the OMC. This can also be safe for a slab-on-grade where the probable heave is less than 38 mm (Das, 2016). Target densities can be achieved by standard compaction methods and control. However, the method may not be effective for soils of high swell potential (Jones & Jefferson, 2012). For a slab on grade, the compacted expansive clay can be used in conjunction with a base course of open graded aggregates so long as the slab is independent of the grade beams, columns and walls and the allowable pressure is about the same as the allowable strength of the subsoil (Kalantari, 2012). However clay is more difficult to compact than granular material unless it contains sufficient air voids (Thomas, 2001). Therefore compaction cannot guarantee adequate improvement of expansive clays. For this reason, this study recommends treatment with chemicals.

2.12.4.2 Mechanical stabilization

Mechanical stabilization is the process where two or more soils of different grading are mixed to achieve the required physical properties. The proportion of material added usually exceeds 10% unlike incorporation of stabilizing agents and may be as high as 50%. Mechanical stabilization has drawbacks particularly in regions of high rainfall. In addition, Mechanical stabilization cannot always be achieved and even where it is achieved it is often necessary to further improve the properties of the stabilized material by adding a stabilizing agent (Chittoori, 2008; Sherwood, 1993). The stability of cohesive soils of low strength may be improved by adding coarse material; granular materials that are unstable due to lack of binding materials can be improved by adding fine material. The grading of the mixture is important to ensure that most of the voids are filled (O'Flaherty, 1974). Although this process is an improvement of removing the expansive soil and replacing it with suitable material as discussed in section 2.12.2, it still involves haulage of suitable material for mixing with the expansive clay and haulage to spoil of excess expansive material which is costly. Therefore this study investigated stabilization with admixtures.

2.12.4.3 Thermal stabilization

Thermal change causes a marked improvement in the properties of the soil. Thermal stabilization is done either by heating the soil or by cooling it.

(a).Heating

Various researchers have worked on improvement of expansive soils using heat. Li et al. (2014) investigated the physical, mineralogical and micro morphological properties of an expansive soil heated at different temperatures for 6 hours in the range from room temperature to 140⁰C. They established that in addition to loss of free water, physical absorbed water and chemically bound water, the microstructure experienced some obvious changes. The particle size and the density both increased rapidly up to 100⁰C, but took a slow increase or decline for temperatures higher than 100⁰C. They found that the strength of the heated expansive soil was higher than that of the unheated soil. Wang et al. (2008) found that the swelling potentials of two expansive commercial clays, Edgar Plastic Kaolin (EPK) and Western Bentonite (WB), diminish when heated to about 400⁰C for EPK and 600⁰C for WB. The

predominant clay minerals were kaolinite and montmorillonite. They concluded that heating can effectively stop expansive soils from swelling.

Thermal treatment reduces the swelling capacity and compressibility of clay soils (Bell, 1993). It has been used to prevent shear failure in clay soils and to stabilize clay slopes (Beles, 1957 as cited in Bell, 1993). It has been used to stabilize collapsible loess by increasing its strength appreciably (Litinov, 1960 as cited in Bell, 1993; Terzaghi et al., 1996).

As a clay soil is heated, it loses adsorbed water and hardens and its strength increases. If heated to a high enough temperature, the crystalline structure of clay minerals undergoes irreversible changes and the soil will remain hard (Akinshipe & Kornellius, 2017; Bell, 1993). As the temperature is increased to more than 100⁰C, the adsorbed water is driven off and the strength is further increased. At temperatures of between 200⁰C and 400⁰C soils containing large proportions of organic colloids and colloidal minerals, the effect of water on swelling and shrinkage starts to reduce as fine particles begin to aggregate into granules. When the soil is heated to temperatures of 400⁰C to 600⁰C, some irreversible chemical changes occur which make the soil non-plastic and non-expansive. The clay clods are converted into aggregates. A significant reduction in the plasticity index of clay occurs at 400⁰C. After it has been heated to 600⁰C, the moisture absorption capacity is significantly reduced. Permeability initially increases on heating up to 600⁰C or 700⁰C. Above these temperatures, permeability decreases slightly as fusion and vitrification set in (Bell, 1993). With further increase in temperature, there is some fusion and vitrification, and a brick like material is obtained which can be used as an artificial aggregate for mechanical stabilization.

Heated soil as a stabilizer is very promising (O'Flaherty, 1974). In-situ methods that have been successfully used include driving exhaust gases from burning oil, at temperatures of about 1000⁰C, into holes in the ground. The soil must not be saturated because the latent heat of evaporation of water makes it too expensive (Bell, 1993). This type of treatment has been done for depths of up to 20 m. A borehole of 0.15 to 0.20 m in diameter stabilizes a column of some ten times as wide and depths of up to 10 m can be stabilized in 10 days (Evstatiev, 1988 cited in Bell,

1993). This treatment has competed economically with pile foundations for large structures (Terzaghi et al., 1996).

As an alternative to in-situ methods, electric heaters have been used. In one system compressed air is blown through an electric heater at the top of a borehole, achieving temperatures ranging from 500⁰C to 1200⁰C. In another, the heaters are inserted in the borehole, baking the soil around the borehole to form a column. Interlocking columns can form structural walls and mats. This system treats soils regardless of water content or gas permeability. However, power and fuel consumption are high (Bell, 1993).

The changes that occur in clay soils brought about by heating produces hard durable construction materials with the following properties (O'Flaherty, 1974):

- Reduced compressibility, both soaked and unsoaked, due to decreased water sensitivity
- Increased permeability due to shrinkage of clays by desiccation and the formation of cracks
- Decreased plasticity index and a reduction in attraction of water.

Although heating is a high energy consumer, it has been used to convert waste into useful products. Rashad (2013) in a review of heat conversion of kaolin into metakaolin concluded that production of metakaolin can be done by calcining waste paper sludge or kaolin in temperature range of between 700⁰C and 800⁰C for 1 to 12 hours of heating, with the best temperature range for paper sludge being 600-800⁰C for 2-5 hours. He stated that none of the researches gave an optimal heating temperature. 2 hours was adopted in this research as heating time, based on the minimum of the range of 2-5 hours suggested by Rashad (2013), so as to minimize costs because the 6 hours used by Li et al. (2014) is too high. No strength results have been published to the knowledge of the researcher in the temperatures above 400⁰C which are known to change the chemical structure of soils. This heating period gave acceptable results. Merino et al. (2010) developed simple mathematical equations to predict weight loss and porosity as a function of the treatment temperature, for soils treated up to 900⁰C.

In this research, the effect of heat on black cotton soil was investigated for soil heated to temperatures of 200, 300, 400, 500, 600, 700, 800, 1000 and 1200⁰C for 2 hours in a closed kiln. Further, the effect of stabilizing black cotton with heated soil and various mixes of hydrated lime and molasses was investigated.

(b).Freezing

Cooling causes a small loss of strength of clayey soils due to an increase in inter-particle repulsion. However, if the temperature is reduced to the freezing point, the pore water freezes and the soil is stabilized. Ice so formed acts as cementing agent. Water in cohesion less soils freezes at about 0⁰C. However in cohesive soils, water may freeze at a much lower temperature. The strength of the soil increases as more and more water freezes. This method is used only in some special cases. It has been successfully used to solidify soils beneath foundations. The method is commonly used when advancing tunnels or shafts through loose silt and fine sand. It has been used as aid to excavation below ground water level in permeable soil and rock, due to the strength and impermeability of frozen grounds. However, the method is expensive and rarely justifiable economically (Murthy, 2007; O'Flaherty, 1974; Terzaghi et al., 1996). Therefore effect of freezing was not investigated in this research; it is not practical in the tropics, because the low temperatures cannot be sustained.

2.12.4.4 Stabilization by pre-wetting.

Pre-wetting is increasing the moisture content by ponding such that the soil achieves most of the heave before construction. But since the seepage of water through highly plastic soils soil is slow, this method is slow. 4-5% of hydrated lime may then be added to the top layer of the soil after ponding to make it less plastic and more workable (Gromko, 1974). The main challenge with this technique is that shrinkage will result once the soil eventually dries.

2.12.4.5 Chemical Stabilization

In chemical stabilization, soils are stabilized by adding different chemicals. The main advantage of chemical stabilization is that setting time and curing time can be

controlled. Chemical stabilization is however generally more expensive than other types of stabilization (O'Flaherty, 1974; Bell, 1993; Murthy, 2007).

Chemical stabilization can be brought about in a number of ways (Chittoori, 2008; Petry & Armstrong, 1989):

- The chemical reacts with the soil changing the properties forming a new material with improved properties.
- Bonding the soil particles together by cementing them thus reducing the effect of water on the structure
- By waterproofing thereby maintaining the moisture content at the low levels at which the soil would have the adequate strength required for the intended purpose
- By a combination of waterproofing and bonding.

The following chemicals have been used in varying degrees of success:

(a).Calcium Chloride

When calcium chloride is added to soil, it causes colloidal reaction and alters the characteristics of soil water. Calcium chloride causes a slight increase in the maximum dry density. However, the optimum water content is slightly lower than that of untreated soil. It causes a small decrease in the strength of the soil. The construction methods are similar to those used for lime stabilization. The quantity of calcium chloride required is about 0.5% of the weight of the soil (Bell, 1993). Since calcium chloride causes a decrease in strength it cannot be effective in stabilizing the soil under study whose strength was low for it to be used as a fill or as a subgrade material.

(b).Sodium Chloride

Davoudi and Kabir (2011) used sodium chloride to stabilize low plasticity fine grain soils, and established that 1 and 2% sodium chloride caused an increase in UCS of 28% and 18% respectively at 28 days. There was no improvement at 3%, with a decrease at higher percentages. They attributed the increase with 1% sodium chloride to creation of ionic bands between the soil particles, but as the salt increased, the increased sodium ions resulted in soil dispersing such that at 3%, the UCS was

basically the same as that of the untreated soil, while at 5%, the dispersing effect dominated the ionic effect, resulting in a 44% reduction in strength. They thus concluded that only 1% of sodium chloride should be used, for a strength increase of 28%. They also established that there is an increase in UCS for soil stabilized with 2%, 4%, 6% and 8% lime mixed with 1, 2, 3, 4, and 5% sodium chloride regardless of the percentage of lime. There was an increased improvement as the sodium chloride increased, with the effect becoming negligible after 5% concentration.

Bell (1993) states that the quantity of sodium chloride required for stabilization is about 1% of the soil weight and that on site, the salt should be mixed with the soil either by the mix-in-place method or by the plant-mix method. It should not be applied directly to the surface.

However in this research sodium chloride could not be considered because the strength increase from its use is too low to effectively stabilize the soil under study.

(c).Sodium Silicate

Sodium silicates, as well as other alkali silicates, have been successfully used for soil stabilization. The chemical is used as solution in water, known as water glass. The chemical is injected into the soil. Sodium silicate gives strength to soil when it reacts with it. It also makes the soil impervious. It also acts as dispersing agent. The maximum compacted density is increased. The quantity of the chemical required varies from 0.1% to 0.2% of the weight of the soil (Bell, 1993). Hower, Ayibiowu and Ola (2015) established that 4% sodium silicate increased the CBR of black cotton soil by 192% and the plasticity index reduced by 47%. In the work by Bell (1993) and Ayibiowu and Ola (2015), the effect of the stabilizer on prolonged flooding was not tested which was the subject of this research.

(d).Polymers

Polymers are long chained molecules formed by polymerizing of certain organic chemicals called monomers. Polymers may be natural or synthetic. When a polymer is added to the soil, reactions take place. Sometimes, the monomers are added with a catalyst to the soil. In that case, polymerization occurs along with the reaction (Terzaghi et al., 1996). Mirzababaei, Arulrajah and Ouston (2017) stabilized an expansive clay using two polymers, polyvinyl alcohol (PVA) and Butane Tetra

Carboxylic Acid (BTCA) and found that they improved the UCS and ductility of clay soils significantly. The UCS increased from 10 kPa to 116 kPa with 1.5% addition of PVA at 14 days curing. The treated samples did not disintegrate when soaked in water for 4 days soaking unlike the untreated samples which broke down within 30 minutes. This showed that PVA could be effective in improving the strength of expansive soils with high water contents. BTCA was used mainly for making PVA hydrogels insoluble in water. However, since one of the aims of treating expansive clays is to use local materials, this research aimed to treat the black cotton in-place with a combination of the heated black cotton and locally available lime and molasses.

(e).Chrome Lignin

The chemical lignin is obtained as a by-product during the manufacture of paper from wood. Chrome lignin is formed from black liquor obtained during sulphite paper manufacture. Sodium bicarbonate or potassium bicarbonate is added to the soil and it slowly reacts to cause bonding of particles. The quantity of lignin required varies from 5% to 20% by weight (O'Flaherty, 1974). Canakci, Aziz and Celik (2015) compared the effect of stabilizing expansive soils with lignin, rice husk powder and rice husk ash. They found that for a 7 days curing period, lignin caused the greatest improvement in the plasticity index, but had the lowest UCS value. The values were a significant improvement from the untreated soil for each of the stabilizers. No soaking was done, and the effect of long term flooding was not investigated.

(f).Cement stabilization

Cement stabilization is done by mixing pulverized soil and Portland cement with water and compacting the mix to attain a strong material, whose strength increases with curing time. The material obtained by mixing soil and cement is known as soil-cement. The soil-cement becomes a hard and durable structural material as the cement hydrates and develops strength. The method is more suited for granular and clayey soils with low plasticity, while expansive clays respond better to lime stabilization. (Murthy, 2007; Das, 2016). Since expansive soils have high plasticity, lime stabilization was chosen instead of cement stabilization.

(g).Use of Molasses

M'Ndegwa (2011) carried out studies on the effect of cane molasses on strength of expansive clay soils and he concluded that stabilization of expansive clay soil with molasses reduced its swelling tendencies and increased the CBR value but the CBR values obtained were lower value than those obtained from lime stabilization. Long term performance was not studied. However, according to O'Flaherty (1974) molasses is a cementing agent but since the cement formed is water-soluble, the strength of the stabilized material reduces significantly when subjected to water/moisture conditions common where fills for roads and buildings are done. It performs very well if water can be kept away. In this research therefore the effect of mixes of molasses, lime and heated soil in various proportions was evaluated to assess their performance as stabilizers for black cotton soil.

Rifa'I, Nugroho, and Wicaksono (2006) established that molasses application improved soil-cement reaction with water, causing larger grain sizes to form. The application caused increase in strength as measured in terms of UCS and CBR. No investigations were done on long term curing and flooding. In this study, lime was used instead of cement since lime has a better effect on the plasticity than cement

(h).Lime stabilization

Lime stabilization is done by adding lime to soil. It is useful for stabilization of clayey soils. According to Wang (2002), the ability of a stabilizer to provide a sufficient amount of calcium is the most important feature in stabilization of clayey soils, which is readily available in lime.

The most striking effect of mixing lime to a clay soil is its immediate impact on the plasticity of the soil. It normally results in reduction of the plasticity index, due to the increase of plastic limit and decrease of the liquid limit, as shown in Figure 2.24 for London clay, which is a clay of very high plasticity. Lime addition also causes a decrease in the soil moisture absorption capability resulting in enhanced volume stability. The resulting material has a greater strength, a higher modulus of elasticity as well as lower shrinkage and swelling properties. It is more friable than the original clay, and is therefore more suitable in fills, embankment construction as well as for use as a subgrade and sub-base material in road construction (Das, 2016; Murthy,

2007; Bell, 1993; O’Flaherty, 2002). However, lime addition to clayey soils increases its strength to a certain limit after which addition of lime causes a decrease in strength (Bell, 1996).

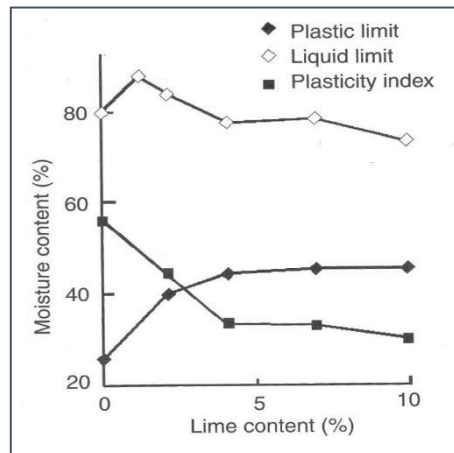


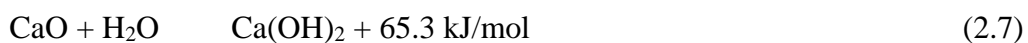
Figure 2.24. Effect of addition of lime on plasticity of London Clay

Source: O’Flaherty (2002)

There are basically 4 types of lime used in stabilization which are (Das, 2016; Mehta, et al., 2014; O’Flaherty, 2002):

- (i) Calcitic or high-calcium quicklime (CaO)
- (ii) Calcitic or high-calcium hydrated lime [Ca (OH)₂]
- (iii) Dolomitic monohydrate lime [Ca (OH)₂ + MgO]
- (iv) Dolomitic quicklime (CaO + MgO)

Both quick lime and hydrated lime release heat upon contact with water; however, quicklime has a highly exothermic hydration reaction thus giving off more heat. as shown in making it more dangerous in use (O’Flaherty, 2002). Mehta et al. (2014) states that hydrated lime is more often used because it is less caustic than quicklime. Equation 2.7 shows the reaction of quick lime with water to form hydrated lime (Thomas, 2001).



Since quicklime is dangerous and because it is not readily available in Kenya, hydrated lime was used instead of quicklime in this research.

The amount of lime required for stabilization varies between 2% - 10% of the soil. Ingles and Metcalf (1972) suggested that the following amounts of lime may be used as a rough guide.

- (i) 3-4% of lime is required for silty clays, heavy clays and very heavy clays.
- (ii) 3-8% of lime is required for heavy clays and very heavy clays
- (iii) 1% of lime (by weight of dry soil) may be allowed for each 10% of clay in the soil

There are two forms of improvement using lime, namely, modification and stabilization (Dang et al., 2016a; Dang, Fatahi, & Khabaz, 2016b; Beetham, et al., 2015; Chittoori, 2008). If quicklime is used in stabilization, the first process is principally drying of the soil which involves the reaction of quicklime with water to form hydrated lime, thus reducing excess water as shown in Equation 2.7. A lot of heat is generated, causing evaporation further reducing the moisture content (Greaves, 1996) after which modification and stabilization ensue.

Once the hydrated lime is introduced into the soil directly or from reaction of quicklime with water, it dissociates to provide calcium ions Ca^{2+} and hydroxyl groups, which then react with the clay-soil system (Bergado, 1996). The hydroxyl groups raise the pH to about 12.45 (Beetham, et al., 2015). The most substantial improvement in modification occurs in clays of moderate to high plasticity, although it can be done for almost all fine-grained soils. Modification occurs in a cation exchange process where the divalent calcium cations supplied by the lime replace the monovalent cations adsorbed on the clay mineral surface causing the diffuse double layer to shrink (Bohn, 2002) which in turn causes the electro-static charges on adjacent clay particles to interact to a greater extent. The cation exchange process continues until all charges on the interlayer and edges are satisfied (see Figures 2.5 and 2.6). The hydrated lime also reacts with the clay mineral surface in the high pH environment brought about by the lime-water system, changing the texture of the clay soils. Lime modification requires between 1-3% lime as $\text{Ca}(\text{OH})_2$ and is a rapid process which forms cementitious products resulting in a progressive flocculation-

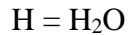
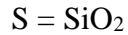
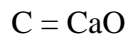
agglomeration process forming larger particles which translates into improved engineering properties which include reduction in plasticity, moisture holding capacity, swell, improved workability and stability. Modification is dependent on the amount of clay, the clay's activity and the cation that had been adsorbed on the clay. Clays with high cation exchange capacity such as sodium-dominated montmorillonitic clays are affected more by lime than say kaolinitic clays which have a low cation exchange capacity (Dang et al., 2016; Das, 2016; Obuzor et al., 2012; O'Flaherty, 2002; Bell, 1993).

After modification, stabilization takes place which is a time-dependent pozzolanic and hydration reaction with further additions of lime, with the best results expected from long cured specimens. Clay minerals are natural pozzolans and have the ability to react with lime to produce cementitious products (Sherwood, 1993). The calcium ions of the stabilizer and the silica and alumina of the clay minerals react in a high pH environment (pH >9) to form cementitious products such as calcium silicate hydrates (CSH), calcium aluminate hydrates (CAH), and calcium aluminium silicate hydrates (CASH), with the principal cementitious product being CSH. Formation of these hydrates is shown in Equations 2.8-2.10. These pozzolanic products contribute to the flocculation process by bonding adjacent flocculated soil particles together into larger-sized aggregates, and as curing occurs, they strengthen the soil. The pozzolanic reaction starts within hours and may continue for many years. The result is enhanced strength, a significantly impermeable barrier due to reduction of pore spaces, volume stability and increased density; thus a durable and strong material is produced, giving long-term service to structures built on the stabilized material (Dang et al., 2016; Das, 2016; O'Flaherty, 2002; Prusinki & Bhattacharja, 1999; Bell, 1993; Sherwood, 1993, Arabi & Wild, 1989).

Formation of CSH, CAH, and CASH:



Where



The following affects lime treatment of soils (Beetham, et al., 2015; Thomas, 2001):

(i). Type and percentage of clay minerals.

As noted above modification is affected by the amount of clay, the clay's activity and the cation that had been adsorbed on the clay affect the stabilization process, with clays with high cation exchange being affected most, although all clays are attacked by lime. Those possessing the highest available silica normally react more strongly. The mineralogy and cation exchange were therefore established for the neat, heated and stabilized soil in this research.

(ii). Organic matter.

Organic matter adsorbs calcium ions in lime affecting the strength development of lime treated soils. As noted above, calcium ions are required for formation of calcium silicate hydrates (CSH), calcium aluminate hydrates (CAH), and calcium aluminium silicate hydrates (CASH). Organic compounds in soil also react with lime reducing the pH level below that which is required for formation of the cementitious products. Tests for the organic content were therefore done in this research for the neat, heated and stabilized soil.

(iii). Compaction

A highly permeable stabilized soil will cause ingress of water. In water of neutral pH and low levels of Ca^{2+} , the calcium silicate hydrates (CSH) gel de-constitutes into $\text{Ca}(\text{OH})_2$ and silicate (Taylor, 1990). This raises the pH and Ca^{2+} of the water. With the supply of more water the CSH gel will continue to de-constitute. For a low permeable material, initial Ca^{2+} leaching is lower, and stops after a short time. Thus, a highly permeable material can potentially lose strength through softening and removal of the pozzolanic products (McAllister & Petry, 1992; Runigo, Cuisinier,

Cui, Ferber, & Deneele, 2009). Compaction should therefore achieve a minimum air void value throughout the fill (Sherwood, 1992).

(iv).Sulphates

In the presence of high levels of sulphates lime stabilization of clays results in expansion due to formation of ettringite [$\text{Ca}_6\text{Al}_2(\text{OH})_{12}(\text{SO}_4)_3 \cdot 26\text{H}_2\text{O}$] or thaumasite [$\text{CaSiO}_3 \cdot \text{CaCO}_3 \cdot \text{CaSO}_4 \cdot 14.5\text{H}_2\text{O}$], which are expansive and strength depleting. They have the capacity of causing distress to infrastructures built on them resulting in high cost implications or failure (Mitchell & Soga, 2005; Rajasekaran and Rao, 2005; Bell, 2007; Obuzor et al., 2012; Beetham et al., 2015; Kavak & Bilgen, 2016).

Ettringite and Thaumasite Formation

Ettringite formation requires a pH greater than 10 and adequate supply of Ca^{2+} , SO_4^{2-} , Al_2O_3 and H_2O (Little, Herbert & Kunagali, 2005) while thaumasite formation requires a very cold environment (4-15°C) and humid conditions (Collett, Crammond, Swamy & Sharp, 2004; Winter, 2009). It can also generate from alteration of ettringite at temperatures lower than 15°C (Little et al., 2005; Winter, 2009, Thomas, 2001). The following reactants combine to form thaumasite: Ca^{2+} , Si^{4+} , SO_4^{2-} , CO_3^{-2} and H_2O (Little et al., 2005).

To prevent ettringite formation, one or more of the reactants needs to be limited or prevented from accessing the stabilized soil, ettringite growth can be allowed before final compaction to form wide distribution of small ettringite crystals which can be accommodated in the spaces between the clay particle thus avoiding expansion hotspots (Little et al., 2005), or amending the chemical balance in the system to precipitate a non-deleterious mineralogy (Beetham et al., 2015).

Presence of sulphates is the main cause for formation of ettringite and thaumasite. To prevent heave due to sulphates, sulphate rich soils need to be avoided, and where ground water may form a source for sulphate transport. Beetham et al. (2015) suggested a maximum sulphates level of 0.25% but may be increased to 1% if soaked CBR tests do not indicate a swell problem (Highways Agency, 2009), although the suitability of soaked CBR tests to identify sulphate swell test has been questioned (Highways Consultancy Group, 2008). Further low air voids in

compacted soil of less than 5% greatly limit the potential of water ingress (Perry, MacNeil & Wilson, 1996) thus limiting leaching of calcium ions from any preformed CSH gel (McAllister & Petry, 1992). High porosity would also give ingress of sulphates from external locations (Beetham et al., 2015). For this reason, the level of sulphates was tested in the neat, the heated and the treated soil in this research.

Lime has been used as the main stabilizer for expansive for a long time. However, it has limitations where water logging is likely to be encountered (Obuzor et al., 2011; Obuzor et al., 2012; MOTC, 1987). Karatai, et. al. (2016) established that though the amount of lime could be greatly reduced when rice husk ash was added in stabilization of black cotton soil, the strength of the stabilized material reduced when soaked in water for 7 days. They concluded that the lime-rice husk ash stabilized black cotton soil can only be used in areas where excessive moisture is not likely encountered. Therefore in this research, various combinations of hydrated lime, heated soil and molasses were used to stabilize expansive soil to investigate their effectiveness in various simulated flooding conditions.

(i).Stabilizing using pozzolanic materials

A pozzolan is siliceous or an alumina-siliceous material which possesses no cementitious value but in finely divided form and in the presence of moisture, chemically reacts with lime at ordinary temperatures to form compounds possessing cementitious properties (Sridharan & Prakash, 2007) These materials are divided into natural and artificial pozzolans. Natural pozzolans include clayey or siliceous materials of volcanic origin such as tuff and trass. Others are calcined shale, calcined clay and metakaolin. Artificial pozzolans include fly ash, ground granulated blast-furnace slag and silica fume (O'Flaherty, 1974). Others are ashes such as fly ash, bagasse ash and rice husk ash. The latter have been used combined with lime.

Natural pozzolans are produced as a result of volcanic activity when silica rich magma meets with large quantities of underground water in volcanic conduits. Under high temperature and pressure the steam reacts with the dissolved carbon dioxide and sulphur gases and during the volcanic eruption the magma produces air borne cinder-like material with excellent pozzolanic characteristics (O'Flaherty, 1974).

Artificial pozzolanic materials include fly ash, ground granulated blast furnace slag (GGBS) and silica fume. Fly ash has mainly been used when activated with lime. GGBS and silica fume are discussed below, including what is lacking in their use to date.

(j).Ground granulated blast furnace slag

GGBS is made from blast furnace slag; the latter is a nonmetallic by-product of the manufacturing process of iron. Depending on the method of cooling the molten slag, various slag products are obtained, including GGBS. In fine form it has cementitious properties (Cokea, Yazici, & Ozaydin, 2009). Pathak, Pandey, Murari and Singh (2014) used GGBS to stabilize a clay soil and found that the CBR and compressive strength increased with increased GGBS, the percentage cohesion decreased, the angle of internal friction increased considerably while the plasticity improved. The overall effect of this was an improvement of the clay.

Yadu and Tripathi (2013) investigated the effect of stabilizing a soft clay with GGBS, and found that there was improvement of the physical and strength properties. The maximum dry density increased while the optimum moisture content decreased with the addition of GGBS. The swelling behaviour of the soil reduced significantly. They established that based on strength properties 9% of GGBS is optimal, which increased the soaked CBR and unconfined compression strength by 400% and 28% respectively, and reduced the free swelling index and swelling pressure by about 67% and 21% respectively. Cokea et al. (2009) established that using GGBS to stabilize expansive clay decreased its plasticity index, the clay fraction, and the swell, while both the silt fraction and the specific gravity increased, resulting in an improved soil. None of these methods studied the effect of long term curing or prolonged flood on the stabilized soil, which were investigated in this study.

(k).Silica fume

Silica fume, also referred to as micro silica or condensed silica fume, is a by-product of the reduction of high-purity quartz with coal in an electric arc furnace in the manufacture of silicon or ferrosilicon alloy. In this process, the silica fumes rise as an oxidized vapour from the 2000°C furnaces which condense upon cooling, after

which it is processed to remove impurities and to control particle size. The fume is essentially silicon dioxide (usually more than 90%) in non-crystalline (amorphous) form, which is pozzolanic (Negi, Yadai, & Singai, 2013a; O'Flaherty, 1974).

Gupta and Sharma (2014) studied the effect of micro-silica fume on the characteristic of expansive clays. They found that the soaked CBR increased from 2.69% to 5.87% with 10% silica fume. They concluded that expansive clay stabilized with micro silica fume can be used as subgrade material for flexible pavements in low traffic volume rural roads. Negi, Yadai and Singai (2013a) in studying the effect of silica fume on index properties of black cotton soil found that the index and the swelling properties of the black cotton soil improved. Negi, Yadai and Singai (2013b) found that stabilizing black cotton with micro silica fume increased the OMC and reduced the MDD with increase in silica fume content, and that the CBR increased by 72% with 20% silica fume. Further, the addition of silica fume increased the unconfined compression strength by 31%, that there was a significant decrease in the swelling characteristics, while the degree of expansiveness reduced from “high” to “low”. They concluded that silica fume has the potential to change the engineering characteristics of expansive clays like black cotton soil.

None of these studies considered the effect of long term curing or flooding on the stabilized soil. This research thus investigated the effect of long term curing long term flooding on stabilized soil.

2.12.4.6 Enhancement of stabilizers by alkali activation.

Lime and cement have been successfully used to activate stabilizers so as to enhance their cementitious and pozzolanic properties (Sharma & Sivapullaiah, 2016). The process which is called alkali activation is gaining attention because it converts waste by-products into beneficial materials (Mehta & Siddique, 2016) and requires lower temperatures in production and emits less CO₂ than in the production of cement (Provis & Deventer, 2014). Alkali-activated materials are therefore seen as alternative and supplementary binders to cement depending on local availability of raw materials (Provis, 2017), and effective as stabilizers (Sharma & Sivapullaiah, 2016). The materials are also called geopolymers although strictly alkali-activated materials have low silica and alumina contents with a high calcium content, while

geopolymers have a low calcium content. Alkali-activated materials comprise either two-part or one-part geopolymers formed in a chemical reaction. The reactions for the former involve a concentrated aqueous solution of alkali hydroxide, silicate carbonate or sulphate and a solid aluminosilicate precursor, in addition to water. The one-part geopolymers are prepared by mixing a solid alkali-activator with a solid aluminosilicate precursor with or without need for calcination in addition to water. They are easier to handle as compared to the two-part geopolymers due to difficulties of handling viscous, corrosive and hazardous alkali activator solutions and therefore more research effort is being directed towards them (Luukkonen et al., 2018).

Class F low calcium content fly ash (which is pozzolanic) made from coal combustion alone or combined with blast furnace slag is the most common aluminosilicate precursor in one-part geopolymers (Luukkonen et al., 2018; Hajimohammadi & Deventer, 2017; Nematollah, Sanjayan, Qiu & Yang, 2017). GGBS is used as a calcium-rich aluminosilicate precursor alkali-activated material. It is frequently mixed with class F fly ash or can be used alone. Other one-part geopolymers are fayalite slag formed in secondary smelting of copper or nickel, calcined clays (kaolin dehydroxylated into metakaolin, bentonite, albite, potash feldspar, pyrophyllite and natryal zeolite), rice hull and husk ashes, geothermal silica, silica fume and red mud. The latter is a by-product from production of alumina from bauxite by the Bayer process (Luukkonen et al., 2018; Feng, Provis & Deventer, 2012; Deventer, Feng & Duxson, 2010).

The source of the activators (the alkali) in a one-part geopolymer mix is any material that provides alkali cations, raises the pH of the reaction mixture, and facilitates dissolution (Provis & Deventer, 2014). These include NaOH, Na_2SiO_3 , Na_2SiO_3 , Na_2CO_3 , NaAlO_2 , CaSO_4 , CaSO_4 , Na_2SO_4 , KOH, red mud, maize stalk and cob ash. Others are CaO, MgO, $\text{CaMg}(\text{CO}_3)_2$ and $\text{Ca}(\text{OH})_2$ which supply alkaline earth cations to systems with low calcium content (Luukkonen et al., 2018; Li, Sun & Li, 2010). Low calcium precursors require calcium rich activators such as lime.

In soil stabilization the more frequently used aluminosilicate precursors are ground blast furnace slag, bagasse ash, fly ash and rice husk ash with lime as the main

activator. Heated black cotton soil has not been used to-date as a precursor. In this study, heated black cotton and molasses were used with hydrated lime as the activator. The main precursors are discussed in following pages.

(a).Lime and ground granulated blast furnace slag (GGBS)

According to Wild, Kinuthia, Jones & Higgins (1998) who used lime and GGBS to improve the UCS of a sulphate bearing clay, the lime in a clay-lime mix provides the required alkaline environment for the slag activation and hydration, at the same time producing modification of the clay. It is after this that the time dependent pozzolanic and hydration reactions occur to form the cementitious products, as discussed in section 2.7.4.7 9(g), resulting in a strong non-swelling material. The GGBS they used had CaO (41.99%), SiO₂ (35.34%) and Al₂O₃ (11.59%) as the main oxides, while Fe₂O₃ was negligible. It was thus a non-pozzolonic calcium-rich aluminosilicate precursor. However they did not investigate long term curing and flooding scenarios.

Obuzor et al. (2011) studied the utilization of lime activated GGBS to reduce the deleterious effect of flooding on stabilized road structural materials. The study involved stabilization of a soil called Lower Oxford Clay, which was known to experience expansive behaviour when stabilized with lime. It involved varying levels of lime and GGBS for a total binder content of 16%. Some of the results they came up with are:

- That the presence of GGBS increased the formation of Calcium-Silicate-Hydrate gel matrix which forms the cementing medium in soil stabilization
- That at longer curing periods the GGBS was able to enhance the densification of the system to reduce water ingress, allowing choice between a well-drained material and that which is highly impermeable in embankment construction since behavior of embankments is highly influenced by drainage characteristics of the fill material.

Obuzor et al., (2012) studied stabilization of low bearing capacity soils with lime-activated GGBS as a mitigation measure to floods on road embankments constructed on flood plains. The total binder content was 16%. They came up with the following results:

- That lime-activated blast furnace slag is a better binder for clay than lime.
- That GGBS only required small amounts of lime for activation to stabilize clay; the strength of the stabilized material increased with a reduction in lime content.
- Although the durability of lime-slag bound clay decreased with increasing age under extended period of soaking in water for ten days, it was durable enough for application in road embankments in flood-prone areas.

Kavak and Bilgen (2016) studied the stabilization a clay from Ankara (Turkey) with a low CBR value of 4% with a relatively high plasticity index of 35% using GGBS and lime and concluded that the mixes had advantages over lime stabilization in that very a very high CBR of 184 was obtained while the UCS increased by 500%; the mixture also prevented formation of ettringite which occurs in lime stabilized sulphate rich soils, and making the GGBS-lime stabilized soil have lower swelling effects lime-stabilization; further, GGBS which is a by-product is turned into a valuable road construction material. The samples were cured for periods of up-to one year, and the strength continued to increase The total binder content was 8.8%. Effects of flooding were not investigated.

Wild et al. (1998), Obuzor et al. (2011), Obuzor et al. (2012) and Kavak and Bilgen (2016) established that lime-activated GGBS can be used to stabilize clays. GGBS is not available in Kenya. Wild et al. (1998) and Kavak and Bilgen (2016) did not consider prolonged flooding. This research thus investigated the effect of activating heated soil and molasses with lime for normal use, normal and prolonged flooding.

(b).Lime and bagasse ash

Dang et al. (2016) investigated the effect on the engineering properties of expansive clays stabilized with bagasse ash and hydrated lime. Bagasse ash is a pozzolanic aluminosilicate precursor low in calcium which needed lime as an activator. They found that increasing lime-bagasse ash mixture from 0 to 25% caused a significant decrease in linear shrinkage more significantly than that of bagasse ash and mixed with expansive clay. Stabilized specimens were cured for 7 and 28 days, and CBR

tests taken for unsoaked and after 7 days soak. The unsoaked specimens reported higher strength than the soaked ones. In both the soaked and unsoaked specimens, the CBR improved significantly as the lime-bagasse ash mixture was increased from 0 to 25%, as to satisfy requirements for subgrade and sub-base for road construction. They concluded that the use of bagasse ash which is a waste by product will not only contribute to environmental benefits, but also to sustainable development and high construction cost savings due to reduction of amounts of conventional stabilizers such as lime and cement. But they did not investigate the effect of long term curing or flooding on the stabilized soil.

Wubshet and Tadesse (2014) stabilized expansive soil using lime, bagasse ash and a combination of the two and found that there was an improvement in the CBR with a minor improvement for bagasse ash only, while the greatest improvement was with lime mixed with bagasse ash. The total binder content for lime and bagasse was 18%.

In the lime-activated bagasse ash treatment of expansive clay by Dang et al. (2016) and Wubshet and Tadesse (2014) effects of long term curing and flooding were not considered, which were investigated in this research.

(c) Lime and Fly Ash

Fly ash is a by-product of the combustion of pulverized coal in electric power generating plants. Upon ignition in the furnace, most of the volatile matter and carbon in the coal are burned off. During combustion, the coal's mineral impurities (such as clay, feldspar, quartz, and shale) fuse in suspension and are carried away from the combustion chamber by the exhaust gases. In the process, the fused material cools and solidifies into spherical glassy particles called fly ash (Sridharan & Prakash, 2007). Class F low calcium content fly ash which is pozzolanic has been activated with lime and used in stabilizing black cotton soil. A number of cases are cited below:

Satyanarayayana et al. (2016) studied the effect of the engineering properties of expansive soil stabilized with fly ash and lime mixes and established that stabilizing expansive soil with mixes of lime and fly ash enabled the soil to achieve high strength values making it useful as a good bearing material. The total binder content

was 50%. They measured strength in terms of UCS on samples cured for 7 and 28 days. They did not consider long term curing or flooding.

Ravichandran, Krishnan, Javani, Annadurai and Gunturi (2015) stabilized expansive clays with a mixture of phosphogypsum and fly ash. Phosphogypsum which has a Calcium Oxide (CaO) percentage of 32.27%, is a by-product in the manufacture of ammonium phosphate fertilizer from rock phosphate using sulphuric acid. CBR results were based on samples cured for 3 and 7 days and soaked for 4 days. The total binder content was 11%. The result showed an increase in CBR and a substantial reduction in the free swell index. They concluded that this mixture performs satisfactorily as a cheap stabilizing agent for clayey soils. No tests were done for long term curing or flooding.

Parmar, Khorasiya and Shah (2013) stabilized black cotton soil with lime and fly ash and established that that OMC, MDD, free swell index and soil salinity were improved significantly. The binder content was 40%. No tests were done for strength, long term curing or flooding.

In stabilizing expansive clay with lime-activated fly ash, Satyanarayayana et al. (2016), Ravichandran et al. (2013 and Parmar et al. (2013) did no tests for the impact of long term curing or flooding; further Parmar (2013) did not carry out any tests for strength, long term curing or flooding. In this study, the effect of strength on long term curing and flooding were investigated.

(d).Lime and Rice Husk Ash

Karatai et al. (2016) stabilized an expansive clay soil using mixes of lime and rice husk ash (RHA) and established that using 20% of RHA and approximately 2% lime increased the CBR by 800%, reduced plasticity by approximately 90% and decreased free swell by approximately 70%. They thus concluded that a RHA lime mixture can be effectively used as a stabilizer for expansive soils. Though increasing the lime content increased the strength, they limited it to the minimum required to achieve CBR value of 8% which is adequate for use of the stabilized soil as a subgrade material for road construction, in order to limit costs with incorporation of an affordable green technology in construction. However, they also found that the

strength decreased when the stabilized material was exposed to excess moisture and recommended the material be used for arid and semi-arid areas.

Shrivastava, Singhai and Yadav (2014) studied the effect of lime and rice husk ash on the engineering properties of black cotton soil and found that the OMC increased by 20%, the CBR increased by 387%, while the unconfined compressive strength increased by 30%. The total binder content was 25%. There were no test done for swell, plasticity or long term curing or flooding.

Kumar and Preethi (2014) investigated the behaviour of a clayey soil stabilized with rice husk ash and lime and found that 6% lime and 10% rice husk ash improved the CBR by 90.65% after 14 cure. There were no test done for swell, plasticity, or effect of long term curing or flooding.

Though there was improvement of strength of the expansive clay stabilized with lime-activated rice husk ash in the work by Karatai et al. (2016), Shrivastava, et al. (2014) and Kumar and Preethi (2014). No tests were done for long term curing and flooding, although Karatai et al. (2016) demonstrated that the strength could not be sustained in conditions of excess moisture. This research investigated the effect of long term flooding with soil stabilized with heated soil and molasses.

2.12.5 Treatment of Expansive Soils for Structures.

As stated in section 2.12(v), the methods available for improving soils for other structures are:

- (a) Constructing structures to withstand the expansive soil by:
 - Strengthening structures to withstand heave.
 - Constructing structures flexible enough to withstand the differential soil changes
 - Putting up structures on isolated deep foundations below the active zone
- (b) Moisture Control Barriers
- (c) Grouting/Injection.

These methods are discussed hereunder. Although they are principally for heavy structures some of them are applicable to light structures.

2.12.5.1 Constructing structures to withstand the expansive soil.

Recommended methods of constructing structures to withstand heave are shown in Table 2.9 (Gromko, 1974). These recommendations depend on the geometry of the structure, the total predicted heave and the type of structure. In particular for a light structure such as a ground floor slab, the building should be rigid and the slab designed to resist bending and should be independent of grade beams where the predicted heave is between 6.35 and 12.7 mm or 12.7 and 50.8 for L/H is 1.25 or 2.5 respectively, where L/H is the ratio between the length and the height of the wall panels supporting the slab. No precaution is required for lower predicted heaves. Where the predicted heave is between 12.7 and 50.8 mm or 50.8 and 101.6 for L/H is 1.25 or 2.5 respectively, the ground floor slab should have clear flexible joints, contact between structural units avoided and water proof material be inserted in the joints. For higher heaves the ground floor slab should be suspended. In this research a model building was constructed to determine the heave on the slab constructed on stabilized soil fill, from which recommendations on construction procedures were made.

Table 2.9: Construction Procedures for Expansive Clay Soils

Total predicted heave (mm)					
L/H=1.25	L/H=2.5	Recommended construction		Method	Remarks
0 to 6.35	12.7	No precaution			
6.35 to 12.7	12.7 to 50.8	Rigid building movement reinforcement necessary)	tolerating (steel As	<i>Foundations:</i> Pads Strip footings mat (waffle) Floor slabs Waffle tile <i>Walls:</i>	Footings should be small and deep, consistent with soil-bearing capacity. Mats should resist bending Slabs should be designed to resist bending, and should be independent of grade beams Walls on a mat should be as flexible as the mat. There should be no vertical rigid connections. Brickwork should be strengthened with tie bars or bands.
12.7 to 50.8	50.8 to 101.6	Building Movement	damping	<i>Joints:</i> Clear Flexible <i>Walls :</i> Flexible Unit construction Steel frame <i>Foundations</i> Three point Cellular jacks	Contacts between structural units should be avoided, or flexible, waterproof material may be inserted in the joints. Walls or rectangular building units should heave as a unit. Cellular foundations allow slight soil expansion to reduce swelling pressure. Adjustable jacks can be inconvenient to owners. Three-point loading allows motion without duress.
>50.8	>101.6	Building independent of movement		<i>Foundation drilled shaft:</i> Straight shaft Bell Bottom <i>Suspended floor:</i>	Smallest-diameter and widely spaced shafts compatible with the load should be placed. Clearance should be allowed under grade beams. Floor should be suspended on grade beams 305 to 460 mm above the soil.

Source: Gromko (1974)

2.12.5.2 Installation of moisture control barriers.

This is done by installing vertical moisture barriers about 1.5 m deep around the perimeter of the slabs for slabs-on-grade type of construction. The moisture barriers which may be made of trenches filled with gravel, lean concrete or impervious membranes control the moisture variation in the soil thus reducing the long-term effect of differential heave (Das, 2016). The main challenge with this method is that construction of the substructure must be fully completed within the dry season, which cannot be guaranteed, and thus the need to research into other methods

2.12.5.3 Stabilization by grouting/ injection

In this method of stabilization, stabilizers are introduced by injection into the soil, where they harden. As the grouting is always done under pressure, the stabilizers with high viscosity are only suitable for soils with high permeability. The method is suitable for stabilizing buried zones of relatively limited extent, such as pervious stratum below a dam. The method is used to improve the soil that cannot be disturbed. An area close to an existing building can be stabilized by this method. However the grouting method is expensive when compared with direct blending methods (Terzaghi et al., 1996).

Grouting techniques can be classified as explained hereunder

(a).Slurry Grouting

This method of stabilization is used to stabilize expansive clays where stabilizers are introduced into the expansive soil by pressure injection. Lime-slurry or lime-fly-ash slurry is injected to depths of 4-5m or sometimes even deeper to cover the active zone. Single or multiple injections can be done depending on the soil conditions on site, as shown in Figure 2.25 (Das, 2016). However this study considered the ground floor slabs only not foundations.

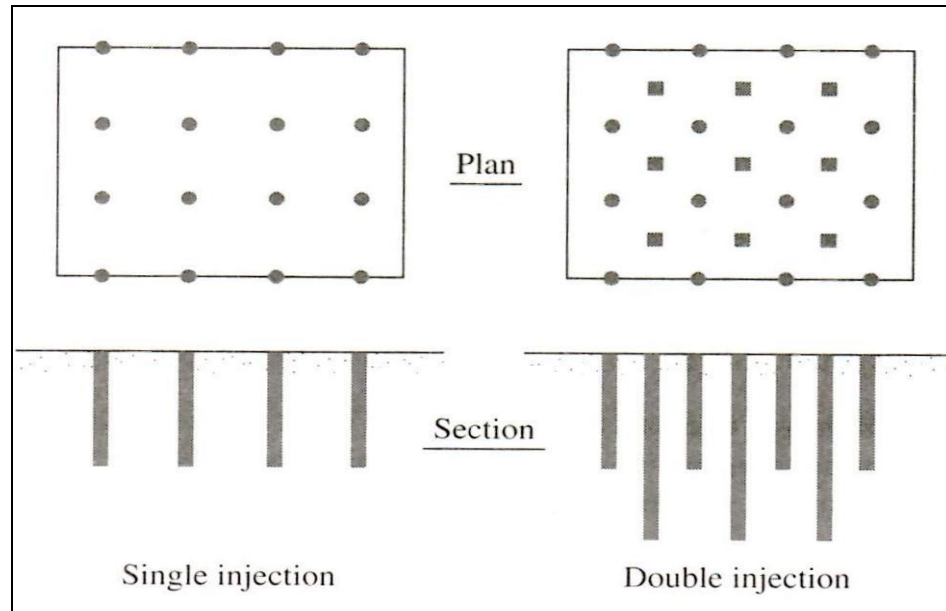


Figure 2.25: Multiple Lime Slurry Injection Planning for a Building Pad.

(Source: Das, 2016)

(b).Other grouting methods

The other grouting methods are cement, clay, chemical, chrome-lignin polymer and bituminous grouting. They are mainly used to stabilize sands and silts not expansive clays (Terzaghi et al., 1996; O’Flaherty, 1974).

2.13 Economic Considerations.

Economic evaluations are concerned with the difference in economic results from alternative courses of action. The evaluation must take into account the timing of expenses and cash inflows of the different projects to be evaluated, which occur at different times, and have to be converted into a form that can be compared. The

evaluation may be considerations for the cost of the projects or their profitability (Harris & McCaffer, 2006).

For cost comparisons, the intent is to choose the least expensive project. The initial capital cost and the running costs including maintenance and any residue value must be considered. There are two main evaluation methods; the present worth, which is the most popular, and the equivalent annual cost method. In the present worth method, all the costs are discounted to the present value at a chosen interest rate for the useful life of the project. The project with the smaller present worth is taken as the more economic. In the equivalent annual costs method, the initial capital or investment cost is converted into an equivalent annual cost over the useful life of the project, at a chosen interest rate, and this added to the already established yearly running cost. Where there is a residue value, it is first converted into a present value and used to reduce the initial investment before converting it into an equivalent annual cost. The project with the lower equivalent annual cost is chosen as the more economic (Harris & McCaffer, 2006).

Profitability comparisons apply where investment in a project leads to a return, and the intent is to determine whether the return on the investment is adequate; where there are alternatives, comparison is done and the alternative with the better return chosen. Various metrics are used which include the payback period, the net present value, and the internal rate of return. The latter method is also called the discounted cash flow yield (Harris & McCaffer, 2006). The payback period is the time taken by a project to recover its initial cost. The project with the least payback period is chosen as the better alternative. The net present value is the net value of cash inflows and outflows, all discounted to the present value at a chosen interest rate. A project with a positive net present value is considered profitable while the project with the highest net present value is selected when comparing alternatives. The internal rate of return is the discount rate at which the net present value of a project is zero. Where the internal rate of return exceeds the minimum desired rate of return, then the project is accepted. When comparing

alternatives, the project with the highest internal rate of return is chosen (Harris & McCaffer, 2006).

Where considerations are made for costs and cash inflows from the start of the project to the end of its useful life, then the evaluation is called life cycle costing. It considers alternative designs, taking into account initial investment costs, future running costs including maintenance, the income and disposal. Full appraisal is done at the design stage since the client and the design team have total control over the design before any commitments have been made. The metrics used in life cycle costing are the net present value and the internal rate of return; the project with the highest positive net present value or the highest internal rate of return is selected as the more economical (Harris & McCaffer, 2006; Smith, 2002).

In this research, the economic comparison was done for the cost of the fill using conventional methods, and the fill using the product of the research. It is expected that once the fill has been done, the running costs, monetary benefits in service and residue values for the road project or the building will be the same. The main difference in cost is the initial capital cost. Thus, discounting methods were not applied since the costs were based on present day costs.

2.14 Summary of the Review and Research Gap

The foregoing sections detail literature review on formation of the various types of clay minerals and how they influence expansive behaviour of soils and how these soils can be classified. The methods of assessing expansive behaviour and treating expansive clays are highlighted, including how costs of the stabilised soils can be assessed.

The three main groups of clay minerals are the kaolinite, the illite and the smectite, with the montmorillonite being the most common of the latter. The kaolinites are the most stable followed by the illites, with the smectite being the most expansive, especially the montmorillonites which have extremely high water absorption, swelling and shrinkage

characteristics. They have very high cation exchange capacities and the basal spacing ranges from 9.6 Å to complete separation (Knappett & Craig, 2012; Das, 2008; Mitchell & Soga, 2005; Terzaghi, et al., 1996; Smith & Smith, 1988). However, most soils consist of soils of more than one type of clay mineral, and the properties influenced by the constituent minerals (Mitchell & Soga, 2005; Terzaghi, et al., 1996).

Soil classification places soils within a limited number of behavioural soil groups on the basis of the grading and plasticity characteristics which provide useful guides as to how a disturbed soil will behave under various conditions of moisture content. In this research the BSCS was adopted (Knappett & Craig, 2012; O'Flaherty, 2002; BS 1377 Part 2, 1990; Smith & Smith, 1988).

The properties of expansive clays highlighted are the mineralogy and the engineering properties. The mineralogy helps establish the clay mineral present in a soil (Knappett & Craig, 2012; West, 1995; Knappett & Craig, 2012; Murthy, 2007; Sridharan & Prakash, 2007; Mitchell & Soga, 2005; Terzaghi, et al., 1996). The engineering properties which include the index properties of atterberg limits and the linear shrinkage properties are used to evaluate the swelling potential of the soil; they give an indication of the potential damage expansive soils can cause to structures and pavements built on them. The expansive characteristics of the clays cause damages which translate into billions of dollars worldwide (Tripathy et al., 2002; Rao & Thyagara, 2007; Dang et al., 2016; Chen, 1988; Bhavsar & Patel, 2014; Nelson & Miller, 1992; Das, 2016; Bell, 2007; Murthy, 2007; Mitchell & Soga, 2005; Sridharan & Prakash, 2000). Therefore there is a great need for an efficient method of treating expansive soils using locally available materials which would translate in cost savings.

The various methods of treating expansive clays to be used as fill material are discussed in section 2.12. For road construction the main methods are avoiding the expansive clays by re-alignment, replacing with suitable material, designing for low strength and allowing for regular maintenance and stabilization. Re-alignment is only feasible where the

expansive clay is of limited extent and in non-built areas (Jones & Jefferson, 2012; MOTC, 1987), whereas replacement with suitable materials is only economic for reasonable thicknesses and for borrow and spoil areas close to the site. It depletes arable land and has high cost implications due to burning of fossil fuels and emission of greenhouse gases (Prusinski & Bhattacharja, 1999; Yang & Zheng, 2006; Obuzor, 2012). Designing for low strength and allowing for regular maintenance has challenges where the traffic growth exceeds anticipated and where allocation of adequate maintenance funds is a challenge (Jones & Jefferson, 2012).

The various soil stabilization methods are compaction, mechanical stabilization, thermal treatment, pre-wetting and stabilization through addition of chemicals/additives. Mechanical stabilization is not effective in regions of high rainfall, and often a stabilizing agent has to be added because it cannot always be achieved (Chittoori, 2008; Sherwood, 1993) It also involves haulage which is expensive as in the case of removal and replacement.

Thermal treatment involves heating or freezing. The current research work on stabilizing by heating gives ranges of heating temperatures and heating periods, without any specific temperature or heating period for the soil to be suitable as a fill material and has concentrated on swelling characteristics, without any studies on strength changes except for work by Li et al.(2014) who used low temperatures for a very long period of 6 hours, and tested both swelling characteristics and strength. The research by Wang et al. (2008) investigated effect of temperature on swelling characteristics of two commercial expansive clays. This research investigated the optimum temperature at which a naturally occurring black cotton soil achieves adequate swelling and strength properties for it to be used as a fill material for a short heating period of 2 hours, based on work by Rashad (2013) who suggested that 2-5 hours may be adequate heating periods from a review of current heating periods in production of metakolin. Freezing is very expensive and cannot be sustained in hot climates; it was therefore not considered in this study.

Pre-wetting is a slow method, hydrated lime may have to be added to the top layer to make it less plastic and more workable; it is not practical because the swelling characteristics are restored once the soil dries up.

Of the chemicals/additives used for stabilization as discussed in section 2.12.4.5 calcium chloride causes a decrease in strength (Bell, 1993) and sodium chloride increases strength by a negligible amount (Davoudi & Kabir, 2011). Work by Ayibiou and Ola (2015), Mirzababaei et al. (2017) and Canakei et al. (2015) on treatment of expansive clays with sodium silicate, polymers and chrome lignin respectively established that they increased the strength and reduce the plasticity index. However, no tests were done for prolonged curing and flooding conditions. Cement is more suited for granular and clayey soils with low plasticity; expansive clays respond better to lime (Murthy, 2007; Das, 2016). Lime has been used as a stabilizing agent for expansive clays a long time; it improves the strength and the swelling characteristics. However it has been found to be ineffective in regions prone to high moisture and flooding (Obuzor, 2012, MOTC, 1987). A number of stabilizers from various industries have been developed for soil stabilization in recent years. GGBS is a calcium oxide rich cementitious which reduces the swelling properties and increases the strength of expansive clays (Pathak et al., 2014; Yadu & Tripali, 2013; Coker et al., 2009) However, it has not been tested for prolonged curing and flooding. Silica fume is a pozzolanic material which improves the swelling and strength characteristics of expansive clays (Gupta & Sharma, 2014; Negi, et al., 2013a; Negi, et al., 2013b). No tests have done for prolonged curing and flooding.

The current trend is to amend stabilizers with activators like lime and cement to enhance their cementitious and pozzolanic properties to make one-part geopolymers prepared by mixing a solid alkali-activator with a solid aluminosilicate precursor with or without need for calcination in addition to water (Luukkonen et al., 2018; Sharma & Sivapulliah, 2016). The precursors in use for stabilization of expansive clays as discussed in section 2.12.4.6 are calcium-rich GGBS and pozzolonic baggase ash, fly ash and rice husk ash. Obuzor (2012) and Obuzor (2011) established that lime-activated GGBS turned

expansive clays into strong, non-expansive materials, which could withstand long term curing and severe flooding. Kavak & Bilgen (2016) established that expansive clay stabilized with lime-activated GGBS could withstand long term curing but they did not test it for flooding. Lime activated baggase ash is effective as a stabilizer (Dang et al., 2016; Wubshet & Tadesse, 2014), but it has not been tested for long term curing and flooding conditions. In stabilizing expansive clay with lime-activated fly ash, Satyanarayana et al. (2016), Ravichandran et al. (2013 and Parmar et al. (2013) found that lime-activated fly ash was effective as a stabilizer, but no tests have been done to establish the impact of long term curing or flooding. Karatai et al. (2016), Shrivastava, et al. (2014) and Kumar & Preethi (2014), established that lime-activated rice husk ash was effective as a stabilizer. However Karatai et al. (2016) demonstrated that the strength could not be sustained in conditions of excess moisture. Thus the only work which has been done to assess effectiveness of one-part geopolymers for long term curing and flooding is on lime-activated GGBS. Use of one-part geopolymers emphasizes use of locally available materials (Provis, 2017). Since GGBS and fly ash are not available in Kenya, this study investigated suitability of lime-activated pozzolanic heated black cotton soil mixed with molasses as a stabilizer for long term curing and flooding.

None of the studies discussed considered the economy of the treatment method adopted. This research evaluated the economy of using the stabilized material developed in the research.

2.15 Conceptual Framework

Black cotton is characterized by low strength and high swelling characteristics. This study undertook to stabilize the soil with heat, and a blend of heated soil, lime and molasses to make it suitable as a fill material mainly for road construction, and also for ground floor slabs. Studies to date have not given specific heating periods and temperature for soils improved by heating. The effectiveness of lime-activated heated black cotton soil as a pozzolanic alumino-silicate precursor mixed with molasses was

investigated, and the stabilized material tested for strength, swelling properties and durability against long term curing and simulated flooding conditions. The conceptual framework is shown in Figure 2.26.

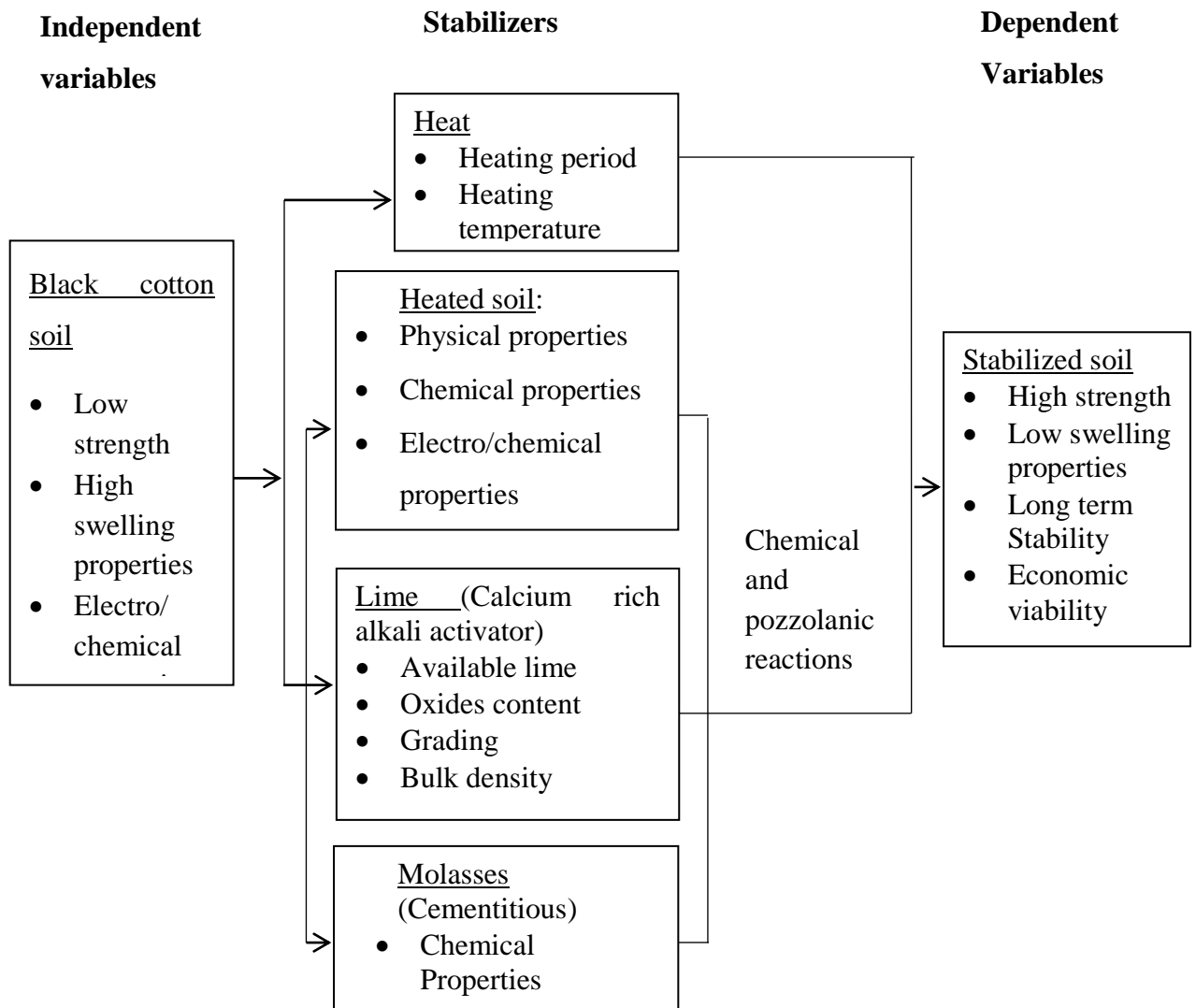


Figure 2.26: Conceptual Framework

CHAPTER THREE

MATERIALS AND METHODS

3.1 Introduction

This chapter describes the research methods used in the study. The methods were mainly laboratory tests as primary data. The cost of fill material was established from secondary data. Standard laboratory tests are described only briefly with references made to the appropriate standard. The laboratory tests were to determine properties of neat black cotton soil, heated black cotton soil and black cotton soil stabilized with various mixes of heated black cotton soil, lime and molasses. Costs of fill and stabilizing the soil were also established.

3.2 To determine the effect of heat on expansive clay soils

Laboratory tests were conducted on neat black cotton soil (NBC) and on the black cotton soil heated to various temperatures, to establish the effect of heat on black cotton soil. The materials, the experimental methods and data collection and analysis methods are discussed hereunder.

3.2.1 Materials

The expansive soil used in this research was black cotton soil obtained from Mwhoko area of Kiambu County, about 25 km North East of Nairobi City in Kenya. The material used was located 500 mm below the ground level to avoid vegetable matter. Since soils are fairly variable, it was necessary to establish that the tested soil is expansive, thus being a fair representative of such soils. Tests were conducted on air dried neat black cotton (NBC) soil to establish its properties, and tests on the neat soil heated at 200⁰C, 300⁰C, 400⁰C, 500⁰C, 600⁰C, 700⁰C, 800⁰C, 1000⁰C, and 1200⁰C to determine the effect of heat on the black cotton soil. Heating was done for 2 hours in a Elsklo s.r.o closed

electric kiln with a power rating of 108 kVA, to eliminate emission of greenhouse gases (Plate 3.1). The tests were done after the soil had cooled to room temperature, thus allowing for any possible reversal of properties.



Plate 3.1: Heating kiln with soil sample in position

3.2.2 Tests conducted

The following tests were conducted:

- Physical tests comprising particle size distribution, specific gravity, atterberg limits and free swell. They were conducted in accordance with BS1377 Part 2:1990 except free swell which was done according to the method proposed by Holtz and Gibbs (1950).
- Mechanical tests which were compaction characteristics, California Bearing Ratio (CBR) and Unconfined Compressive Strength (UCS).

- Chemical and Electro-chemical and tests comprising oxide analysis using the X-ray fluorescence (XRF), X-ray diffraction (XRD), cation exchange tests, organic matter content, loss on ignition, sulphates, the pH and chloride content.

3.2.3 Physical Tests

3.2.3.1 Particle Size Distribution

(a).Experimental Set Up

The particle size distribution was done in accordance with BS1377 Part 2:1990, by wet sieving and sedimentation by the hydrometer analysis. In accordance with this standard, combined sieving and sedimentation procedures enable a continuous particle size distribution curve to be plotted from the size of the coarsest particles down to the clay size.

Wet sieving which is applicable to cohesion-less soils, was done by passing an oven dried sample of soil through a nest of sieves in descending order. The hydrometer analysis was done on material passing the 63 microns sieve, to determine the the proportions of sand, silt and clay.

(b).Data Collection and Analysis Methods

The results of the wet sieving and the hydrometer analysis were combined and plotted on a semi-logarithmic scale of the percentage of material passing each sieve against the sieve sizes to give the grading properties from which the coefficient of uniformity, C_u and coefficient of curvature C_c were determined as shown in Equations 3.1 and 3.2 (Knappet & Craig, 2012; O’Flaherty, 2002):

$$C_u = \frac{D_{60}}{D_{10}} \quad (3.1)$$

$$C_c = \frac{D_{30}^2}{D_{10} \times D_{60}} \quad (3.2)$$

Where

D_{10} is the particle diameter at which 10% by mass is finer

D_{30} is the particle diameter at which 30% by mass is finer

D_{60} is the particle diameter at which 60% by mass is finer

3.2.3.2 Determination of Atterberg Limits and Free Swell.

(a).Experimental Set Up

The atterberg limits are the liquid limit, the plastic limit, the plasticity index and linear shrinkage. They were carried out in accordance with BS1377 (Part 2):1990. Free swell was done in accordance with the method proposed by Holtz & Gibbs (1956).

(i).Liquid Limit (W_L).

The liquid limit (W_L) test was carried out using the cone penetrometer method (Plate 3.2) since it is easier to carry out and gives more reproducible results than the earlier Casagrande test, according to BS1377 (Part 2):1990.

The test was carried out on a sample of soil passing the 425 μm sieve, which was mixed with distilled water and made into a homogeneous paste. The mixed soil was then placed into a metallic cylindrical plasticity index cup, which was then placed on the cone penetrometer apparatus as shown in Plate 3.2.

(ii).Plastic Limit (W_P)

The plastic limit test (W_P) was carried out from the paste from the liquid limit test. The material was plastic enough to be shaped into a ball. The ball was moulded and rolled until a 3 mm thread sheared longitudinally and transversely. Samples were then taken for moisture content determination.

(iii).Plasticity Index (Ip)

The plasticity index (Ip) is the difference between the liquid limit and the plastic limit (Das, 2008; BS1377 - Part 2: 1990; Smith & Smith, 1998).

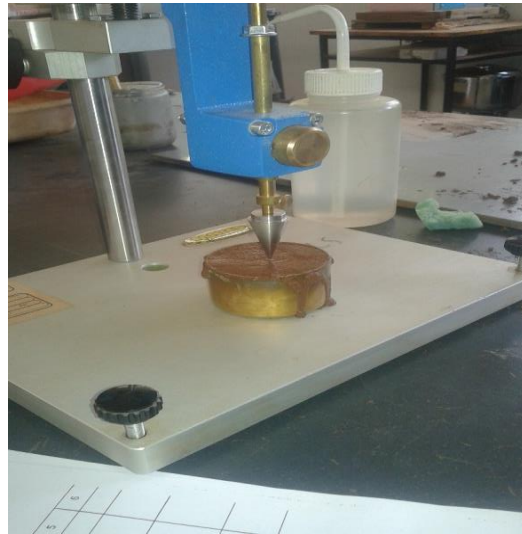


Plate 3.2: Cone penetrometer set-up with specimen in position

(iv).Linear Shrinkage

The linear shrinkage test was done on a sample passing a 425 micron sieve, having the moisture content of the liquid limit sample. The sample was placed in a brass linear shrinkage mould, air dried for 2 days until the soil shrunk from the walls of the mould, after which it was oven dried at 105°C. The length of the oven-dry specimen and the original length were measured.

(v) Free Swell

The free swell test was carried out on material passing the 425 micron sieve. 10 ml of the material oven dried at 105°C was slowly poured into a 100 ml graduated cylinder filled with distilled water (Plate 3.3). The soil was allowed to settle and the initial volume of

the soil taken. The mixture was allowed to stand for 24 hours, and the swelled volume measured from which the free swell was computed.



Plate 3.3: Free Swell Test

(b).Data Collection and Analysis Methods

(i).Liquid Limit.

Cone penetrations were made into the soil sample (Plate 3.2) for 5 seconds and the soil moisture content for each penetration were taken in the penetration range of 15 to 25 mm. The liquid limit was determined as the moisture content for a 20 mm penetration from a plot of the moisture content against the cone penetration (Figure 3.1)

(ii).Plastic Limit (WP)

From the moulded and rolled specimen described in 3.2.3.2(a) (ii) above, the portions of the crumbled thread were gathered together and moisture content determined. as the plastic limit. The average moisture content of four samples was taken as the plastic limit.

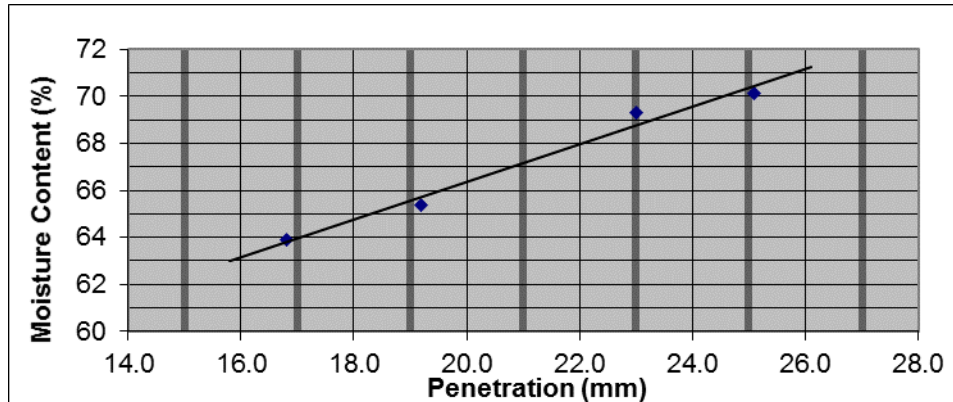


Figure 3.1: Typical plot of moisture content against the cone penetration

(Source: Smith & Smith, 1998)

(iii) Plasticity Index (I_p)

This was calculated as the difference between the Liquid Limit and the Plastic Limit, as follows (BS1377 - Part 2: 1990):

$$I_p = W_L - W_p \tag{3.3}$$

Where:

I_p is the plasticity index

W_L is the liquid limit

W_p is the plastic limit

(iv).Linear Shrinkage

The length of the oven-dry specimen described in 3.2.3.2 (a) (iv) after shrinkage and the original length were measured and the linear shrinkage calculated as a percentage of the original length of the specimen as follows (BS1377 - Part 2: 1990):

$$\text{Percentage of linear shrinkage} = \left(1 - \frac{L_d}{L_o}\right) \times 100. \quad (3.4)$$

Where:

L_d is the length of the oven-dry specimen

L_o is the original length

(v) Free Swell.

This was determined from the initial volume and the final swollen volume described in 3.2.3.2 (a) (v) as follows (Holts & Gibbs, 1950):

$$\text{Free swell} = \frac{L_f - L_i}{L_i} \times 100. \quad (3.5)$$

Where:

L_f is the final volume

L_i is the initial volume

3.2.3.4 Determination of Specific Gravity.

(a).Experimental Set Up

The test which is also called particle size density was done in accordance with BS 1377 – Part 2: 1990 on oven dried soil at 105°C, ground as to pass 2mm BS test sieve.

(b).Data Analysis and Collection Method

Masses of of the density bottle (m_1), density bottle and dry soil (m_2), the bottle, soil and water (m_3) and the bottle full of water (m_4) were taken and the specific gravity computed from Equation 3.6 as follows:

$$G_s = \frac{m_2 - m_1}{(m_4 - m_1) - (m_3 - m_2)} \quad (3.6)$$

3.2.4 Mechanical Tests

These were compaction characteristics, California Bearing Ratio (CBR) and Unconfined Compressive Strength (UCS). The former two tests were done in accordance with BS 1377: (Part 4) 1990, whereas the latter in accordance with BS 1377: (Part 7) 1990.

3.2.4.1 Compaction Characteristics - Dry density/moisture content relationships

(a).Experimental Set Up

The light manual compaction test using a 2.5 kg rammer (Proctor, T99) was used on an air dried pulverized material passing a 5 mm test sieve which is the normal test for soils with particles up-to medium-gravel size (BS 1377, Part 4: 1990). The moisture contents and masses of soil for a specific volume were taken for increasing levels of moisture content.

(b).Data Collection and Analysis Procedure

The bulk density was determined at various moisture contents. The dry density was then computed from the bulk density using Equation 3.7 and a plot of dry density versus moisture content done from which the maximum dry density at the optimum moisture content was determined. From specific gravity determined from section 3.2.3.4, the air voids were computed from the relation given in Equation 3.8 and a plot of air void curves done on the dry density versus moisture content curve (Knappet & Craig, 2012).

$$\rho_d = \frac{\rho}{1+w} \quad (3.7)$$

$$\rho_d = \frac{G_s(1-V_a)}{1+wG_s} \rho_w \quad (3.8)$$

Where:

ρ = bulk density

ρ_d = dry density

ρ_w = density of water

V_a = air voids

G_s = specific gravity

w = moisture content

3.2.4.2 Determination of the California Bearing Ratio (CBR)

(a).Experimental Set Up

According to BS 1377, Part 4: 1990, the CBR value gives the relationship between force and penetration when a cylindrical plunger of a standard cross-section is made to penetrate the soil at a given rate. The CBR then is determined as the ratio of the applied force to a standard force at certain values of penetration, expressed as a percentage. A dial gauge was placed on the compacted sample in a mould, the original reading taken and the material soaked for 4 days (Plate 3.4). After 4 days soak, the final dial reading was taken, and force-penetration measurements were taken for both upper and lower ends of the sample (Plate 3.5). A plot was made of force on plunger versus penetration, from which the CBR then determined.



Plate 3.4: CBR specimens in soaking basin.



Plate 3.5: CBR test – specimen in position

(b) Data Collection and Analysis Method

(i).CBR

In accordance with standard BS1377 (Part 4) 1990 the CBR value was taken as the average resistance to penetration of 2.5 mm and 5.00 mm of a standard cylindrical plunger of cross sectional area of 1935 mm² for the top and bottom of the specimen respectively, expressed as a percentage of 13.2 kN and 20 kN respectively which are the resistances of the plunger in crushed aggregate for the same penetrations.

(ii).CBR Swell

From the initial reading before soaking and the final reading after soaking, the CBR swell was computed as a the percentage using Equation 3.9.

$$\text{CBR Swell (\%)} = \frac{d2-d1}{h} \times 100 \quad (3.9)$$

Where

d2 = the final gauge reading

d1 = the initial gauge reading

h = height of the mould

3.2.4.3 Unconfined Compressive Strength (UCS).

(a).Experimental Set Up

This test was carried out in accordance with BS 1377 (Part 7):1990. The test measures the compressive strength of a cylindrical soil specimen to which no lateral support is offered. Some of the prepared test specimens are shown in Plate 3.6.



Plate 3.6: Test Specimens

A cylindrical cohesive specimen 50 mm diameter and 100 mm high was subjected to an axial load which was increased at a rate of 1 mm per minute until failure occurred, giving the compressive strength of the soil. It was carried out within a short enough time to ensure that no drainage of water is permitted out of the specimen. This was done for the neat and heated soil at temperatures of 200⁰C, 300⁰C, 400⁰C, 500⁰C using a load frame apparatus (Plate 3.7). No specimens were made for temperatures of 600⁰C and above because they could not bond. Load-deflection measurements were taken on a compression machine



Plate 3.7: Specimen in position

(b).Data Collection and Analysis Methods

The axial compressive stress was calculated for each set of readings, and a plot of the compressive stress against the displacement drawn.

The UCS was determined as follows (Smith & Smith, 1998):

$$UCS = \frac{\text{Max Stress at failure}}{2} \quad (3.10)$$

3.2.5 Electro-chemical and Chemical tests

The tests conducted were mineralogy content using the X-ray diffraction (XRD), oxide analysis using the X-ray Fluorescence (XRF), cation exchange tests, organic matter content, loss on ignition, sulphates, the pH and chloride content. The XRF, organic matter content, loss on ignition, sulphates, the pH and chloride content in accordance BS1377 (Part 3), 1990, whereas the cation ion exchange (CEC) tests were conducted in accordance with ASTM D7503-10 (2010).

3.2.5.1 X-Ray Diffraction (XRD)

(a) Experimental Set Up

The X-Ray diffraction analysis was used to establish the mineralogical content of the neat soil and for soil heated at temperatures of 200, 300, 400, 500, 600, 700 and 800⁰C to identify the clay minerals present especially the smectite (montmorillonite) group and albites, and how the mineralogical content changed as the soil was heated. It was done using a benchtop Bruker D2 Phaser diffractometer (Plate 3.9) at the soils laboratories of the International Centre for Research in Agroforestry (ICRAF), Nairobi. The diffractometer was connected to a computer which gave displays of the spectrums obtained similar to those shown in Plate 3.9. X-rays were directed to powdered samples which were scanned through a range of incident angles to attain all possible diffraction directions since the powdered material is randomly oriented.

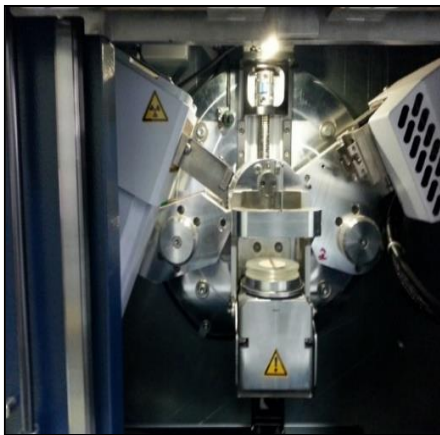


Plate 3.8: XRD (Sample in position)

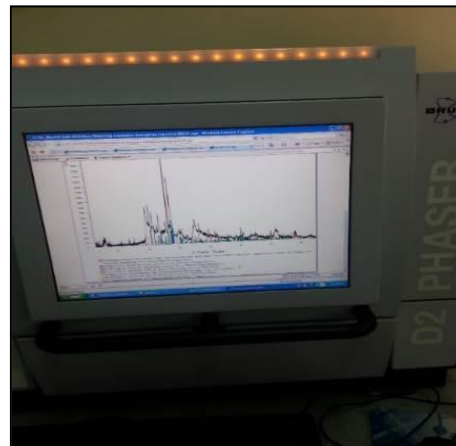


Plate 3.9: Typical Spectra Display

In the test, each air dried sample was made to powder form by milling in a micro-milling mill (Plate 3.10). 9 g of ethanol was added to 5 g of this powdered sample and mixed for 12 minutes. The mixture was transferred to a centrifuge tube and then placed in a centrifuge at 4000 rpm for 10 minutes for the soil to settle at the bottom. The ethanol was then decanted, 0.5 ml of hexane was added to the sample to break up any bonds after which it was well mixed in a vortex mixer (Plate 3.11). The mixture was then oven dried at 80°C for 4 hours, followed by sieving through a 250 micron BS test sieve. The sample was then compacted in a sample holder using a scalp blade (Plate 3.12). The sample in the sample holder (Plate 3.13) was then transferred to the diffractometer (Plate 3.9) for measurement where it was scanned for 30 minutes.



Plate 3.10: Milling



Plate 3.11: Vortex Mixer



Plate 3.12: Compaction

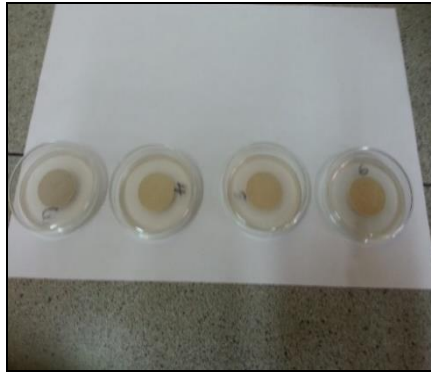


Plate 3.13: Samples in disc holder

(b) Data Collection and Analysis Method

The X-ray diffractometer was connected to a computer which displayed combined plots of the intensity of the diffracted rays against the phase angle, θ for each mineral in the sample (Plate 3.11). Since each crystalline material has its own characteristic atomic structure which diffracts X-rays in a characteristic pattern (Mitchell & Soga, 2005) the diffraction pattern of each sample was compared with a spectrum for known materials to help identify the minerals present. The analysis was based on Bragg's Law (Mitchell & Soga, 2005) given in Equation 2.1 which relates the wavelength of electronic radiation to the diffraction angle, θ and the crystal plane spacing d . An inbuilt software converted the diffraction peaks to d -spacing to allow identification of the mineral since each mineral has a set of unique d -spacing. This software is linked to the International Center for Diffraction Data (ICDD) which maintains the data base of inorganic spectra (Li et al., 2014). The d -spacing of each sample was thus compared with those in the database giving the minerals present.

3.2.5.2 X-ray Fluorescence (XRF) / Determination of the Oxides Content

(a).Experimental Set Up

The X-ray fluorescence test was done on samples ground to a fine powder to analyse the soil for oxide composition. It was carried out using a Ray ny EDX-800HS energy dispersive X-ray spectrometer at the Materials Testing and Research Division of the Ministry of Transport and Infrastructure, Nairobi.

A specimen ground to fine powder was placed on a sample holder and positioned on the XRF spectrometer for analysis (Plate 3.15). X-rays were directed to the specimen, and the resulting X-ray spectrum collected for analysis,

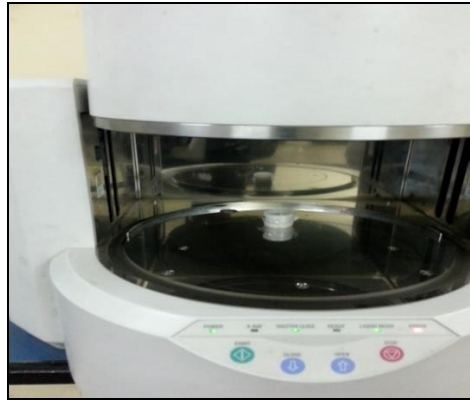


Plate 3.14: XRF test - specimen in position.

(b).Data Collection and Analysis Method

When materials are excited with high energy X-rays, they become ionized (<https://www..xos.com/XRF>). A tightly held inner electron is dislodged and gets replaced by an outer electron. This causes an energy release in form of a characteristic fluorescent X-ray radiation indicating the type of atom present which can be used to detect the

abundance of the elements present in a sample, since the energy of the emitted photon depends on the movement of the electron from one orbit to another in a particular element. Since most samples have many elements present, use of a dispersive spectrometer allows the separation of a complex emitted X-ray spectrum into characteristic wavelengths for each element present. The intensity of the energy released measured using detectors is proportional to the abundance of the element in the sample. In this test, the spectrometer used was calibrated in terms of the various wavelengths from which the elemental composition of the soil was obtained when the sample was run.

3.2.5.3 Organic Matter Content

(a).Experimental Set Up

This test was done to determine the percentage by mass of organic matter present in a soil sample oven dried to 105⁰C. The organic matter was oxidized using a mixture of potassium dichromate solution and concentrated sulphuric acid, followed by a titration process with ferrous sulphate. Titration was also done on a blank without the soil. The loss of ignition was then determined.

(b).Data Collection and Analysis Method

The mass of the soil used was weighed in grams, and from the titration process, the total volumes in ml of potassium dichromate used to oxidize the organic matter, ferrous sulphate used in the mixture and the ferrous sulphate used in the blank were measured. The organic content was then computed using Equations 3.10 and 3.11 as follows:

$$V = 10.5x \left(1 - \frac{Y}{X}\right) \quad (3.11)$$

Where

V = Total volume V in ml of potassium dichromate used to oxidize the organic matter

Y = the total volume of ferrous sulphate used in the mixture

X= the total volume of ferrous sulphate used in the blank.

$$C=0.67x\frac{V}{m} \quad (3.11)$$

Where

C = the percentage organic matter content

m = the mass of soil used in g.

3.2.5.2 Loss on Ignition

(a).Experimental Set Up

This test was done to determine the percentage by mass that is lost from a soil by ignition at 1000⁰C in an electric muffle furnace. A sample was dried at 50⁰C for 12 hours, then passed through a 2 mm BS sieve, from which 10 g was taken and pulverized, passed the 425 microns BS sieve, and divided into 2 equal samples which were placed in pre-dried and weighed crucibles, then dried in an oven at 50⁰C for 4 hours. The crucibles were then placed in an unheated muffle furnace, which was then heated to 1000⁰C for 8 hours. The samples were then removed from the furnace and placed in a desiccator to cool.

(b).Data Collection and Analysis Method

The masses for the dry crucible, crucible and oven-dry soil specimen and the crucible and specimen after ignition were taken. The Loss on Ignition was computed as the percentage loss in mass using Equation 3.12 as follows:

$$LOI (\%) = \frac{m_1 - m_2}{m_1 - m_C} \times 100 \quad (3.12)$$

Where

LOI = Loss on Ignition

m1= mass of the crucible and oven-dry soil specimen (in g)

m2 = mass of crucible and specimen after ignition (in g)

mC = mass of the crucible (in g)

3.2.5.4. Sulphate Test

(a).Experimental Set Up

The test was done for neat black soil and heated black cotton soil at temperatures of 200°C, 300°C, 400°C, 500°C, 600°C and 700°C since the purpose of the test was to determine if the level of the sulphate content would affect soil treatment with lime.

A sample passing the 2 mm sieve was pulverized and passed through the 425 micron sieve. From this, a specimen weighing 10 g was oven dried at 75⁰C and then allowed to cool at room temperature in a desiccator. A 2 g representative sample was put in a weighed 500 ml beaker, the beaker and contents weighed and the exact mass of the specimen established. 100 ml of 10% hydrochloric acid was added into the beaker which was then covered with a cover glass and the contents boiled and simmered in a fume cupboard for 15 minutes. Precipitation of sesquioxides was then done by adding a few drops of nitric acids the specimen continued to boil. Ammonia solution was added from a burette with constant stirring until the sesquioxides were precipitated and a red litmus paper turned blue by the liquid. Filtration of the solution was then done through a hardened filter paper into a 500 mL conical beaker, after which the beaker and the residue were thoroughly washed with distilled water until the washings were chloride free as indicated by absence of turbidity when a drop silver nitrate was added to a small volume of the solution. The filtrate solution was then brought to the boil and 10 mL of barium chloride added drop by drop with constant stirring and the boiling continued until the

precipitate was properly formed. The solution was allowed to stand at just below boiling point for 30 minutes after which it was allowed to cool to room temperature. The liquid and the precipitate of barium sulphate were transferred to a filter crucible. The precipitate was washed several times with hot distilled water until the washings were free from chloride. The filter crucible was then removed from the filter flask, dried for 30 minutes at 110⁰C, and the temperature gradually raised to 800⁰C in a furnace until there was no further loss of mass. The crucible was thereafter cooled at room temperature in the desiccator and the mass of the precipitate obtained by weighing.

(b).Data Collection and Analysis Method

The mass of the test specimen and the ignited precipitate were measured, and the sulphate content as SO₃ and as SO₄ were computed using Equations 3.13 and 3.14 respectively.

$$SO_3 = 34.3x \frac{m_2}{m_1} \% \quad (3.13)$$

Where

m₁ is the mass of test specimen (in g);

m₂ is the mass of the ignited precipitate (in g).

$$SO_4 = 1.2 \times SO_3 \quad (3.14)$$

3.2.5.5 Determination of pH value

(a).Experimental Set Up

This test was done to assess whether the soil can be stabilized with lime since acidic soils are unsuitable for stabilization with lime. 30 grams of an air dried sample passing a 3 mm BS test sieve, was weighed out into a 100 ml beaker after which 75 ml of distilled water was added to it. The suspension was stirred for a few minutes and the beaker covered with a cover glass and after which it was allowed to stand overnight. The suspension was stirred again immediately before testing.

(b).Data Collection and Analysis Method

The electrode of a calibrated pH meter was washed with distilled water and immersed in the soil-water suspension. Three pH readings of the suspension were taken with brief stirring between each reading. The readings were taken when they agreed to within 0.05 pH units; the calibration of the pH meter was rechecked, and results were accepted only when the adjustment of the instrument was within 0.05 pH units. The pH was reported to the nearest 0.1 pH unit.

3.2.5.6 Determination of Chlorides

(a).Experimental Set Up

A 50 grams oven dried sample at a temperature of 105⁰C passing through a 1.0 mm BS test sieve. was placed into a plastic bottle and 250 ml of water added after which the lid was tightened and the mixture shaken for 1 hour. A titration process using 0.1N silver nitrate using potassium chromate as the indicator. The volume of silver nitrate in millilitres used was measured.

(b).Data Collection and Analysis Method

The % chlorides were computed as follows:

$$\% \text{Chlorides} = \frac{V \times 3.55 \times 250 \times 100}{1000 \times 100 \times 50} \quad (3.15)$$

Where:

V = volume of silver nitrate used in millilitres.

3.2.5.8 Cation Exchangeable Capacity (CEC)

(a) Experimental Set Up

The test was done to establish the exchangeable cations and CEC on neat soil and soil heated at temperatures of 200⁰C, 300⁰C, 400⁰C, 500⁰C, 600⁰C and 700⁰C. Four specimens were prepared for each test. The tests were carried out at the National

Agricultural Research Laboratories of the Kenya Agricultural & Livestock Research Institute and Cropnuts Laboratory Services, both of Nairobi.

For each sample, 2.5 g of air-dried soil was leached with 100 ml 1N ammonium acetate buffered at pH 7. The leachate was analysed for exchangeable cations of calcium, magnesium, potassium and sodium. Calcium and magnesium cations were analysed using a flame photometer while potassium and sodium were analysed using an Atomic Absorption Spectrometer. The samples were then washed with 100 ml ethyl alcohol and further leached with 100 ml of 1N potassium chloride at pH 7 and the leachate used to determine the CEC. The CEC, as nitrogen, was determined by distillation followed by titration with 0.01 N hydrochloric acid. Blanks were prepared without the sample, and titration value obtained.

(b) Data Collection and Analysis Method.

Calcium and magnesium cations were analysed using a flame photometer while potassium and sodium were analysed using an Atomic Absorption Spectrometer, from which the respective exchangeable cation was calculated.

After the titration of the distilled leachate the titration value was corrected for blanks, and the CEC computed in terms of meq/100g using Equation 3.16.

$$CEC = (T_1 - T_b) \times N \times \frac{1000}{m} \quad (3.16)$$

Where:

T_1 = Titration value of the leachate

T_b = Titration value of the blank

N = normality of the potassium chloride (= 0.01)

m = mass of the soil sample

3.3 To determine the effect of different proportions of stabilizers on the properties of expansive clay soils.

Tests were done for the properties of the stabilizing agents namely lime and molasses. The expansive soil was stabilized with different proportions of heated soil, lime and molasses, and their strength in terms of UCS determined for 7, 14, 21 and 28 days of moist curing at room temperature to establish an optimum mix. Specimens were then made of the optimum mix, and subjected to various tests after curing periods of 7, 14, 28, 56 and 90 days; The UCS values were determined for various simulated flooding regimes for each of the curing periods and the strength reduction factor determined. A building model of a ground floor slab was done with a fill comprising the optimal mix and displacement of the slab measured for various curing periods of 7, 14, 28, 56 and 90 days; the displacement was also measured for a simulated severe flooding of 10 days after the 90 days curing period.

3.3.1 Materials

The materials used were expansive clay, heated soil, lime and molasses. The properties of expansive clay and heated soil were established as detailed in section 3.2. Properties of lime and molasses were established as detailed in this section. The lime used was hydrated lime sourced from Athi River Mining of Nairobi. It was preferred to quicklime because the latter is exothermic, extremely dangerous and capable of causing severe caustic burns (Mehta et al., 2014; O’Flaherty, 2002) and is not readily available in Kenya. Molasses was sourced from four factories in western Kenya and the molasses meeting the requirements of the Kenya Bureau of Standards was used in stabilization. These factories were Chemilil, Kibos, Sony Sugar and Muhoroni.

3.3.2 Tests on Lime

The tests for lime were loss on ignition, available lime (sugar method), oxide content (Iron, Aluminum, and Magnesium), grading, and bulk density were done in accordance

with KS EAS 73:2000 – (Building limes - quicklime and hydrated lime -specifications and methods)

3.3.2.1 Loss on Ignition

(a).Experimental Set Up

1 g of sample was weighed and heated to constant mass at a temperature of 1000⁰C in a furnace in a crucible for 25 minutes. The sample was then removed from the furnace and placed in a desiccator to cool, and then weighed

(b).Data Collection and Analysis Method

The masses for the dry crucible, crucible and oven-dry soil specimen and the crucible and specimen after ignition were taken. The Loss on Ignition was computed as the percentage loss in mass as shown in Equation 3.12 section 3.2.5.4(b) above.

3.3.2.2 Determination of available lime, as CaO

(a).Experimental Set Up

This was determined using the Sugar method. In this test, 1 g of lime was weighed into a 500 ml volumetric flask and wetted with rectified spirit (a highly concentrated ethanol purified by means of repeated distillation). The mixture was well shaken with 400 ml of sugar solution for three hours. It was then filtered through a coarse dry filter paper into a dry flask. The first 25 ml of the filtrate was discarded and 50 ml pipetted out into another flask to which 50 ml of standard hydrochloric acid was added. The solution was titrated with standard sodium hydroxide solution using phenolphthalein as indicator. A blank was run using 50 ml of sugar solution and 50 ml of standard hydrochloric acid. Then the available lime was computed.

(b).Data Collection and Analysis Method

The available lime (as CaO) was computed as shown in Equation 3.17.

$$\text{Available lime} = 28.04 \times \frac{(A-B)M}{m} \% \quad (3.17)$$

Where:

A = volume in mL of standard sodium hydroxide solution used
the blank.

B = volume in mL of standard sodium hydroxide solution use with
the material

M = molarity of standard sodium hydroxide solution.

m = mass in gm of the material taken for the test

3.3.2.3 Determination of Iron (III) Oxide (Fe₂O₃), Aluminium Oxide (Al₂O₃), Magnesium Oxide (MgO)

(a).Experimental Set Up

The solution used for determination of Iron (III) Oxide Aluminium Oxide and Magnesium Oxide was prepared as follows:

(i).Decomposition

1 g of lime was decomposed by heating with 3 g of Sodium Peroxide in a furnace at a temperature of 500°C. The sintered mass was then cooled to room temperature, transferred to a 400 ml beaker to which 150 ml of water was added, and the mixture heated until it the solids completely dissolved. 50 ml of concentrated hydrochloric acid was added till the mixture was clear, followed by addition of 1 ml of sulphuric acid, after which the mixture was boiled for 30 minutes.

(ii).Precipitation

The decomposed solution was evaporated to dryness and allowed to cool. It was then precipitated by mixing with 5 ml of water and 10 ml of concentrated hydrochloric acid, to which some ash less filter paper pulp and 5 ml of polyethylene oxide solution were added ensuring continuous stirring. 10 ml of water were added followed by stirring, the mixture left to stand for 5 minutes, after which filtration was done through a medium filter paper and the filtrate collected in a 500 ml volumetric flask followed by rinsing with hot hydrochloric acid. The filter and the precipitate were then washed with hot hydrochloric acid, followed by rinsing with hot water until the residue was free from chloride ions as tested by the silver nitrate test. The washings were collected in a 500 ml volumetric flask while the filter and the precipitate were ignited at 175°C to constant mass. The volatilized residue was decomposed and then added to the filtrate and washings in the 500 ml volumetric flask. This solution was used for determination of Iron (III) Oxide (Fe_2O_3), Aluminium Oxide (Al_2O_3), and Magnesium Oxide (MgO).

(iii).Determination of Iron (III) Oxide (Fe_2O_3)

100 ml of the solution prepared as in (ii) above was pipetted into a beaker, made up with water and 0.5 g and 0.35 g of amino-acetic acid and sulfosalicylic acid indicator respectively added, after which the pH of the solution was adjusted to 1.5 with ammonium oxide using a pH-meter. The solution was then heated to 47.5°C followed by a titration process with 0.03 mol/l EDTA solution at a temperature of 50°C and the volume V_{10} of EDTA used in titration established, from which the percentage of Fe_2O_3 was determined.

(iv).Determination of Aluminium Oxide (Al_2O_3)

The solution used in determination of Fe_2O_3 was cooled to room temperature and 5 ml of acetic acid added followed by addition of ammonium acetate solution drop by drop until a pH of 3.05 was obtained. The solution was brought to the boil after which three drops and ten drops of copper complexonate solution and PAN indicator respectively added.

Titration with 0.03 mol/l EDTA solution was done and the volume V_{11} of EDTA used in titration established, from which Al_2O_3 was determined as a percentage.

(v).Determination of Magnesium Oxide (MgO)

25 ml of the solution prepared as in (ii) above was pipetted into a beaker, made up to volume with water after which 25 ml of triethanolamine solution was added. The pH was adjusted to 12.5 with a sodium hydroxide solution; titration was done using 0.03 mol/l EGTA solution and the volume V_{12} used for titration established. After this, 50 ml of the solution prepared as in (ii) above was pipetted into a beaker, 50 ml of triethanolamine solution added together with a volume V_{13} ml of EGTA solution computed as shown in Equation 3.18 followed by dilution with water, after which the pH was adjusted to 10.5 with concentrated ammonium hydroxide. 0.1 g of methyl thymol blue indicator was added and titration with 0.01 mol/l DCTA solution was done the volume V_{14} used for titration established, from which MgO was determined as a percentage.

$$V_{13} = 2 V_{12} + 1.5 \tag{3.18}$$

(a) Data Collection and Analysis Method

The volumes V_{10} , V_{11} and V_{14} were established and the percentages of Fe_2O_3 , Al_2O_3 and MgO were calculated using Equations 3.19, 3.20 and 3.21 respectively.

$$Fe_2O_3 = 1.1977 \frac{V_{10} X f D}{m_{23}} \tag{3.19}$$

$$Al_2O_3 = 0.7647 \frac{V_{11} X f D}{m_{23}} \tag{3.20}$$

$$MgO = 0.4031 \frac{V_{14} X f D}{m_{23}} \tag{3.21}$$

Where

V_{10} and V_{11} are the volumes of 0.03 mol/l EDTA solution used for the titration V_{14} is the volume of 0.01 mol/l DCTA solution used for the titration.

f_D is a factor used in standardizing the 0.03 mol/l EDTA solution;

m_{23} is the mass of the lime test portion.

3.3.2.4 Grading.

(a).Experimental Set Up

This was done to establish the amount of residue retained on 0.2 mm and 0.075mm test sieves. 100 g of lime was placed on the 0.075mm test sieve after which it was sieved by washing with water passed through a spray nozzle at a pressure of 140 kPa. The residue was then dried and transferred to a container. The 0.2 mm sieve was nested above the 0.075 mm sieve, after which the residue was placed on the 0.2 mm sieve. Sieving was then done through both sieves by washing with water through a spray nozzle at a pressure of 140 kPa, until the water became clear. The residue in both sieves was then dried to constant mass at a temperature of 80°C, and then weighed.

(b) Data Collection and Analysis Method

The fineness for the 0.2 mm sieve and the 0.075 mm sieves were computed as shown in Equations 3.22 and 3.23 respectively.

0.2 mm sieve

$$\text{Fineness (\% m/m)} = \frac{W_1 \times 100}{M} \quad (3.22)$$

Where,

W_1 = weight of dried residue on 0.2 mm sieve

M = sample weight

(ii) 0.075 mm sieve

$$\text{Fineness (\% m/m)} = \frac{(W1+W2) \times 100}{M} \quad (3.23)$$

Where

W1 = weight of dried residue on 0.2 mm sieve

W2 = weight of dried residue that passed the 0.2 mm sieve but was retained
on the 0.075 mm sieve

M = sample weight

3.3.3.1 Bulk density

(a) Experimental Set Up

A lime sample was poured into a pre-weighed graduated cylinder of mass M_1 , and the volume V taken. The weight of the cylinder with the sample was taken and recorded as M_2 . The bulk density was then determined.

(b) Data Collection and Analysis Method

The bulk density was computed as shown in Equation 3.24

$$\rho = \frac{m_2 - m_1}{v} \quad (3.24)$$

Where,

ρ = bulk density

m_2 = mass of the cylinder and the sample

m_1 = mass of the cylinder

v = volume of the sample

3.3.3 Tests on Molasses

The tests conducted were brix test, tests for total invert sugar, total ash, calcium, phosphorous, potassium, magnesium, sulphated ash, density, pH and sucrose. They were conducted as per KS 953:2008- Molasses for industrial use – Specification, except total ash and sulphated ash which was done as per KS 58: 2007 - Sampling Methods and Chemical Analysis of Sugar – Test Methods, whereas potassium and magnesium were determined as per KS 63: 2000 (Method of test for animal feedstuffs - 3rd Edition)

3.3.3.2 Brix Test

(a).Experimental Set Up

The test which is a measure of the dry matter was done using the refractometer method. A sample of molasses was diluted with water to make a 50% solution. A few mL of this solution were pipetted through a clarity filter rubber. It was then pipetted and poured onto the glass prism cover of the refractometer.

(b).Data Collection and Analysis Method

With the molasses solution as prepared in (a) above placed on the refractometer, the brix was measured by pressing the read function on the refractometer; the reading obtained was multiplied by two to get the result. The reading is measure of the refractive index of the molasses, as light passes through it.

(a).Experimental Set Up

5 g of raw molasses were dissolved in distilled water and transferred to a 250 ml volumetric flask to which 10 ml of neutralized lead acetate solution was added, and the mix vigorously shaken. 5 ml of potassium oxalate solution was then added and the mix shaken again. The flask was filled with distilled water, the solution well mixed and allowed to precipitate. This was then filtered and the filtrate collected for analysis by a titration process.

(b).Data Collection and Analysis Method

Once the filtrate in (a) above was taken through the titration process, the total invert sugar was determined from conversion tables which gave the sugar level from the titrate value.

3.3.3.3 Determination of Total Ash

(a).Experimental Set Up

10 gm of molasses was heated in a 100 ml platinum dish at 100°C, until all the water was expelled. A few drops of pure olive oil were added and the mixture heated slowly over a flame until swelling stopped. The mixture was then ashed by placing the dish and contents in a furnace at a temperature of 525°C and left until white ash was obtained. The ash was then cooled, moistened with water, dried on a steam bath and finally on a hot plate. The mixture was then re-ashed at 525°C to constant weight, and the mass of the ash taken.

(b).Data Collection and Analysis Method

Once the molasses was ashed, the total ash was computed as the ratio between the mass of the ash and the original mass of the molasses expressed as a percentage.

3.3.3.2 Determination of Density

(a).Experimental Set Up

The diluted molasses from the invert sugar test was transferred into a 100 mL volumetric flask to the 100 mL mark, after which the weight was taken from which the density was determined.

(b).Data Collection and Analysis Method

After taking the weight of the 100 ml of molasses the density was calculated from Equation 3.25.

$$\text{Density} = 2x \frac{\{(\text{Weight of 100 mL}) - 1\}}{100} \text{ g/cm}^3 \quad (3.25)$$

3.3.3.5 Determination of Calcium as Ca²⁺

(a).Experimental Set Up

2 g of molasses were converted to ash in a furnace at 800°C, from which 0.001 g of ash was taken and put into a 100 ml volumetric flask. Drops of HCl were added and then allowed to stand to hydrolyze. Distilled water was added, the mixture thoroughly shaken and made to volume with distilled water. 25 ml of the homogenized sample was pipetted out into a conical flask, after which 5 mL of ammonium buffer solution was added and 0.1 g of ferrochrome black indicator. Titration was done against 0.02 N EDTA (disodium ethylene diamine tetra acetate).

(b).Data Analysis and Collection Method

The titrate value from the titration and the weight of the sample were taken and the % calcium content was calculated using Equation 3.26

$$\% \text{Ca}^{2+} = \frac{(T \times 0.0008016 \times 100 \times 100\% \text{ ash})}{W \times 25 \times 100} \quad (3.26)$$

Where:

T = the titrate value

W = the weight of the sample

3.3.3.6 Determination of Potassium and Magnesium

(a).Experimental Set Up

2 g of molasses diluted to a known concentration was weighed and converted to ash in a furnace at 800°C, after which it was ignited at 550°C. It was then dissolved in 5 ml 5N HNO₃, followed by filtration into a 500 ml volumetric flask. The filter paper was washed until free from chlorides. The filtrate was then placed in a flame photometer, and the concentration of potassium determined from a calibration chart. The test was similar for determination of magnesium, except that the filtrate was placed in a calibrated micro

plasma atomic emission spectrometer, from which the concentration of magnesium determined.

(b).Data Collection and Analysis Method

The concentration of potassium was determined from a calibrated curve of a flame photometer, while the concentration of magnesium was determined from a calibrated micro plasma atomic emission spectrometer. The concentrations were measured in parts per million.

3.3.3.7 Determination of Phosphorous as P

(a) Experimental Set Up

4 g of sample were weighed to an accuracy of 0. 0001 g in a conical flask, to which 30 ml of water and 20 ml of concentrated. nitric acid were added followed by gentle boiling for 40 minutes until organic matter was oxidized and the solution became near clear. The mixer was then filtered through a 540 filter paper into a 500 ml conical flask after which the filter was washed 5 times with 20 ml portions of distilled water. 30ml quimociac was added followed by heating to boiling on a hot plate for 1 minute. The flask was then removed from the hot plate, swirled and let it cool to room temperature as the precipitate settled. Filtration through a cold pre-weighed P4 cintered crucible previously conditioned to constant mass at 250°C was done after which the precipitate was washed thoroughly with distilled water. The crucible was oven-dried at 250°C to constant mass followed by cooling to room temperature in a desiccator after which the weight was taken.

(b).Data Collection and Analysis Method

The phosphorous content as P₂O₅ was determined from Equation 3.27 from which the amount of phosphorous as P was determined as follows from Equation 3.28:

$$\text{Phosphorus as P}_2\text{O}_5 \% = 3.207 \times \frac{M_1}{M_0} \tag{3.27}$$

Where

M_0 = mass in grams of the test portion,

M_I = mass, in grams of the precipitate.

Phosphorus as P % m/m = 0.4364 x % P_2O_5 . (3.28)

3.3.3.8 Determination of Sulphated Ash

(a).Experimental Set Up

A sample 5 g of molasses was weighed into a pre-dried and weighed crucible, after which it was moistened with 0.5 ml of concentrated sulphuric acid. The solution was heated gently in a fume chamber until the sample was carbonized, after which it was placed in a muffle furnace at 550⁰C for two hours. The sample was removed, cooled and moistened again with two drops of concentrated sulphuric acid and put in the furnace at 800⁰C until the carbon disappeared after which it was cooled in a desiccator to room temperature. The ash was then weighed.

(b).Data Collection and Analysis Procedure

The sulphated ash was determined as the ratio between the ash residue from the heated molasses and the weight of the sample of the molasses taken, expressed as a percentage in indicated in Equation 3.29.

$$\text{Sulphated ash} = \frac{W_a}{W_s} \times 100 \quad (3.29)$$

Where:

W_a = mass of the ash residue

W_s = mass of the sample

3.3.3.9 Determination of pH

(a).Experimental Set Up

An electrometric pH meter was standardized 80:2:6:12 against two standard buffer solutions, one having a pH value of 4 and the other a value of 9. 10 g of the molasses sample was dissolved in a 100 mL of water in a beaker at room temperature.

(b).Data Collection and Analysis Procedure

The pH of the prepared solution of molasses was measured using the standardized pH meter. Two measurements were taken, and the mean of the two pH values reported as the pH of the molasses.

3.3.4 Tests on Stabilized Materials

Air dried black cotton soil was stabilized with varying mixes of heated soil, lime and molasses. Wang et al. (2008) found that the highest temperature they heated expansive clay was 600⁰C for it to lose swelling characteristics. In this research the trial mixes were thus done with temperatures of 200⁰C, 300⁰C, 400⁰C, 500⁰C 600⁰C and 700⁰C. A hand mixing process was adopted to ensure a homogeneous mix; As noted in section 2.12.4.5, the total binder contents more frequently in use range from 16 to 25 %. Therefore this adopted a maximum stabilizer content of 20%. The specimens made were cylindrical 50 mm in diameter and 100 mm in length at moisture contents of 24%, 28%, 32%, and 36% enveloping the optimum moisture content of the neat material. Compaction was done in three layers and the specimen extruded with a metal plunger from a cylindrical mould, after which they were wrapped with cling film to regulate moisture movement. The mix ratios used were as in Table 3.1, and UCS tests were done in accordance with ASTM D559/D559M-15 (2015): Standard Test Methods for Wetting and Drying Compacted Soil-Cement Mixtures. The method is used to determine the resistance of compacted soil-cement specimen to repeated wetting and drying. This was done on each specimen after moist curing at room temperature (6 Plate 3.14) for periods of 7, 14, 21, and 28 days. A

total of 320 specimens were made for each temperature, for heating at 200 and 300°C, and 272 for the other temperatures, making a total of 1728 specimens. From tests on these specimens the optimum mix was established for the specimen giving the highest value of UCS, at an acceptable temperature of heated soil, lime and molasses content. Specimens were then made for the optimum mix, making four specimens for each test. The tests conducted were:

- Physical tests comprising particle size distribution, atterberg limits and free swell. They were done as described in 3.2.3.
- Mechanical tests which included compaction characteristics, California Bearing Ratio (CBR) and Unconfined Compressive Strength (UCS) were done as described in 3.2.4. As discussed in section 2.9.2.1(b), there is a trend to use UCS as an alternative to CBR as the default test for strength for road works (Thomas, 2001) since UCS is better than CBR as a strength test because CBR being a penetration method is not good for soils taking tension as in roads (Bandara et al., 2017). The CBR tests also uses too much material (Thomas, 2001). Thus the strength tests for the trial mixes and the optimum mix for the various curing and flooding regimes were done in terms of UCS. However, since the 7 day cure 7 day soak CBR test is the standard test for stabilized materials, it was done for the optimum mix in this research to compare the results with strength requirements in Kenya which are still in terms of CBR (MOTC, 1986; MOTC, 1987).
- X-ray diffraction (XRD) and cation exchange capacity (CEC) tests were done as described in 3.2.5.

The tests were done after moist curing at room temperature for periods of 7, 14, 28, 56, and 90 days, with samples wrapped in cling film to prevent rapid moisture loss (Plates 3.16). To simulate different flooding scenarios, UCS tests were done for specimens partially soaked in water for 4 and 10 days to simulate a normal flood, and fully soaked/submerged for 4 and 10 days to simulate severe flooding, after each of the moist

curing periods (Obuzor et al., 2011 ; Obuzor et. al. 2012). Hossain, Lachemi & Easa (2006) considered a moist curing period of 84 days followed by immersion in water for 7 days. The simulation by Obuzor et al. (2011) and Obuzor et al. (2012) was adopted since it represents a more severe scenario. The partial soaking was achieved by immersing 10 mm of the specimen in water.

A building model measuring 2x2m (Plates 3. 18 and Figure 3.2) was done for a fill of 1.0 m comprising the optimal mix and displacement measured for various curing periods. 7, 14, 28, 56 and 90 days moist cure, followed by 10 days side flooding. To ensure water access to the fill during the curing and flooding, gaps were left in the walling during construction. Sand was used as backfill between the outer wall and natural ground, to help retain moisture. The sand was wetted continuously during the curing period, and flooded for 10 days after the end of 90 days. The slab was constructed free of the wall by using a 25mm separation joint formed by a compressible material. A live loading of 3 kNm⁻² representing classroom loading (BS 6399 Part 1, 1996: Loading for Buildings) was simulating by providing a uniform layer of soil on the slab, whose depth was based on its bulk density. Steel pointers embedded in the slab was used to measure the displacement with a datum set on the wall which was founded on a hard strata.



Plate 3.15: Specimens curing



Plate 3.16: Partial soaking

Table 3.1: Mix proportions for stabilization (in percentages)

Neat Material	Heated Soil	Lime	Molasses
80	5	0	15
80	10	0	10
80	15	0	5
80	20	0	0
80	6	2	12
80	10	2	8
80	14	2	4
80	18	2	0
80	4	4	12
80	8	4	8
80	12	4	4
80	16	4	0
80	4	6	10
80	6	6	8
80	10	6	4
80	14	6	0
80	6	8	6
80	8	8	4
80	10	8	2
80	12	8	0

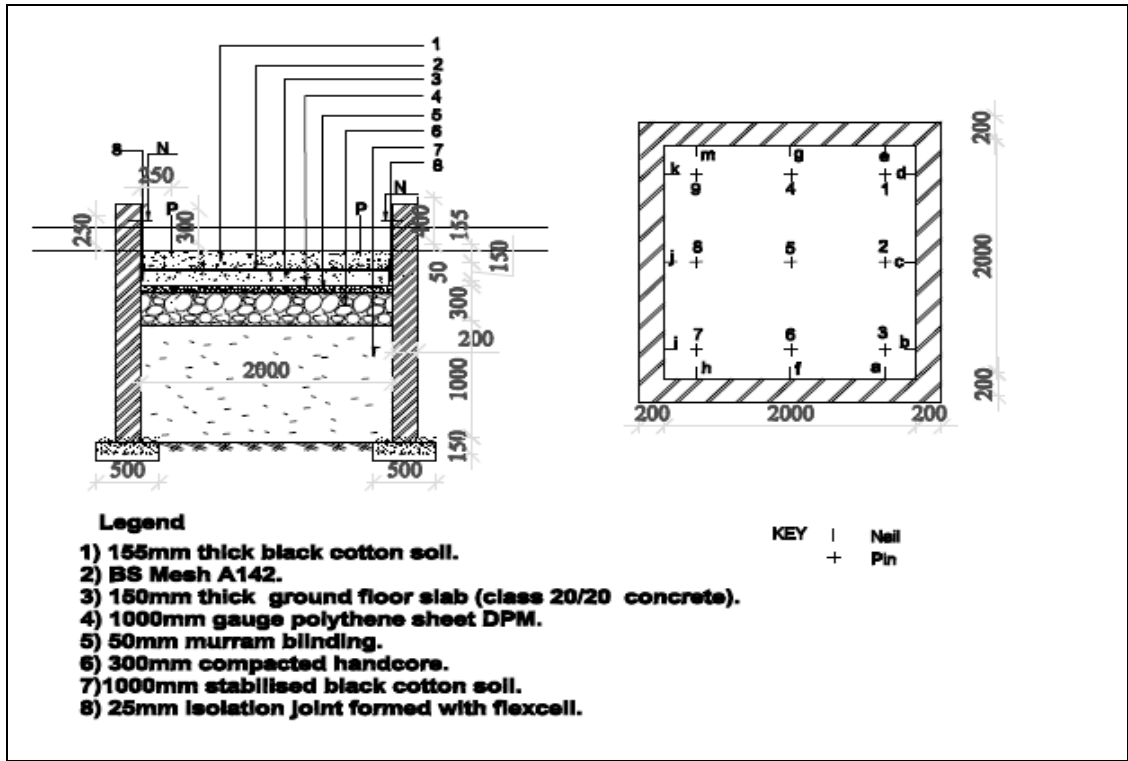


Figure 3.2: Schematic drawing of model house



(a) Walling under construction



(b) Progressive filling



(c) Stabilized fill curing

Plate 3.17: Building Model under Construction

3.3.4.1 Trial Mixes

Using the same procedure as described in 3.2.4.3, experimental set up, data collection and analysis UCS was done for the mixes shown on Table 3.1 for moist curing periods of 7, 14, 21 and 28 days from which the optimal mix was established.

3.3.4.2 Optimal Mix

(a) Physical Tests.

The experimental set up, data collection and analyses for particle size distribution, specific gravity, atterberg limits and free swell were done using the procedure described in section 3.2.3.

(b) Mechanical Tests

The experimental set up, data collection and analyses for compaction, CBR and UCS were done as in section 3.2.4. From the UCS results, the strength reduction factor for each of the flooding regime was computed as the ratio of the UCS value for the flooded specimen divided by the UCS for the un-flooded specimen multiplied by 100 (Hossain et al., 2006; Obuzor et al., 2011 ; Obuzor et. al. 2012).

(c).XRD and Exchangeable Cations Tests.

XRD, exchangeable cations and CEC data collection and analyses were done as described in sections 3.2.5.1 and 3.2.5.2 respectively.

(d).Building Model

Displacement measurements for the fill curing periods of 7, 14, 28, 56 and 90 days were done using steel pegs embedded in the concrete slab relative to a datum set on nails on the walling which was founded on a firm strata. These were collected and the displacement against the curing period measured.

3.4 Evaluating the economic viability of using heated soil, lime and molasses as stabilizers for expansive clay soils.

3.4.1 Introduction

The economic viability was done to obtain a comparative cost of the following construction methods per cubic meter of fill:

- Filling using conventional methods of cutting and disposing the expansive clay followed by replacement with suitable material for roads and buildings.
- Filling using improved materials from this research for roads. Expansive clay heated at 700°C, and expansive soil stabilized with 6 % lime and 14% soil heated at 400°C were established as suitable fill materials.
- In all the comparisons, discounted costs were not considered because all the costs were for the base year.

3.4.2 Construction options

3.4.2.1 Cutting and replacing with suitable fill material.

1m depth of cutting and replacement will be considered since in road construction, excavation of the expansive soil can be done to a depth of about 1 m and carting it to spoil, followed by replacement with a suitable material, of at least class S2 material with CBR of 5-10% (MOTC, 1987), with the requirement that the top 300mm below formation should have a minimum CBR of 8% with the swell and plasticity index less than 1% and 50% respectively (MOTC, 1986). In accordance with MOTC (1986), the process involves cutting to spoil in soft material, compaction of 150mm thick original ground under fills below subgrade, filling in hard or soft material compacted to 95% MDD (AASHTO T99) in layers of 150mm each and an extra-over item for fill in soft material for compaction of top 300mm below formation in fills/subgrade to 100% (AASHTO T99). The fill in hard or soft material depends on which material is readily available. In buildings the excavation and filling process involves excavating over site to reduce levels in normal material for depths not exceeding 1.5m from existing ground level, carting away from site surplus excavation spoil away from site and approved red soil or murrum fill in 150 mm compacted layers, or approved well compacted hardcore depending on availability (MTIH & UD, 2017).

3.4.2.2 Filling using improved materials from research.

The process is as in section 3.4.2.1 except that the fill material is the improved material, which was the heated black cotton soil at 700°C and stabilized material. The material was processed in place without need for excavation.

3.4.2.3 Data Collection and Analysis Methods.

The cost data was obtained as secondary data from the current construction costs from the Kenya National Highways Authority for roads, as shown in Appendix L. The current costs for buildings were obtained from the State Department of Public Works of the Ministry of Transport, Infrastructure, Housing and Urban Development in Kenya (MTIH&UD, 2017). The cost of heating was established from the heat capacity of soil, while the cost of lime was established from the factory costs, adjusted for transport costs and profit. The costs for cutting and replacing expansive clay with suitable material, the cost of using the improved material and the cost of the suspended ground floor slab were then established.

CHAPTER FOUR

RESULTS ANALYSIS AND DISCUSSION

4.1 Introduction

This chapter presents the results analysis and discussion for the properties of heated black cotton soil and the black cotton soil stabilized with mixes of heated black cotton soil lime and molasses. The properties of the neat black cotton soil were established before the soil was heated. The analysis methods are already discussed in chapter 3. The analysis and discussion is primarily based on fill materials for road construction. However, consideration has also been given for filling in buildings because the swelling characteristics also affect the performance of ground floor slabs. The economic viability of use of the heated black cotton soil and the optimal stabilized black cotton soil is also presented. The cost rates were obtained from secondary data.

4.2 Properties of Heated Soil

The results discussed are the physical, mechanical and electro-chemical properties for both the neat and the heated black cotton soil from laboratory tests to establish the effect of heated on the soil.

4.2.1 Physical Properties

The results for the physical properties are grading, swelling characteristics and specific gravity. These are discussed hereunder.

4.2.1.1 Particle Size Distribution/Grading

The grading results are shown in Figure 4.1 for the neat black cotton soil (NBC) and heated black cotton soil (HBC) at temperatures of 200°C to 1200°C, from the data shown in Appendix A.

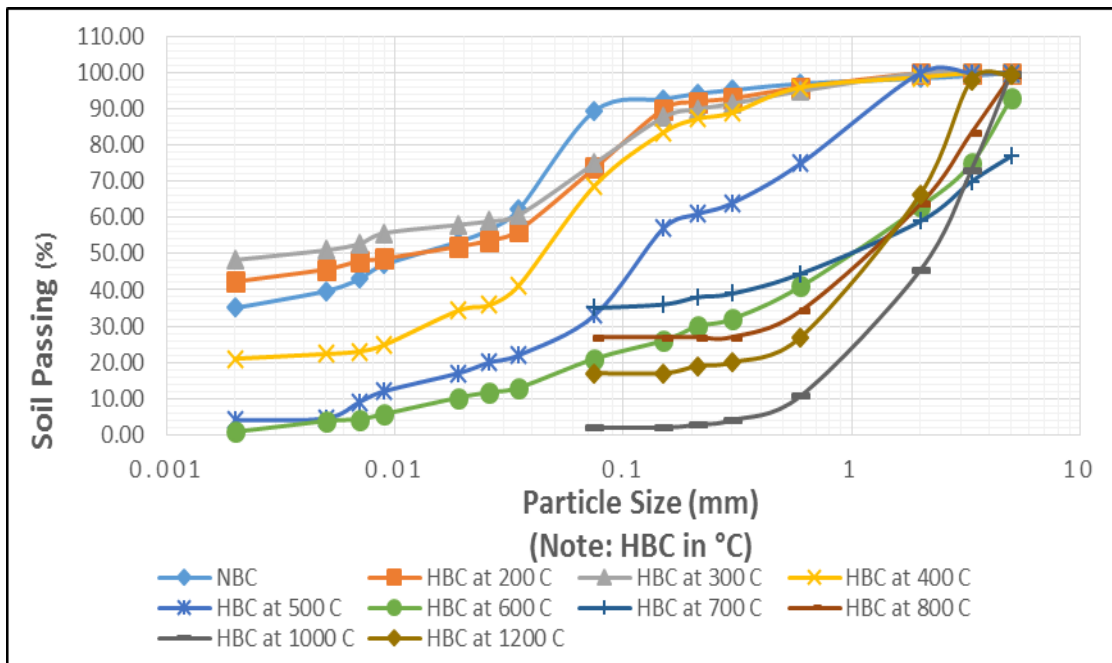


Figure 4.1: Particle Size Distribution

The size ranges of various soil types is shown in Figure 2.14. As discussed in section 2.8, a soil is either coarse or fine. Fine soils have at least 35% by mass finer than 0.06 mm while coarse soils have less than 35% finer than 0.06 mm. From Figure 4.1, the percentage composition of each sample in terms clay, silt, and gravel were read off. Thus, the NBC, HBC at 200, 300 and 400 °C were fine, while HBC at 500°C, 600°C, 700°C, 800°C, 1000°C and 1200°C were coarse.

As heating progressed, the clay fraction increased up to a temperature of 300 °C, then decreased eventually diminishing at a temperature of 600°C. Further, the silt fraction

decreased erratically, becoming negligible at 1000°C. The HBC at 400°C and 500°C had high sand contents, while at 600°C and above, the gravel fraction increased sharply. Expansive soils are fine soils with a high clay fraction, organic silts and flake-shaped inorganic silts. Other inorganic silts, sands and gravels are non-expansive (Mitchell & Soga, 2005). The increase in the clay fraction for the HBC at 200°C and 300°C could be attributed to loss of free and double layer water, together with combustion of organic matter (Kornelius & Akinshipe, 2017; Li et al., 2014; Bell, 1993). Organic matter is a powerful aggregating agent which helps to aggregate the clay, silt and sand. Its loss thus reduces cohesiveness of soil. Therefore the NBC and HBC at 200°C and 300°C would be expected to exhibit expansive characteristics because of the clay fraction. Soils heated at a temperatures of 400°C would be expected to have reduced expansive characteristics, while soils heated at temperatures higher than 400°C would be expected to have negligible expansive properties. Fill materials should have less of the clay fraction to reduce swelling influence. As the temperature increased the HBC became a better fill material on the basis of the grading. However, particle size distribution on its own is not conclusive in determining expansive properties of soils.

The grading types at the various temperature ranges are shown on Table 4.1. D_{10} , D_{30} and D_{60} were obtained from the particle distribution curves shown in Figure 4.1 from which the coefficient of uniformity (C_u) and coefficient of curvature (C_c) were determined from Equations 2.3 and 2.4 respectively. As discussed in section 2.8.2 a soil is well graded if C_u is 5 or more and C_c is between 1 and 3; a poorly graded has C_u equal to or less than 2; a gap graded soil is a poorly graded soil with a near-horizontal “hump” or “step” in its gradation. The neat soil, the HBC at 200°C, 300°C, 400°C, 700°C, 800°C and 1200 °C do not have a D_{10} value thus C_u and C_c cannot be defined and are therefore gap graded. The neat soil and HBC at 200°C, 300°C and 400°C have particles with sizes at the lower and upper sizes, while the HBC at 800°C has a horizontal step in the lower sizes. The HBC at 500°C and 600°C have C_u greater than or equal to 5 and C_c between 1.1 and 3, are well graded with a smooth and upward curve. The HBC at 1000°C has C_u value of

4.3, which is fairly close to 5 and a C_c of 1.1 making it fairly well graded which is also apparent from the grading curve. Well graded soils compact to a dense mass (O’Flaherty, 2002). Thus HBC at 500°C, 600°C and 1000°C will compact to a dense mass, and on the basis of the grading they would make better fill materials than the others.

Table 4.1: Grading Types

Reference	D ₁₀	D ₃₀	D ₆₀	Cu	Cc	Grading Type
NBC	0	0	0.034	-	-	Gap graded
HBC-200°C	0	0	0.042	-	-	Gap graded
HBC-300°C	0	0	0.040	-	-	Gap graded
HBC-400°C	0	0.013	0.065	-	-	Gap graded
HBC-500°C	0.0064	0.060	0.212	33	2.7	Well graded
HBC-600°C	0.019	0.212	1.8	95	1.3	Well graded
HBC-700°C	0	0	2.0	-	-	Gap graded
HBC-800°C	0	0.43	1.8	-	-	Gap graded
HBC-1000°C	0.60	1.3	2.6	4.3	1.1	Well graded
HBC-1200°C	0	0.65	1.8	-	-	Gap graded

As a clay is heated beyond 400⁰C the crystal structure of clay breaks down irreversibly , it losses the combined or chemically bound water from de-hydroxylation leading to formation of larger particles possibly due to electro-chemical bonds resulting in formation of larger particles (Kornelius & Akinshipe, 2017; Li et al., 2014; Wang et al., 2008). According Bell (1993) soils at temperatures of between 200°C and 400⁰C begin to aggregate into granules and when the soil is heated to temperature of 400 ⁰C to 600 ⁰C.

This explains the reduction in the clay fraction from 400⁰C to 600⁰C. Some irreversible changes occur which make the soil non-plastic and non-expansive. Thus the structure of the black cotton under study broke down irreversibly, it experienced de-hydroxylation followed by an irreversible formation of larger particles/granules. According to Bell (1993) fusion and verification sets in at temperatures of 600⁰C and 700⁰C explaining the sharp increase in the gravel fraction. With further increase in temperature, there was some fusion and verification, and a brick like material was obtained, which was similarly reported by Bell (1993).

4.2.1.2 Atterberg Limits and Free Swell

The results of the Atterberg limits and free swell for the NBC and HBC were computed as highlighted in section 3.2.3.2 and tabulated in Appendix B. The variation of the swelling properties is shown in Figure 4.2 for the NBC and the HBC at temperatures of 200⁰C, 300⁰C, 400⁰C, 500⁰C, 600⁰C and 700⁰C. As illustrated in Appendix B, no results could be obtained for HBC at 800⁰C and above because they were cohesion less. The neat black cotton soil under study had a liquid limit (W_L) value of 65%, plasticity index (I_P) of 30%, linear shrinkage (LS) of 15% and a free swell (FS) of 74%. Classification by O'Neill and Poormoayed (1980) shown on Table 2.7 indicates that the neat soil under study had a high swell potential since the W_L exceeded 60%, while the I_P was 30%. The test results are very close to black cotton soils from western Kenya shown on Table 2.4 which had W_L ranging from 55 to 61% and I_P from 26 to 33% and LS from 12 to 17%. According to Bhavsar and Patel (2014), neat black cotton soils in India have W_L values ranging from 50 to 100%, I_P from 20 to 65%, and a LS from 9 to 14%. The results for the soil under study fit closely in this range, and it was therefore an expansive clay.

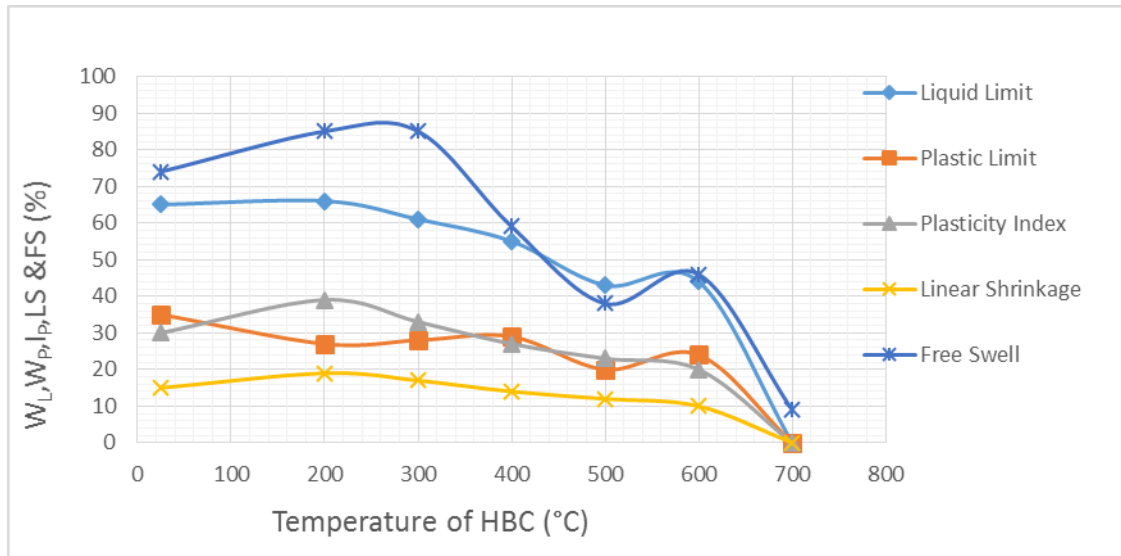


Figure 4.2: Variation of WL, Wp, Ip, LS and FS with Temperature

As illustrated in Figure 4.2, as the heating progressed the W_L , the I_p , LS and FS increased to a maximum value at a temperature of about 250°C after which they all decreased to a minimum at 700°C with the W_L and FS falling very sharply. The W_p decreased erratically; the W_L , W_p , I_p and LS became zero at 700°C . As the temperature was increased, the crystalline structure of clay mineral underwent irreversible chemical changes. The soil experienced reduction in swelling characteristics as the heating progressed beyond 250°C ; significant reductions in the W_L (10%) and FS (15%) occurred at 400°C ; at 600°C , the W_L , W_p , I_p , and FS fell very significantly by 21%, 11%, 10%, 10% and 28% respectively; the W_L , the W_p , the I_p and LS diminished at 700°C , while FS reduced by 65% to 9%. The HBC at 800°C and above were non-plastic.

As indicated in Table 2.7, a soil with W_L and I_p less than 50 and 25% respectively has a potential swell of less than 0.5%, and is classified as having a low swell potential. Holtz and Gibbs (1956) suggested that soils having a free swell below 50 seldom exhibit appreciable volume change even under light loadings, since swelling causes most damage on lightly loaded structures (Wiseman et al., 1985). MOTC (1987) limits the I_p to a

maximum of 50. The soil under study had I_P less than 50, which dropped to 23%, 20% and 0% at 500°C, 600°C and 700°C respectively while the W_L dropped to 43%, 44% and 0% at 500°C, 600°C and 700°C respectively. The FS dropped to below 50% for soil heated at 500°C and over, reducing to 9% at 700°C. Thus at 500°C, 600°C and 700°C, the soil had a low swell potential and will hardly experience a volume change even under light loads. Based on swelling characteristics, expansive soils heated to 500°C, 600°C and 700°C are therefore suitable for use as fill material, on the basis of swelling characteristics.

According to Bell (1993), the fine particles begin to aggregate into granules as heating is increased and at temperatures of 400°C to 600°C, some irreversible changes occur which make the soil non-plastic and non-expansive. From Figure 4.2, the W_L was 55, 43 and 44 for temperatures of 400°C, 500°C and 600°C respectively, while I_P was 27, 23 and 20 for the same temperatures respectively. From Table 2.7, the swelling potential classification started with marginal for 400°C, to low for 500°C and 600°C indicating that the soil at 500°C and above is non-plastic and non-expansive. The reduction in plasticity occurred due to destruction of the crystalline structure of expansive soils from de-hydroxylation as the temperature increased beyond 400°C, resulting in formation of granules possibly due to electro-chemical bonds. Beyond 600°C there was some fusion and verification, and a brick like material was obtained, which was similarly reported by Bell (1993) explaining the diminishing of swelling characteristics.

4.2.1.3 Swell Classification

From Table 2.7 and Figure 4.2, the NBC and HBC at 200°C are classified as having high swell potential, with liquid limit greater than 60% and plasticity exceeding 35%. HBC at 300°C and 400°C have a marginal swell potential, whereas HBC at 500°C, 600°C and 700°C have low swell potential. On the basis of swell potential, HBC at 500°C, 600°C and 700°C are suitable as fill materials for roads and ground floor slabs for buildings.

4.2.1.4 Activity of the Soil

The activity of a soil is defined as indicated in Equation 2.6 as the ratio of plasticity index to the percentage by mass finer than 0.002 mm. The activity levels for the various temperature ranges are shown in Table 4.2.

Table 4.2 : Variation of Activity with Temperature

Reference	Ip (%)	% finer than 0.002mm	Activity	Classification
NBC	30	35	0.86	Normal
HBC at 200 ^o C	39	42	0.93	Normal
HBC at 300 ^o C	33	48	0.69	Inactive
HBC at 400 ^o C	27	21	1.29	Active
HBC at 500 ^o C	23	4	5.75	Active
HBC at 600 ^o C	20	0	-	-
HBC at 700 ^o C	0	0	-	-

As discussed in section 2.9.2.2, activity values are grouped into 5 ranges, viz; inactive has two groups with activity < 0.5 and 0.5-0.75; Normal with activity 0.75-1.25 and two active groups with activities 1.25-2 and >2. the NBC and HBC at 200^oC were therefore classified as normal; HBC at 300^oC as inactive; HBC at 400^oC and at 500 as active; Activity for HBC at 600^oC and higher could not be defined since the clay fraction was 0%. The heated soils are modified thus activity value for heated soil may be of no significance. Since smectite clays have activity values of between 1 and 7 (Table 2.8), the soil under study was a mixed clay because the activity of the NBC was 0.86. This means that the swelling characteristics of the neat soil under study were slightly lower than that of the smectites. From XRD tests shown in Table 4.5, the NBC was a mixed clay. The HBC at 400^oC and 500^oC are classified as active, indicating that the mineralogy changed as a result of heating. The results from swelling characteristics discussed in

section 4.2.1.2 however indicate the swelling behaviour became insignificant for HBC at 500 °C and over, though no values of activity could be obtained for HBC at 600°C and 700°C.

4.2.1.5 Soil Classification

The classification of the NBC and HBC at 200°C, 300°C, 400°C, 500°C, 600°C, 700°C, 800°C, 1000°C and 1200°C is shown in Table 4.3. The classification is based on the British Soil Classification System (BS 5930, 1990) summarized in Table 2.2. It was done in terms of particle size distribution obtained from Figure 4.1 and from the grading type as shown in Table 4.1.

Table 4.3: Variation of Classification with Temperature

Soil Type	Description	Symbol
NBC	Sandy SILT of high plasticity	MHS
HBC at 200°C	Sandy CLAY of high plasticity	CHS
HBC at 300°C	Sandy CLAY of high plasticity	CHS
HBC at 400°C	Sandy CLAY of high plasticity	CHS
HBC at 500°C	Very clayey SAND of intermediate plasticity	SCI
HBC at 600°C	Very clayey SAND of intermediate plasticity	SCI
HBC at 700°C	Gap graded SAND	SPg
HBC at 800°C	Gap graded SAND	SPg
HBC at 1000°C	Well graded SAND	SW
HBC at 1200°C	Gap graded SAND	SPg

From Table 2.2, the symbol for grading type is only included for coarse soils. For fine soils with at least 35% passing the 63 micron sieve, the classification was carried out considering the plasticity chart shown in Figure 2.15 (BS 5930, 1990). The NBC soil had 87% fine material (35% clay and 52% silt) and 13% coarse material (11.5% sand and

1.5% gravel) from Figure 4.1; from Figure 4.2, I_p was 30% while W_L was 65%; It is located below the A-line in Figure 2.15, thus an M soil. More than 50% of the coarse material is sand. It is therefore a sandy SILT of high plasticity with symbol MHS. The other soils were similarly classified and results summarized in Table 4.3. Fill materials should be well graded and either non-plastic or of low plasticity. From the classification shown on Table 4.3, none of the soils met this requirement. However, high plasticity levels are undesirable. Therefore on the basis of classification, HBC at temperatures of 700°C, 800°C, 1000°C and 1200°C would be suitable as fill material. But it has already been established from section 4.2.1.2 that HBC at 800°C and above cannot bond. Thus only HBC at 700°C can be used as a fill material.

4.2.1.6 Specific Gravity

The specific gravities for the NBC and HBC at 200°C, 300°C, 400°C, 500°C, 600°C and 700°C were as shown in Figure 4.3, from the data shown in Appendix C.

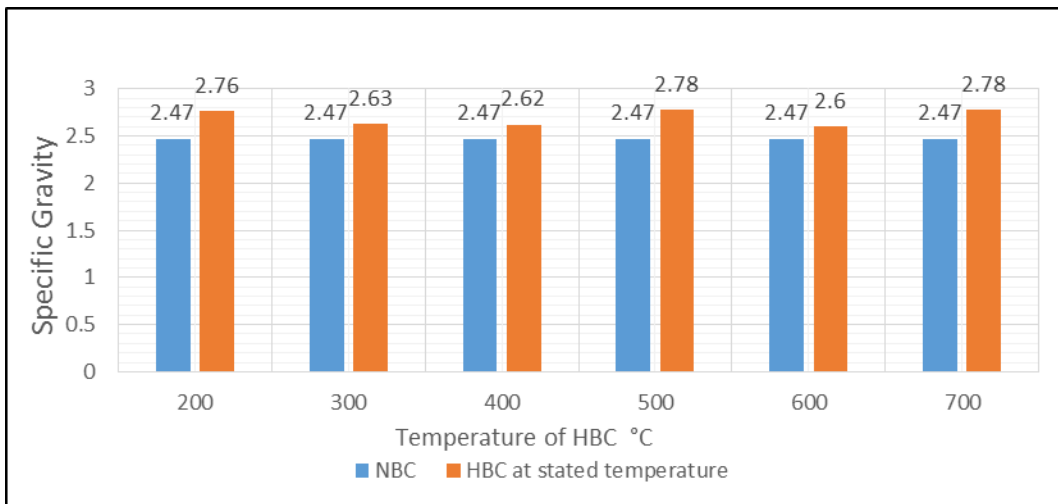


Figure 4.3: Variation of Specific Gravity with Temperature

The specific gravity for the NBC (taken at a room temperature of 25 °C on average) was 2.47 and increased by 11.2, 10.6, 10.6, 11.3, 10.5 and 11.3% at temperatures of 200°C, 300°C, 400°C, 500°C, 600°C and 700°C respectively. The changes in the specific gravity

by heating are within 10.5-11.3%. The increase from NBC to the HBC was probably due to loss of free and double layer water, together with combustion of organic matter for temperatures of 200⁰C and 300 °C causing the particles to pack closer together since the repulsive forces are no longer there; at higher temperatures, the breakdown of the crystalline structure, loss of chemically bound water and formation of granules formed denser material (Kornelius & Akinshipe, 2017; Li et al., 2014; Bell, 1993). Materials of high density are associated with increased strength (Mitchell & Soga, 2005) and are therefore better fill materials. Therefore in terms of specific gravity the heating gave a better fill material.

4.2.2 Mechanical Properties

The mechanical properties evaluated were compaction, the California Bearing Ratio (CBR) and the unconfined compression strength (UCS). The experimental work, data collection and analysis were done as discussed in section 3.2.4.

4.2.2.1 Compaction Characteristics

The compaction characteristics were obtained as detailed in section 3.2.4.1. The results are shown in Figures 4.4 - 4.10 for NBC and HBC soil, the latter at temperatures of 200⁰C, 300⁰C, 400⁰C, 500⁰C, 600⁰C and 700⁰C respectively from which the MDD, OMC and air voids can be read off.

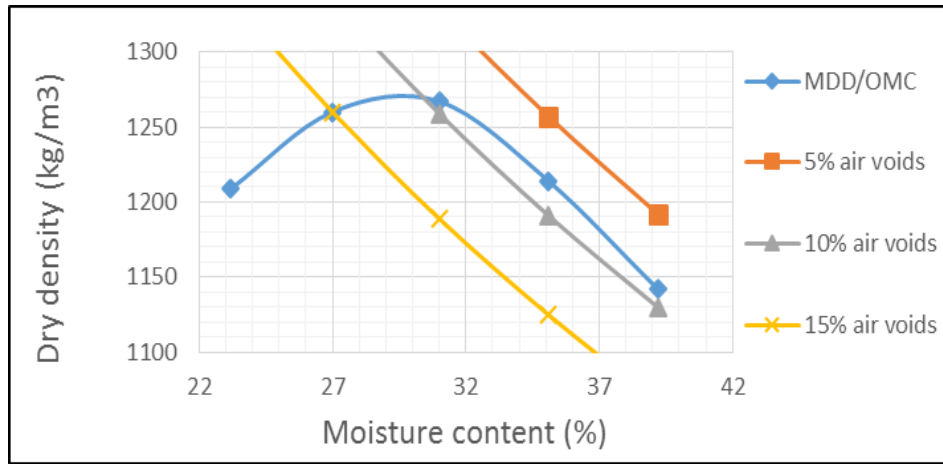


Figure 4.4: Compaction results for NBC.

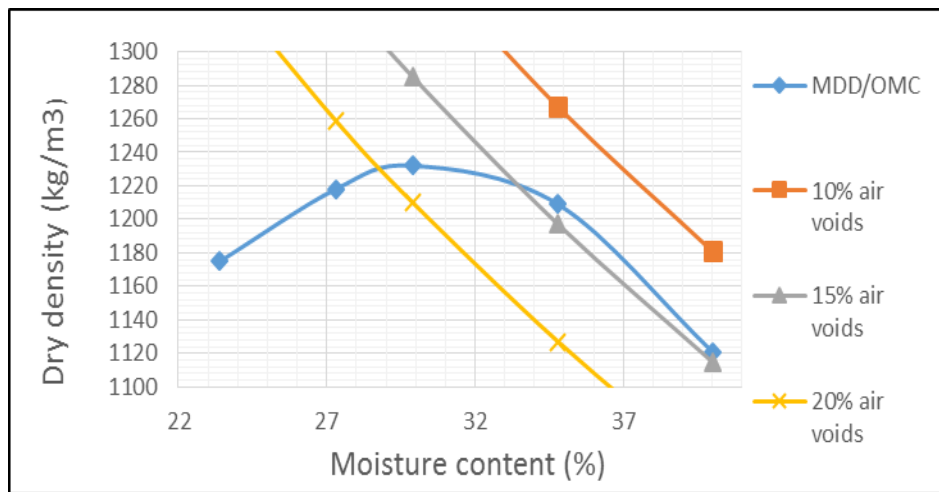


Figure 4.5: Compaction results for HBC at 200°

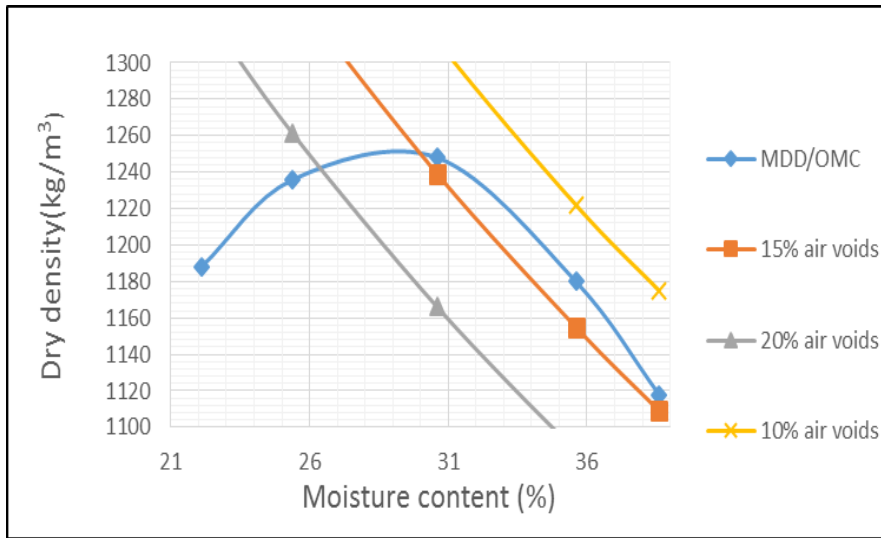


Figure 4.6: Compaction results for HBC at 300°C

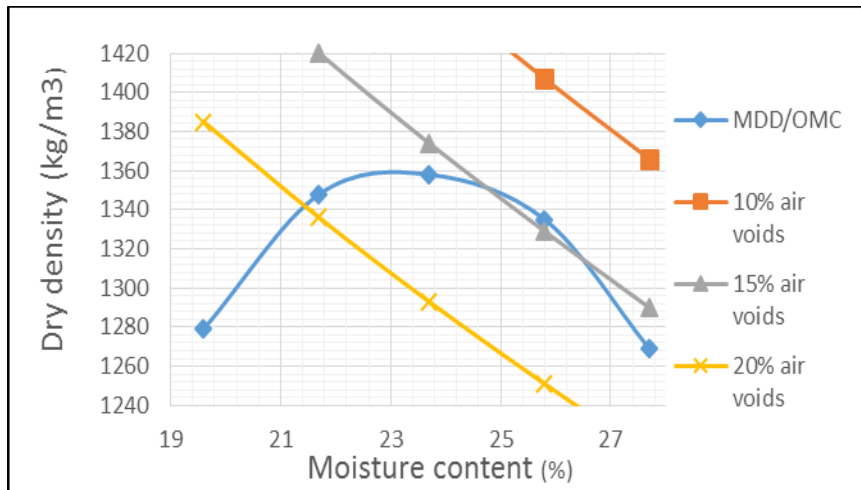


Figure 4.7: Compaction results for HBC at 400°C

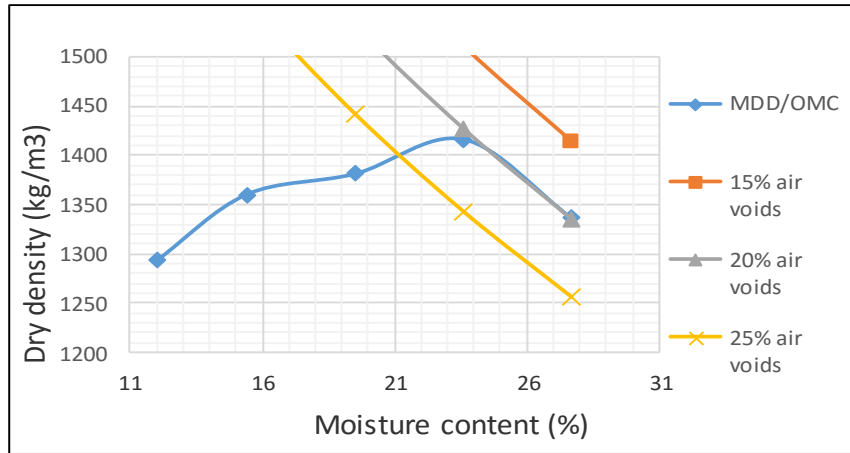


Figure 4.8: Compaction results for HBC at 500°C

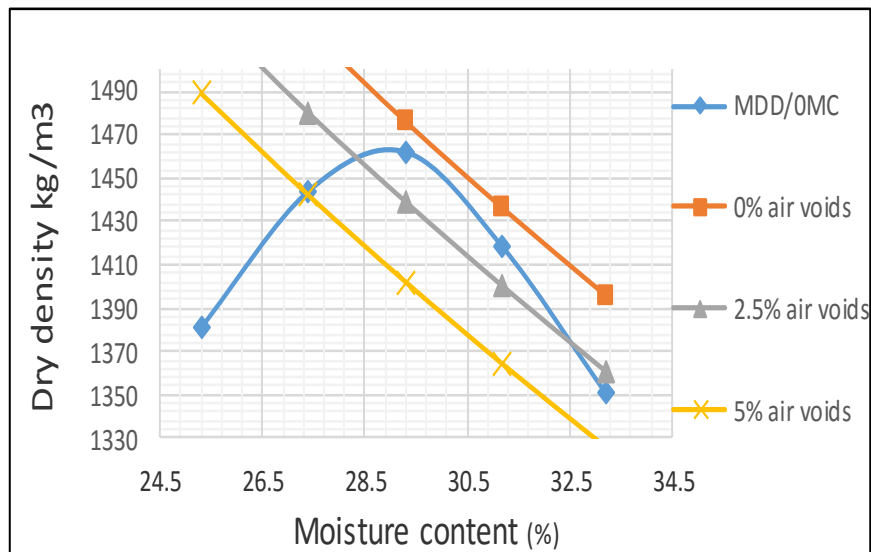


Figure 4.9: Compaction results for HBC at 600°C

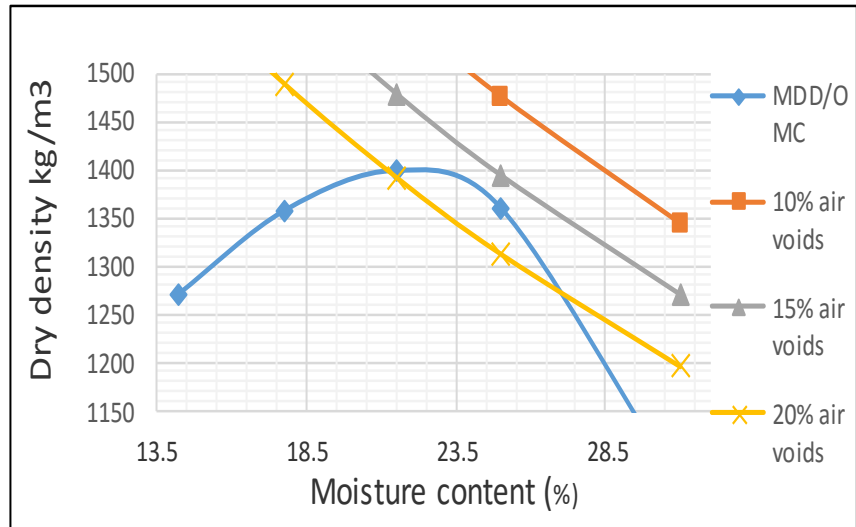


Figure 4.10: Compaction results for HBC at 700°C

The compactions results graphs show the variation of dry density versus the moisture content with air void curves imposed. No results could be obtained for HBC at temperatures of 800°C, 1000°C and 1200°C since at these temperatures soil was found to fuse and form hard clay clods which are cohesion-less. This also agrees with BS 1377: part 4, 1990 which states that it is difficult to define an optimum moisture content for a cohesion-less soil. As shown in Figure 4.4 for the neat black cotton (NBC) soil, the maximum dry density (MDD) was obtained as 1272 Kg/m³ with a corresponding optimum moisture content (OMC) of 29.5% at an air voids (AV) content of 11%. The values for the HBC soils were read off from Figures 4.5 to 4.10. From these, Figures 4.11-4.13 were done, which illustrate variation of MDD, OMC and air voids with temperature respectively. They compare these parameters for HBC at each temperature with those of NBC.

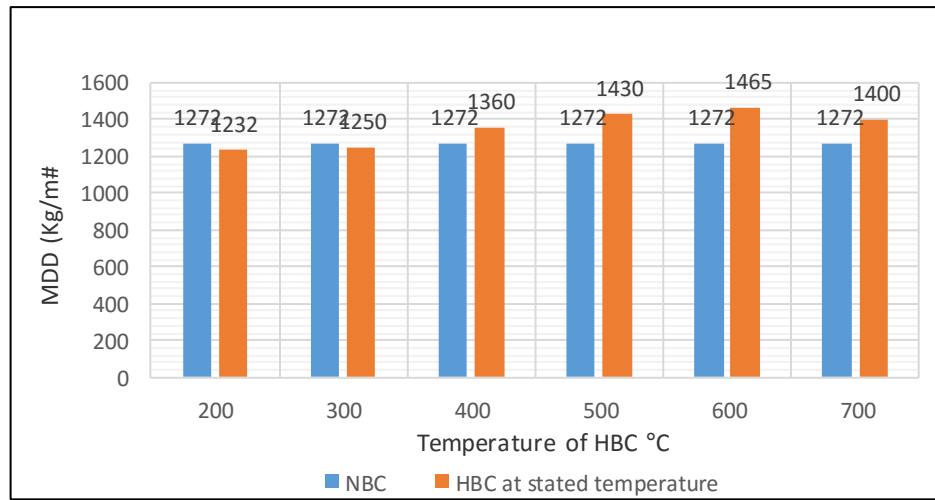


Figure 4.11: MDD versus Temperature of Soil

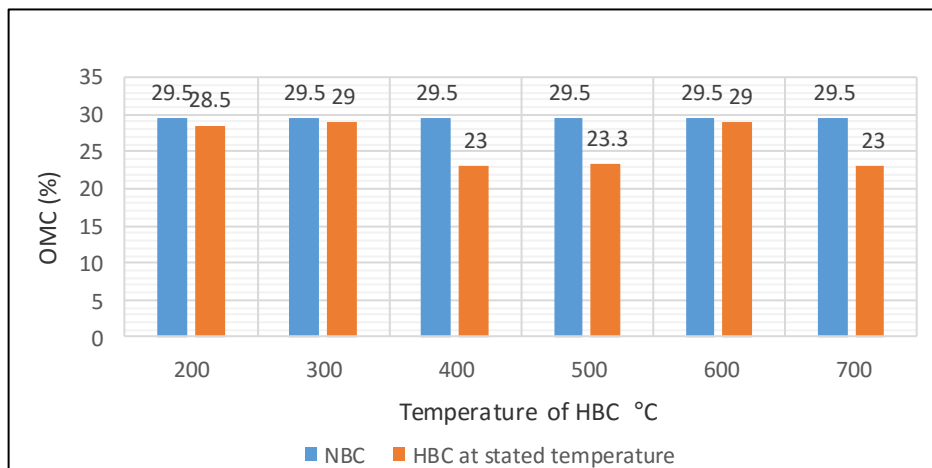


Figure 4.12: OMC versus Temperature of Soil

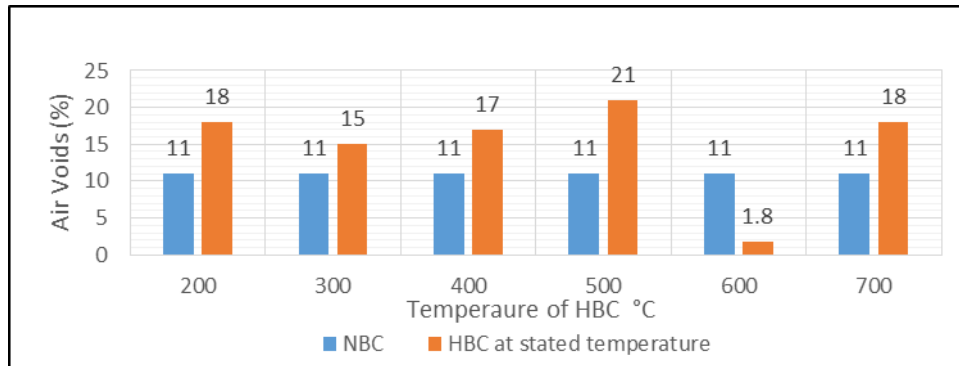


Figure 4.13: Air Voids versus Temperature of Soil

The MDD decreased from 1272 Kg/m³ for the NBC to 1225 for HBC at about 250°C, after which it increased steadily to a maximum of 1465 Kg/m³ for HBC at 600°C and then dropped to 1400 Kg/m³ for HBC at 700°C. The swelling test results shown in Figure 4.2 established that the swelling properties of heated black soil increased upon heating up-to 250°C, explaining the decrease in MDD for HBC up-to 250°C since the increase in swelling properties made the soil lighter. As shown in Figure 4.13 the air voids increased in this same temperature range, possible due to loss of interlayer water in the mineral structure. The increase of MDD from that of the HBC at 250°C to that of the HBC at 600°C may be attributed to formation of granules (Bell, 1993) which seemed to make the soil bond better. Further, the progressive breakdown of the crystalline structure due to heating meant that less and less water was retained in the soil structure due to reduction of the spaces between the layers. The very low air voids of 1.8% content for HBC at 600°C translated into a very high MDD since more space is now occupied by water which is denser than air. This air voids content of 1.8% is very close to the saturation level of 0% air voids. High air voids mean that the soil can absorb more water, which will soften the soil. Thus, low air voids are desirable. On the basis of the air voids, the HBC at 600°C would perform better even when exposed to water. The air voids of the rest heated soils vary from 15% for HBC at 300°C to 21% for HBC at 500°C. therefore

on the basis of air voids, HBC at 500 °C would perform relatively poorer than the others when exposed to water.

The high values of MDD for HBC at 600°C and 700°C can be taken to reflect high values of shear strength and a reduction in compressibility because the shear strength of a soil increases with increasing dry density, while the compressibility decreases with a higher degree of compaction (Knappet & Craig, 2012; O’Flaherty, 2002). The drop in MDD for the HBC at 700°C may be attributed to conversion of clay clods to aggregates with reduced bonding properties. The weight loss for HBC at 200°C and 600°C -700°C, may be attributed to combustion of organic matter and the loss of interlayer water (Figures 2.1 and 2.4) comprising the double layer water (Figure 2.6) and free water at lower temperatures and the loss at higher temperature to loss of the hydroxyls (Figure 2.4). The OMC is about the same for the NBC and the HBC at temperatures of 200°C, 300°C and 600°C, while it reduced significantly for the HBC at 400°C, 500°C and 700°C. However, MDD and OMC are not soil properties of the soil but depend on the compactive effort. A higher compactive effort will give a higher MDD with a lower OMC, as a result of orientation of soil particles, as illustrated in Figure 2.15 (Knappet and Craig, 2012; Murthy, 2007; Smith & Smith, 1998; Terzaghi et al., 1996).

To compare MDD and OMC for soils, the compactive effort is maintained. According to O’Flaherty (2002), the moisture-density test is designed to assist in field compaction of earthworks. The shear strength of a soil is assumed to increase with increasing dry density and the compactive effort used in the laboratory test assumed similar to those achieved in the field if certain equipment are used. Therefore for fine grained silts and clay, the field compaction normally specified is 90-95% of MDD obtained from the standard proctor value at the OMC.

4.3.1.2 California Bearing Ratio (CBR) and CBR Swell.

The CBR and CBR swell were determined as detailed in section 3.2.4.2. for the NBC and HBC at different temperatures. The results are illustrated in Figure 4.14 from the data in Appendix D.

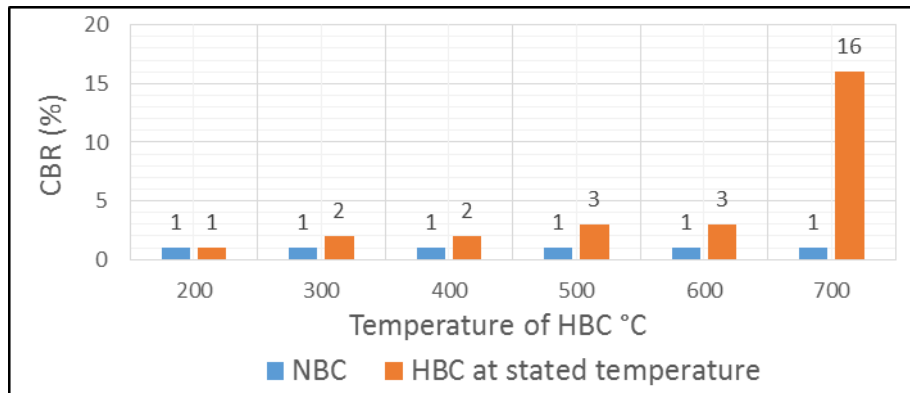


Figure 4.14: CBR versus Temperature of HBC

As shown the CBR value increased with heating, but remained very low up-to 600°C but shot up to 16 for HBC at 700°C. Above this temperature, the soil collapsed and no CBR could be determined. As noted in section 2.10, according to MOTC (1987) materials with values of CBR less than 2 indicate low strength materials, with black cotton being in this category and cannot be used as fill material. Results from other researchers indicate low CBR values for CBR, as indicted in Table 2.3. Thus the NBC is an expansive clay soil on basis of CBR. CBR was 3 for HBC at 600°C and thus lies in class S1 subgrade materials with CBR in the range 2-5 and can be used as fill material for roads in the absence of better materials, provided an improved subgrade of minimum thickness is provided depending on the quality of the improved subgrade and desired new class of subgrade. However, ideally fill material should have CBR exceeding 5, so as to have subgrade in class S2, with CBR in range 5-10. Thus HBC at 700°C with CBR of 16 meets this requirement adequately, and is in class S4, with CBR in range 10-18. Heating makes clays to lose adsorbed water and harden, together with an increase in strength. As the

temperature was increased, the crystalline structure underwent irreversible changes, which caused strength to increase. This explains the high strength at 700°C. Beyond 700°C, the clay lost its cohesive properties and could not bond explaining why no CBR strength could be obtained. The results for CBR swell are shown in Figure 4.15, from data in Appendix D.

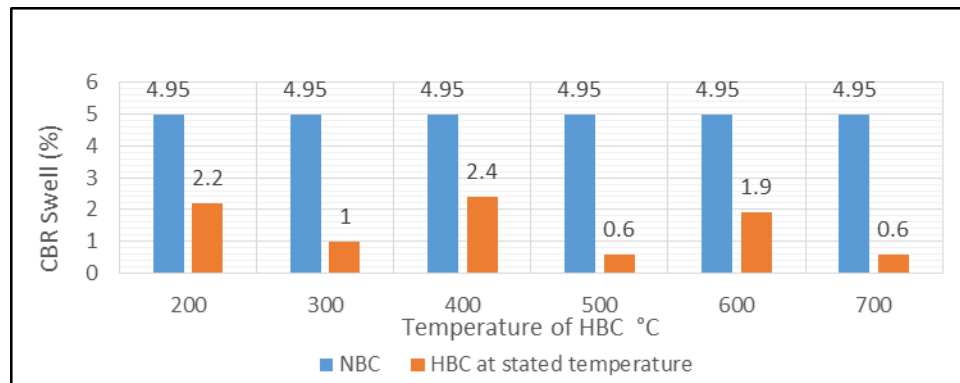


Figure 4.15: CBR Swell versus Temperature of HBC

As shown in Figure 4.15 the CBR swell reduced from a high value of 4.95% for the NBC to 2.2, 1.1, 2.4, 0.6, 1.9 and 0.6 for the HBC at temperatures of 200°C, 300°C, 400°C, 500°C, 600°C and 700°C respectively. The potential to swell reduced due to combustion of organic matter and de-hydroxylation, the soil progressively behaved like a cohesionless soil. MOTC (1987) classifies materials with CBR swell above 3% as unsuitable for fill, but the swell must be less than 2% for use as subgrade material. On the basis of swell, materials heated to at least 500°C are suitable as subgrade material. Therefore HBC at 600°C can be used as fill material in the absence of better materials. HBC at 700°C is the most suitable material because it meets requirements for CBR and swell, for use as fill without need for improved subgrade. The increase in CBR and the reduction in swelling are probably as a result of the irreversible breaking of the crystalline structure which causes clays to exhibit extremely high water absorption, swelling and shrinkage characteristics (Smith & Smith, 1998). Breakdown of the expansive characteristics thus

caused increase in strength and reduction in swell since the expansive state makes the soils to exhibit excessive compressibility, collapse, low shear strength, low bearing capacity and high swell potential, according to Rao and Thyagara (2007). The strength increase and reduction in CBR swell made the soil suitable as a fill material

4.2.2.3: Unconfined Compressive Strength (UCS)

The Unconfined Compressive Strength was done as detailed in section 3.2.4.3 by axially loading 50x100 cylindrical on a compression machine to failure as shown on Plate 3.7. The UCS value was taken as half 50% of the stress at the failure, as indicated in Equation 3.10. The typical failure pattern of the soil samples is shown in Plate 4.1 whereas plate 4.2 shows typical tested specimens; Figure 4.16 shows the UCS values versus temperature from the data in Appendix D.



Plate 4.1: Typical Failure Pattern.



Plate 4.2: Typical Tested Specimens

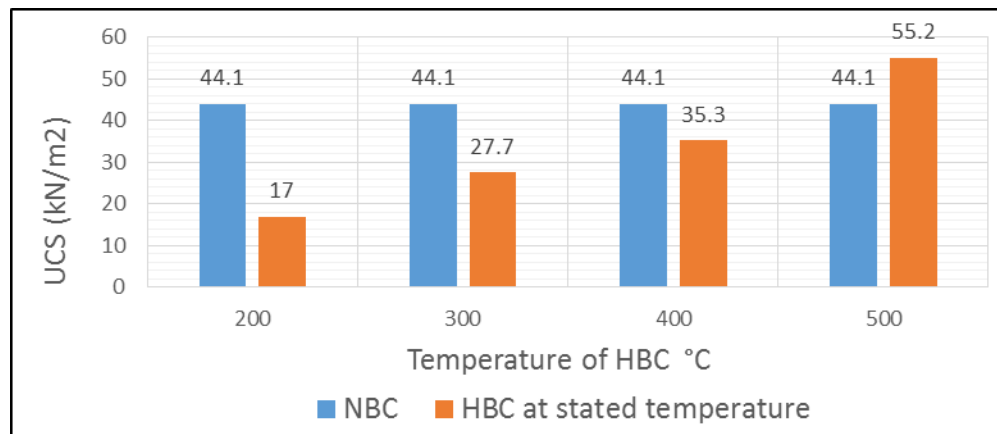


Figure 4.16: UCS values versus temperature

From the results shown in 4.16, the UCS value for the heated soil were all lower than for the neat soil, except for HBC at 500°C, which was 25% higher than for NBC. As the temperature increased, the soil lost interlayer water and organic matter, the latter due to combustion. This caused the soil to lose bond thus showing very low results at 200°C. However the UCS increased steadily from HBC at 200°C to HBC at 300°C by 63%, by

27% for HBC at 300°C to HBC at 400°C and by 56% for HBC at 400°C to HBC at 500°C. The strength increase could be attributed to increase in inter-particle bond as the water reduced. No result could be obtained for HBC at 600°C and higher temperatures. Since there is no side confinement during UCS tests, the lack of cohesion at these temperatures caused the specimens to collapse. The CBR results shown in Figure 4.14 show very low strengths for these temperatures, reflecting some consistency in terms of order of magnitude. Thus, soils in these temperature ranges cannot be used as fill material.

4.2.3 Electro-chemical and Chemical Properties

The chemical and electro-chemical results were obtained for XRD, the XRF, the pH, Loss on Ignition, sulphates, chlorides and the organic matter content. These are discussed hereunder.

4.2.3.1 X-Ray Diffraction.

The XRD test was done as detailed in section 3.2.5.1. The X-ray diffraction diagrams for the NBC and HBC at temperatures of 200°C, 300°C, 400°C, 500°C, 600°C, 700°C and 800°C were obtained graphically as a plot of the intensity of diffracted beam versus the phase angle, 2θ for each specimen, as shown in the Appendix E (Figures E1 to E8). Each angle has an associated basal (d) spacing in compliance with Bragg's Law (Equation 2.1). For each of the minerals, there were mainly 3 broad peaks for 2θ between 5° and 69°, first, second and third order reflections corresponding to n values of 1, 2 and 3 respectively in Equation 2.1. Each figure also shows the minerals present and their percentages, as obtained from the inbuilt software connected to the International Centre of Diffraction Data (ICDD). The number of minerals varied from 5 to 7.

(a) Minerals in the soil

The percentage of each mineral present in the NBC and HBC at 200°C, 300°C, 400°C, 500°C, 600°C 700°C and 800°C were as shown in Table 4.4, as obtained from the data in Appendix E. As discussed in section 2.4, the type and percentage of clay minerals contribute to the swelling characteristics of expansive clays, in a proportion far greater than their percentage by weight (Knappet & Craig, 2012; Das, 2008). A high percentage of non-expansive minerals such as quartz, carbonates, kaolinite and mica-like minerals will reduce the swelling characteristics (Kemp et al., 2005). The clay minerals and their percentages were extracted from Table 4.4 and tabulated in Table 4.5, to enable predict the expansive behaviour of the soil.

Table 4.4: Minerals in the Soil.

Reference Mineral	Soil Sample Type (HBC in °C)							
	NBC	HBC at 200	HBC at 300	HBC at 400	HBC at 500	HBC at 600	HBC at 700	HBC at 800
Quartz	21.8	24.2	22.5	27.0	21.2	18	23.2	26.7
Albite	26.2	31.2	30.1	31.4	28.3	35.3	26.3	25.5
Microcline	38.5	30.2	31.7	34.3	33.9	39.1	36.6	30.8
Kaolinite	1.7	0	0	2.5	3.2	2.5	0	2.0
Montom- rillonite	3.3	5.8	7.1	4.8	0	0	0	0
Muscovite	2.7	5.3	0	4.2	0	5.1	5.0	5.6
Calcite	5.8	5.3	4.5	0	4.2	0	0	9.5
Imogolite	0	3.3	4.0	0	0	0	2.5	0
Tremolite	0	0	0	0	3.4	0	6.4	0

(b) Clay Minerals

The clay minerals are given in Table 4.5 as a percentage of all the minerals. For each sample, the main clay minerals were albite and microcline. They both made up for more than 60% of the minerals in all the specimens, except for HBC at 800°C where the percentage was 56.3%. As discussed in section 2.4, the swelling potential in Albite is mobilized if other factors influencing on the swelling such as presence of water and montomorillonite are fulfilled (Skippervik, 2014).

Table 4.5: Clay Minerals composition as a percentage of the all the minerals

Reference Mineral	Soil Sample Type (HBC in °C)							
	NBC	HBC at 200	HBC at 300	HBC at 400	HBC at 500	HBC at 600	HBC at 700	HBC at 800
Albite	26.2	31.2	30.1	31.4	28.3	35.3	26.3	25.5
Microcline	38.5	30.2	31.7	34.3	33.9	39.1	36.6	30.8
Kaolinite	1.7	-	-	2.5	3.2	2.5	-	2.0
Montomo- rillonite	3.3	5.8	7.1	4.8	-	-	-	-
Muscovite	2.7	-	-	4.2	4.2	5.1	5.0	5.6
Calcite	5.8	5.3	4.5	-	5.8	-	-	9.5
Imogolite	-	3.3	4.0	-	-	-	2.5	-

In the soil under study, the presence of montomorillonite and water mobilized the swell potential. For the neat soil (NBC), montomorillonite, which is responsible for expansive behaviour of the soil was 3.3% by weight of the whole sample. This confirms that the neat soil is a mixed clay. In this research the small percentage of montomorillonite exerted a very great influence on the soils, due to the significant presence of the albite. The montomorillonite increased slightly to 5.8, 7.1 and 4.8% for HBC at 200°C, 300°C and

400°C respectively. This was probably due to loss of water and combustible matter for 200°C and 300°C; but as the crystalline structure started breaking down, montmorillonite started reducing from 400°C, and at 500°C and above montmorillonite could not be traced. explaining why the swelling characteristics decreased at these temperatures.

Since the NBC was expansive, the presence of non-expansive quartz, microcline, kaolinite, muscovite, calcite and imogolite exerted little influence on the soil. The percentage of kaolinite whose structure is shown in Figure 2.2 and remained below 3.2% in the heated soil. Muscovite which is a micalike clay mineral has very strong potassium interlayer bonding. It was 2.7% for NBC, 0% for HBC at 200°C and 300°C, remaining between 4.2 and 5.6% for higher temperatures. Calcite which is calcium carbonate was 5.8% for NBC, reduced slightly to 5.3 and 4.5% for HBC at 200 and 300°C respectively, diminished for HBC at 400°C, 600°C and 700°C. It was 5.8 and 9.5% for HBC at 500 and 800°C. Imogolite levels were minor, being 0% for NBC, HBC at 400°C, 500°C, 600°C and 800°C and low levels of 3.3, 4, and 2.5% for HBC at 200°C, 300°C, and 700°C respectively. Imogolites are formed from double bands of silica tetrahedral, have fine thread-like morphologies and are stable (Mitchell & Soga, 2005).

The most significant result is the diminishing of montmorillonite in HBC at 500 °C and higher temperatures as a result of the breakdown of its structure due to heat, which made the soil non-swelling.

4.2.3.2 Chemical Properties.

Table 4.6 shows the results of the X-Ray fluorescence (XRF), the sulphates, the chlorides, the pH, the loss on ignition (LOI) and the organic matter content (OM) contents for the NBC and HBC at temperatures of 200°C, 300°C, 400°C, 500°C, 600°C, 700°C, 800°C, 1000°C, and 1200°C from the data given in Appendix M

Table 4.6: Chemical Test Results

Reference	Neat Soil	Heat Soil at Temperatures Stated in °C								
		200	300	400	500	600	700	800	1000	1200
SiO ₂ (%)	58.2	38.8	39.1	39.3	39.0	40.0	38.5	40.4	40.9	41.2
Al ₂ O ₃ (%)	15.7	22.2	25.3	26.1	25.8	24.6	25.2	25.2	26.1	25.5
Fe ₂ O ₃ (%)	15.74	22.35	20.1	19.3	19.9	19.6	20.0	18.2	17.4	17.6
K ₂ O (%)	3.09	8.80	8.98	8.95	8.73	9.14	9.09	8.91	9.13	9.27
MnO (%)	1.78	3.82	2.75	2.61	2.86	3.21	3.32	3.20	2.89	3.04
CaO (%)	2.96	2.17	2.04	1.98	2.07	2.05	2.35	2.22	2.05	1.86
TiO ₂ (%)	1.67	1.22	1.18	1.16	1.13	0.98	1.05	0.99	1.04	1.10
SO ₃ (%)	6.6	0.30	0.66	3.9	1.7	2.8	8.1	-	-	-
-x10 ²										
SO ₄ (%)	7.92	0.36	0.72	4.68	2.04	3.36	9.72	-	-	-
- x10 ²										
Cl (%)	0.01	0.044	0.043	0.011	0.032	0.011	0.034	0.019	0.014	0.011
pH	7.42	7.19	7.2	7.35	7.23	7.17	7.0	7.24	9.34	9.77
LOI (%)	12.8	11.8	10.2	10.1	9.2	7.22	6.45	5.18	-	-
OM (%)	0.72	3.24	3.31	1.11	2.18	0.8	1.62	0.64	0.40	0.40

Key:

SiO₂- Silicon (IV) Oxide; Al₂O₃ - Aluminum (III) Oxide; Fe₂O₃ - Iron (III) Oxide; K₂O - Potassium Oxide; MnO - Manganese (II) Oxide; CaO - Calcium Oxide; TiO₂ - Titanium (IV) Oxide; SO₃ - Sulphate (III) Oxide; SO₄ - Sulphates (IV) Oxide; Cl - Chloride Content; LOI - Loss of Ignition; OM - Organic Matter Content

The XRF test was conducted to identify the main oxides in the soil. The oxides in a clay can be used to indicate the clay minerals. The main oxides in the NBC and the HBC were silicon (IV) oxide (SiO₂), aluminum (III) oxide (Al₂O₃) and iron (III) oxide (Fe₂O₃). For

expansive clays, Newman (1987) indicated that the main oxides in a soil containing montmorillonite are SiO_2 , Al_2O_3 and Fe_2O_3 or magnesium oxide (MgO). Bennett and Need (1969) gave the range of the main oxides in a soil containing montmorillonite as SiO_2 (48-56%), Al_2O_3 (11-22%), Fe_2O_3 (>5), and CaO (4-9%). These ranges are fairly consistent with the NBC except for SiO_2 which was 58.2%, and CaO which was 2.96%. In addition to the XRD results discussed in section 4.2.3.1, this further confirms that the test soil contained montmorillonite.

Heating caused a reduction in SiO_2 from about 58% for the NBC to about 39 - 41% for HBC at temperatures between 200°C and 1200°C ; an increase in Al_2O_3 from about 16% to 22% for HBC at 200°C , and to about 25-26% for temperatures higher than 200°C ; Fe_2O_3 increased from about 16% to 22% for HBC at 200°C and to about 18-20% for higher temperatures. These chemical changes are irreversible, since the tests were conducted when the soil had cooled to room temperature. Some of the increases in the oxides may be attributed to loss of inter-layer water (dehydration) at lower temperatures, and hydroxylation as the temperature increased beyond 400°C including the combustion of organic matter. The sum of SiO_2 , Al_2O_3 and Fe_2O_3 exceeded 80% for the NBC and HBC soils meaning that the soil is pozzolanic at all temperatures. According to ASTM C618-15 (2015), the sum of SiO_2 , Al_2O_3 and Fe_2O_3 must be minimum sum of 70% for a soil to be pozzolanic. Clay minerals are natural pozzolons (Greaves, 1996; Sherwood, 1993) which did not change on heating. However, pozzolanic action can only be manifested in the presence of water and calcium hydroxide.

The significant changes in other oxides was mainly in potassium oxide (K_2O) which changed from about 3% for NBC to an average of 9% for the HBC at all temperatures. The organic matter (OM) content was below 3.5% at all temperatures, with the highest value being 3.3% for HBC at 300°C ; the lowest value was obtained as 0.4% for HBC at 1000°C and 1200°C . MOTC (1987) limits OM to a maximum of 5% which is well above the highest value obtained in the NBC and the HBC at all temperatures.

The highest sulphate content as SO_4 was 0.097% for HBC at 700°C . As discussed in section 2.4.2.1, high levels of SO_4 in clays causes the SO_4 to react with CaO for lime stabilized soils, to form ettringite $[\text{Ca}_6\text{Al}_2(\text{OH})_{12}(\text{SO}_4)_3 \cdot 27\text{H}_2\text{O}]$ or thaumasite $[\text{CaSiO}_3 \cdot \text{CaCO}_3 \cdot \text{CaSO}_4 \cdot 14.5\text{H}_2\text{O}]$ which results in expansion and heave of the stabilized soil, causing distress in the structures built on the soil, which may lead to failure (Kavak and Bilgen, 2016; Beetham et al., 2015; Bell, 2007; Mitchell and Soga, 2005; Rajasekaran and Rao, 2005). SO_4 level in lime stabilized clays should therefore not exceed 0.25% (Beetham et al., 2015). Thus, the NBC and HBC at all temperatures could be used in lime stabilization without any risk of formation of ettringite or thaumasite.

The highest chloride level was 0.044% for HBC at 200°C .; all the other levels were lower, for the NBC and HBC. The chloride level has no adverse effect because chlorides are used to stabilize expansive clays; Davoudi and Kabir (2011) used a mix of lime and sodium chloride to stabilize low plasticity fine grain soils, and established that 1 and 2% sodium chloride caused an increase in UCS of 28% and 18% respectively at 28days. There was no improvement at 3%, with a decrease of UCS at higher percentages. They also established that there is an increase in UCS for soil stabilized with lime in the range 2-8% and sodium chloride in the range 1-5% regardless of the percentage of lime, with the effect becoming negligible after 5% concentration.

The pH of the soil indicated a low alkaline level for the NBC and HBC at temperatures of 200°C , 300°C , 400°C , 500°C , 600°C , 700°C and 800°C , ranging from 7.0 to 7.42, the lowest level being 7.0 for HBC at 700°C . However, the pH increased significantly to 9.34 and 9.77 for HBC at 1000°C and 1200°C respectively. High levels of pH are desirable in lime stabilization; Davoudi and Kabir (2011) state that increase in pH caused by the presence of sodium hydroxide in lime-stabilized low plasticity soil mixed with sodium chloride caused increase in UCS. Kabir (2009) cited in Davoudi and Kabir (2011) stated that the solubility of silicates and aluminates which enhances the pozzolanic reaction in lime stabilized soils is a function of the pH of the media. In lime stabilization, sufficient excess lime is added to ensure that the stabilized mixture maintains a pH value of at least

12.4 which is necessary to ensure the long term stability of the hydration products (O’Flaherty, 2002; Dang et al., 2016).

There was a reduction in LOI, progressively decreasing from 12.8% for the neat soil, to the lowest value of 5.2% at 800⁰C. This was due to combustion of organic matter at the high temperatures.

The organic matter (OM) content was below 3.5% at all temperatures, with the highest value being 3.3% for HBC at 300⁰C; the lowest value was obtained as 0.4% for HBC at 1000⁰C and 1200⁰C. MOTC (1987) limits OM to a maximum of 5% for fill material, but it should be less than 3% for subgrades which is met by HBC at 400⁰C and above. Thus on the basis of the OM content, the NBC and HBC at all temperatures are suitable for use as fill materials, but only NBC and HBC temperatures of 400⁰C and above are suitable as subgrade material for results since their OM content is less than 3%.

4.2.3.3 Exchangeable Cation Tests Results

Table 4.7 shows the results of cation exchange capacity (CEC) and related tests which include the soil pH, the electrical conductivity, the exchangeable cations (calcium, magnesium, potassium and sodium) and the base saturation; a sample of the data is given in Appendix M.

Table 4.7: Exchangeable Cation Tests Results

Reference	Neat	200°C	300°C	400°C	500°C	600°C	700°C
Soil pH-H ₂ O (1:2.5)	7.6	7.5	7.5	7.7	7.4	7.7	7.4
Elect. Cond. mS/cm	0.4	0.5	0.8	0.8	0.5	0.5	0.5
C E C (meq/100)	36.6	40.6	36.5	32.1	24.6	23.9	17.3
Calcium (%)	64.7	53.5	55.3	57.6	29.6	35.6	14.1
Magnesium (%)	4.65	4.65	4.2	3.9	2.0	2.8	1.4
Potassium (%)	0.71	0.64	0.8	1.0	2.2	1.6	1.2
Sodium (%)	2.31	1.96	2.4	2.2	2.4	1.6	1.3
Sum (%)	72	60.6	63.3	64.9	36.2	41.6	18.0
% of Calcium	90	88	87	89	82	86	78
Base Saturation (%)	100+	100+	100+	100+	100+	100+	100+

The pH level is in the same range as in Table 4.6, the minor differences being the result of using a different laboratory. The electrical conductivity is a measure of the presence of salts. The cation exchange capacity (CEC) of the NBC was 36.6 meq/100. Table 2.1 gives the CEC values for the common minerals. Common expansive clays like montmorillonite have CEC values of 80-120 meq/100 which is way above the CEC for the NBC. The test soil is therefore not a pure clay but a mixed clay. Mix clays consist of one or more types of clay which influence their properties on the basis of relative proportions (See section 2.4.5).

The CEC increased slightly from the NBC value of 36.6 to 40.6 meq/100 for HBC at 200°C, followed by a steady decrease as the temperature decreased, reaching a minimum value of 17.3 meq/100 for HBC at 700°C. The steady reduction of CEC was caused by breakdown of the mineral structure as heating ensued, which increasingly made the soils better fill materials. The main exchangeable cation is calcium, with a percentage ranging

from a very high value of 90% for the NBC, decreasing slowly to a minimum of 78% for HBC at 700°C. The calcium cation level of 90% for the NBC means that 90% of the available positions around the particle are occupied by calcium cations. The swelling properties would be more severe if sodium were the adsorbed cations, since sodium is more easily adsorbed as indicated by the affinity series: $Al^{3+} > Ca^{2+} > Mg^{2+} > NH_4^+ > K^+ > H^+ > Na^+ > Li^+$ (See section 2.5).

Base saturation is the percentage of the CEC occupied by the basic cations which are Ca^{2+} , Mg^{2+} and K^+ and Na^+ . At pH level lower than 7, the base saturation is lower than CEC, while at pH of 7 or higher the base saturation is equal to CEC. Thus, the base saturation level of 100 indicates that the NBC and HBC at the tested temperatures were alkaline, which is also indicated by the measured pH values. High alkaline levels are good for lime stabilization.

4.2.4 Summary: Effect of Heat on Expansive Clay Soils

The properties of the NBC and the HBC at 700°C are summarized on Table 4.8, indicating a great improvement of the properties of the latter compared to the former, clearly illustrating that the study has obtained a material suitable as a fill material for road construction and buildings, as well as being a suitable subgrade material thus achieving objective one.

Table 4.8: Summary of Results of the NBC and HBC at 700°C

Property	NBC	HBC at 700°C	Requirements	Suitability
Gravel (%)	1.5	41	-	Suitable
Sand (%)	11.5	59	-	Suitable
Silt (%)	52	48	-	Suitable
Clay (%)	35	21	-	Suitable
W _L (%)	65	0	-	Suitable
W _P (%)	35	0	-	Suitable
I _p (%)	30	0	50 max (MOTC, 1987)	Suitable
SL (%)	15	0	-	Suitable
Free Swell (%)	74	9	50 max (Holtz & Gibbs, 1956)	Suitable
Swell Potential	High	Low	Low (Table 2.7)	Suitable
Activity	Normal	Inactive	Inactive- normal	Suitable
BSCS-C	MHS	SPg	-	Suitable
Sp gr	2.4	2.78	-	Suitable
MDD (kN/m ³)	1272	1400	-	Suitable
OMC (%)	29.5	23	-	Suitable
Air voids (%)	11	18	-	Suitable
CBR	1	16	5 min (MOTC, 1987)	Suitable
CBR Swell (%)	4.95	0.6	2 max (MOTC, 1987)	Suitable
UCS (kN/m ³)	44.1	-	-	Suitable
pH	7.42	7.0	-	Suitable
LOI (%)	12.8	6.45	-	Suitable
OM (%)	0.72	1.62	3 max (MOTC, 1987)	Suitable
CEC	36.6	17.3	-	Suitable

Key: SL - Shrinkage Limit; BSCS-C - BSCS classification; Sp gr – Specific gravity

4.3 Properties of black cotton soil stabilized with heated soil, lime and molasses.

The experimental results reported in this section are the properties of black cotton soil stabilized with different proportions of heated soil, lime and molasses, including the properties of lime and molasses, the stabilizing agents. The experimental work was done as described in section 3.3. The properties of heated soil are as was established in section 4.2. The properties of the stabilizing agent are reported first followed by strength of trial mixes in terms of UCS and then the physical, mechanical and chemical properties of the

established optimal mix. The properties for lime and molasses were investigated to check their compliance with appropriate standards.

4.3.1 Properties of Lime

The properties of lime are shown in Table 4.9, together with the requirements from appropriate standards and its suitability as a stabilizer. The properties are from the sample data given in Appendix M.

Table 4.9: Properties Lime

Test	Parameter	Requirement	Suitability
Calcium as CaO (% m/m)	56.6	50.0 min (MOTC, 1986)	Suitable
Iron as Fe ₂ O ₃ (% m/m)	0.4	None	Suitable
Aluminium as Al ₂ O ₃ (% m/m)	1.8	None	Suitable
Magnesium as MgO (% m/m)	0.65	4.5% max (KS EAS 73:2000)	Suitable
Bulk Density (Kg/m ³)	540	None	Suitable
Fineness-Residue on 0.2mm sieve	0.56	1% max (MOTC, 1986)	Suitable
Fineness-Residue on 0.075 mm sieve	7.7	10% max (MOTC, 1986)	Suitable
Loss on Ignition (%) - at 1000 °C	38.1	None	Suitable

From Table 4.9, the lime met all the requirements for calcium, magnesium and fineness. There are no limits given for iron, aluminium and loss on ignition (LOI). Thus lime sourced from Athi River Manufacturing Co. Ltd of Nairobi met the requirements needed for lime stabilization and therefore used in this study.

4.3.2 Properties of Molasses

The test results for molasses are shown in Table 4.10 from the data in Appendix M, against the requirements, including the suitability of molasses. The brix, density, and the total ash values did not meet the required specifications for molasses from Chemili,

Muhoroni and Kibos whereas the all the other properties were satisfactory. Thus only molasses from Sony Sugar was found to be fully compliant, with the pH being marginally above 6. Therefore molasses sourced from Sony Sugar was used for stabilization.

Table 4.10: Properties of Molasses

Test	Factory				Requirements	Suitability
	Kibos	Sony Sugar	Muhoroni	Chemilil		
Brix (°Brix)	78.2	86.52	80.1	40.79	78 min (KS 953:2008)	Chemilil unsuitable
Calcium (% m/m)	1.02	1.01	0.14	0.72	2.1 max (KS 953:2008)	All suitable
Density (g/cm ³)	1.37	1.40	1.32	1.40	1.33 min (KS 953:2008)	All suitable
Magnesium as Mg (ppm)	314.24	0.05	262.13	182.61	500max (KS 63)	All suitable
pH	5.92	6.04	6.3	5.97	5-6 (KS 953:2008)	Muhoroni unsuitable
Phosphorus as P (% m/m)	0.003	0.003	0.003	0.001	0.5 max (KS 953:2008)	All suitable
Potassium as K (ppm)	2.54	1.77	0.61	1.28	5000 max (KS 63)	All suitable
Sulphated Ash (% m/m)	12.30	12.10	11.22	8.50	15 max (KS 58)	All suitable
Total Ash (% m/m)	10.31	7.83	9.39	4.02	10 max (KS 58)	Kibos unsuitable

4.3.3 Test Results for Trial Mixes

Sampled UCS results for the trial mixes are presented in Appendix H for the various mix ratios of heated black cotton soil (HBC) at temperatures of 200°C, 300°C, 400°C, 500°C, 600 °C and 700 °C. Mixes of 800°C and above could not bond, and were therefore not tested. The peak results were extracted; Figure 4.17 shows the typical combined UCS results curves for 400-HBC:lime:molasses mix ratio of 400-14:6:0-36%. 400-14:6:0-36% means that the heated soil was at a temperature of 400⁰ C, and the HBC: lime: molasses were in ratios of 14, 6 and 0% respectively, while the moisture content was 36%.

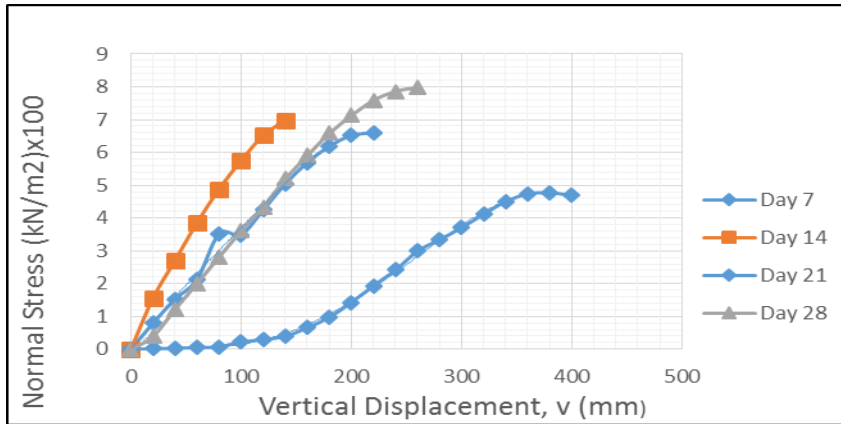


Figure 4.17: Combined curves for 400-14-6-0-36

The four curves shown in Figure 4.17 illustrate the results for curing periods of 7, 14, 21 and 28 days from which the UCS values were obtained as 50% of the stress at failure as shown in Equation 3.10. Similar curves were done for each mix ratio. The neat black cotton soil was maintained at 80% for all mixes, and thus the references were called out only in terms of the stabilizers' mixes and the compaction moisture content, all in percentages, for clarity. The 28 days cure peak results for the maximum values for each temperature are shown in Figures 4.18-4.30

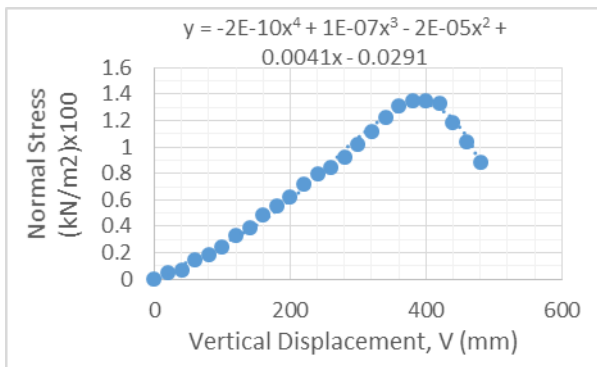


Figure 4.18: 200-14-6-0-28%

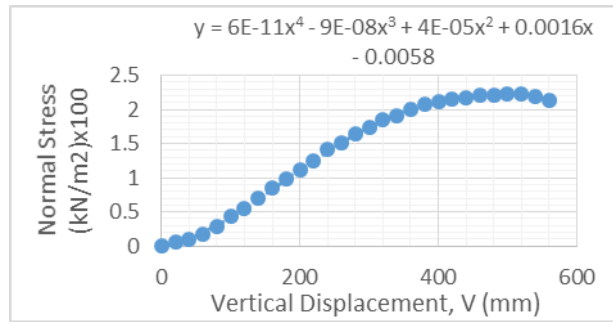


Figure 4.19: 200-8-8-4-28

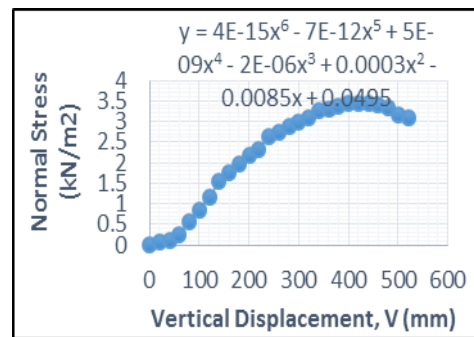
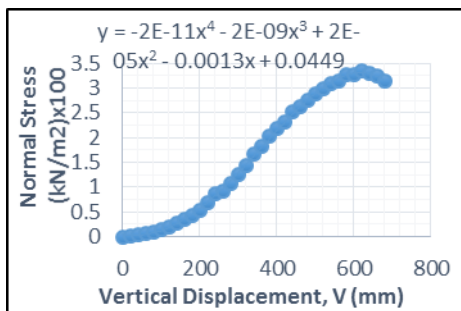


Figure 4.20: 300-10-0-10-32%

Figure 4.21: 300-5-0-15-32%

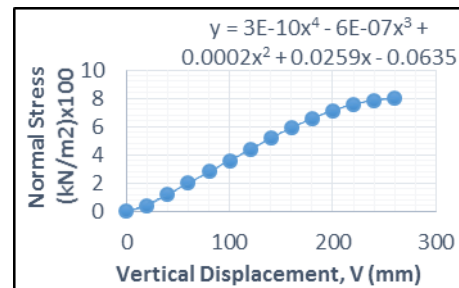
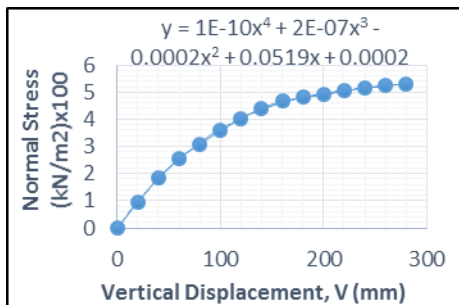


Figure 4.22: 400-16-4-0-36%

Figure 4.23: 400-14-6-0-36%

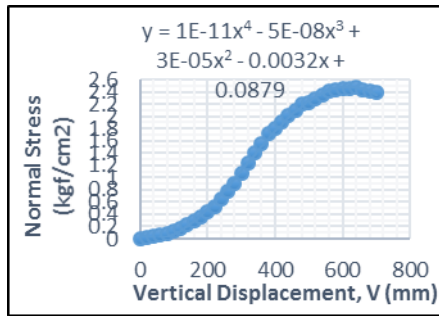


Figure 4.24: 500-10-0-10-36%

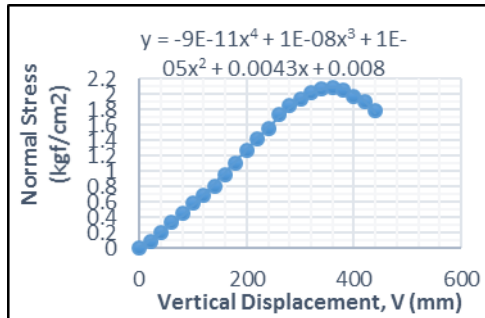


Figure 4.25:500-8-8-4-36%

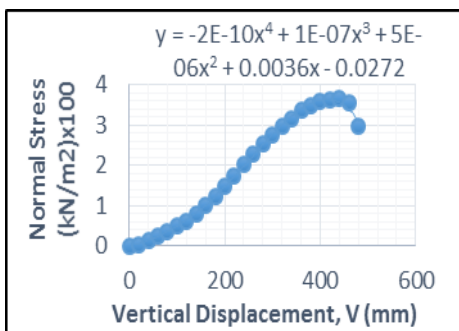


Figure 4.26: 600-16-4-0-36%

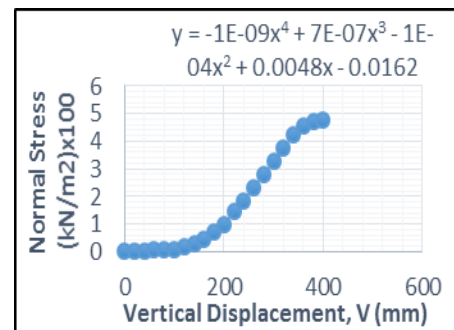


Figure 4.27: 600-14-6-0-36%

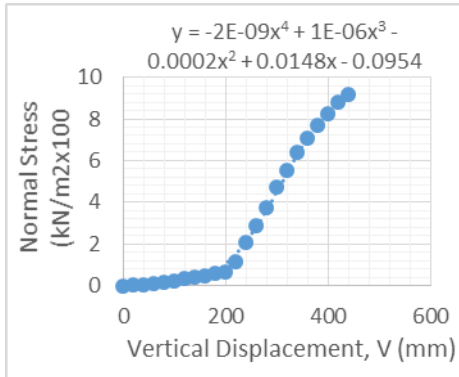


Figure 4.28: 600-12-8-0-36%

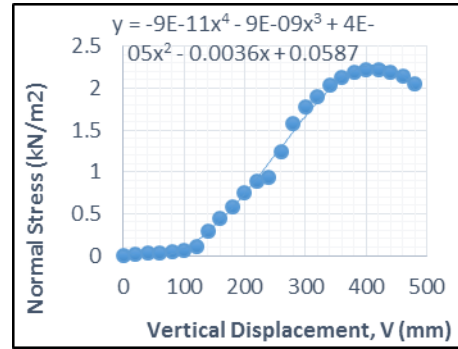


Figure 4.29: 700-10-6-4-36%

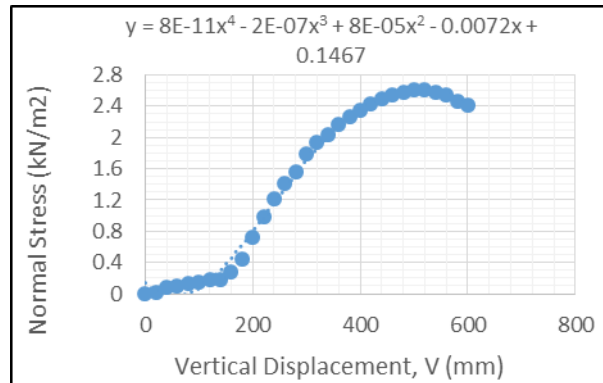


Figure 4.30: 700-10-0-10-36%

Peak values were obtained from mixes of 16-4-0, 14-6-0 and 12-8-0 for all the temperatures. Plots were therefore done for these mixes for each temperature, as shown in Figures 4.31-4.33, so as to select the highest strength for each temperature and then the optimal mix from these.

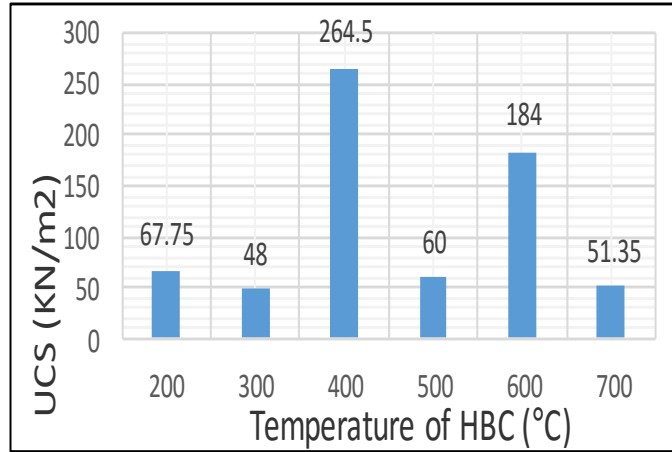


Figure 4.31: UCS for mix ratios of 16-4-0-36%

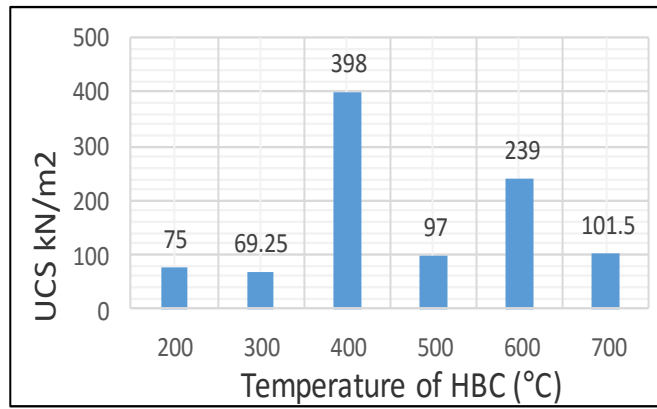


Figure 4.32: UCS for mix ratios of 14-6-0-36%

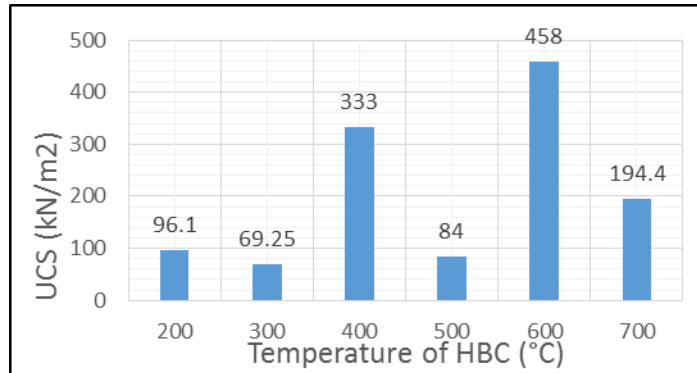


Figure 4.33: UCS for mix ratios of 12-8-0-36%

From Figures 4.31-4.33, the highest UCS values were obtained for the mixes with HBC at 400°C at 264.5, 398, 333 and 458 kN/m² for 400-16-4-0, 400-14-6-0, 400-12-8-0 and 600-12-8-0 respectively, all at 36% compaction moisture content. All the samples with molasses recorded relatively low values. The results show UCS peak values for HBC at 400°C and 600°C, and low values for HBC at 300°C, 500°C and 700 °C. This was very consistent for all the mix ratios. Though the highest strength was 458 kN/m² for 600-12-8-0, the temperature of HBC and lime content are fairly high. Therefore the optimal mix was selected as 400-14-6-0, with a strength of 398 kN/m² for a temperature of HBC at 400°C, and lime content of 6%. In general, strength increases in lime-stabilized soils up to a certain level, after which the strength decreases or remains constant with addition of lime (Bell, 1996; O’Flaherty, 2002). This was very well reflected in this study as shown in Figure 4.35, which shows an optimal lime content of 6.2% where lime was the only stabilizer, after which the strength decreased. High lime levels are also expensive. The lower temperature and lime content for the 400-14-6-0 mix means a relatively cheaper option as compared to the 600-12-8-0 mix. This mix was used in subsequent tests for long term curing and the various flooding regimes.

4.3.4 Variation of UCS with Moisture Content

From the mixing regimes giving peak values represented in Figures 4.31 to 4.33 at 28 days of curing, the variation of UCS with compaction moisture content was plotted as illustrated in Figure 4.34. There was a general increase in strength as the compaction moisture content increased, with a very sharp increase after moisture content of 32%. From the test results the strengths were found to be very low for low levels of moisture content, and increased for moisture content in the range 32 to 36%. The increase was sharp for 400-14-6-0 and 600-12-8-0, moderate for 400-16-4-0, 600-16-4-0, 600-14-6-0 and low for 500-8-8-4, 500-10-0-10 and 700-10-6-4. The sharp increase in strength for the first two sets of results indicate their high stability in high moisture conditions commonly experienced in areas prone to flooding, and are therefore suitable as fill materials in all

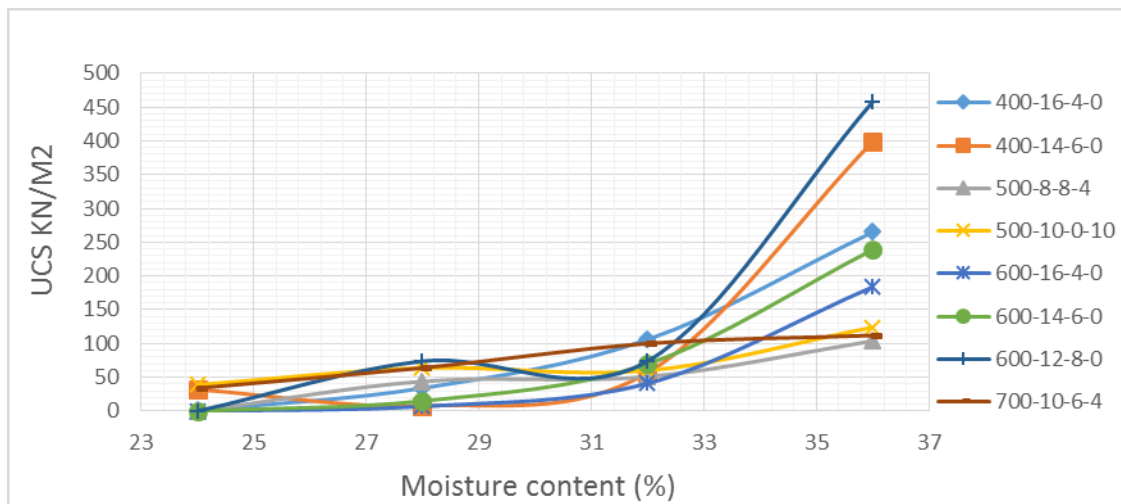


Figure 4.34: Variation of UCS with Moisture Content

areas because high moisture regimes represent the worst case scenario for lime stabilized expansive clays. The moderate increase for 400-16-4-0, 600-16-4-0 and 600-14-6-0 indicates moderate stability. The results of 500-8-8-4, 500-10-0-10 and 700-10-6-4 are for the mixes containing molasses; they are characterized by low strength at high

moisture content levels, since the salts formed are water soluble (O'Flaherty, 1974; M'Ndegwa, 2011), indicating they would be unsuitable as fill materials in flood prone areas.

The results generally indicated high water demand to satisfy hydration of lime. At low moisture contents, there is insufficient lime to allow for efficient compaction; further the little water available is used up rapidly in the hydration process meaning that peak UCS values are attained within a short period of time. Soil-lime reactions continue for long periods until all free lime is used, where there is adequate moisture for highly reactive clays before the pozzolanic reaction is completed (O'Flaherty, 2002). In this study, the strength of the heated soil:lime stabilized black cotton soil continued to increase as the compaction moisture content was increased, due to the enhanced pozzolanic action provided by the heated soil, which performed effective as a low calcium pozzolanic precursor. The molasses caused a dampening effect on the rate of the reaction due to the formation of soluble salts, rendering heated soil:lime:molasses mixture unsuitable as a stabilized material.

4.3.5 UCS for Neat Black Soil (NBC) stabilized with Lime (L)

Figure 4.35 shows the experimental results for UCS for NBC stabilized with lime only at various mixes, at 7 days soak and 7 days cure. The results give an optimum value of 360 kN/m² at a lime content of 6.2 %. Comparing this the UCS value of combined stabilization of NBC with HBC and lime, there was a strength improvement for specimens 400-14-6-0, 400-12-8-0 and 600-12-8-0 whose UCS values were 394, 333 and 458 kN/m² representing an increase of 12%, -8% and 29% respectively above the strength of the HBC treated with lime only. Therefore the optimum mix selected with a mix of 400-14-6-0 is stronger than the mix with lime only, and will therefore perform better as a fill material.

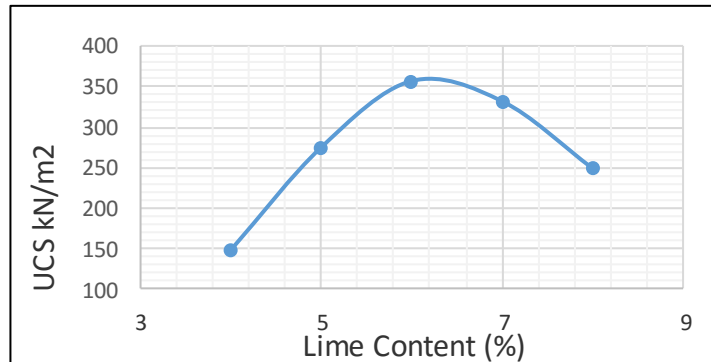


Figure 4.35: UCS for lime-stabilized NBC

4.3.6 Properties of the Optimal Mix

The optimal mix as established in section 4.3.3 is the stabilized soil comprising 80% NBC, 14% HBC at 400 °C, and 6% lime. As discussed in section 3.3.4.2, the properties tested for the optimal mix were physical properties comprising grading and swelling characteristics; mechanical test results comprising compaction, CBR and UCS and chemical and electro-chemical test results comprising XRD, cation exchange capacity, organic matter content, loss on ignition and pH.

4.3.6.1 Physical Properties

The physical properties investigated were grading specific gravity and swelling characteristics. These were done as described in section 3.3.4.2.

(a).Grading/ Particle Size Distribution.

The grading test results are shown in Figures 4.36, for moist curing periods of 7, 14, 28, 56 and 90 days respectively from the data in Appendix G.

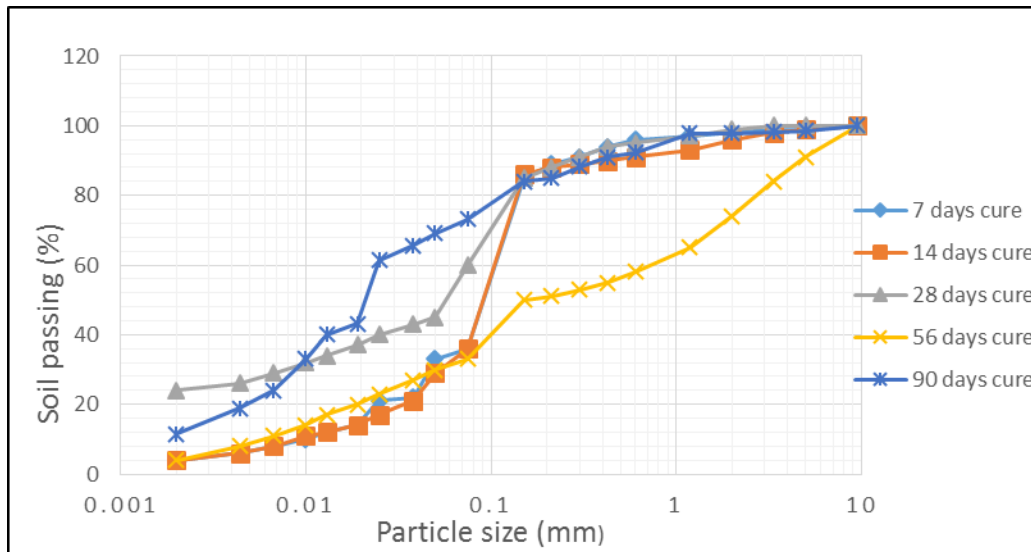


Figure 4.36: Grading curves for various curing periods

From Figure 4.36, the percentage composition of each sample in terms of clay, silt, and gravel were read off. The size ranges of various soil types are shown in Figure 2.12. A soil is either coarse or fine as discussed in section 2.3. Fine soils have at least 35% by mass finer than 0.06 mm while coarse soils have less than 35% finer than 0.06 mm. The samples cured for 7, 28, and 90 days were fine, whereas the samples cured for 14 and 56 days were coarse.

The first process that occurs when hydrated lime is added to a clay is lime modification followed by stabilization as discussed in section 2.12.4.5. Modification is the first process which is a rapid cation exchange process which forms cementitious products in a flocculation-agglomeration process, transforming the clay into a more silt-like or sand-like material which has improved plasticity, swelling, workability, and stability, resulting in a better fill material (O’Flaherty, 1974; O’Flaherty, 2002; Dang et al., 2016). The combined silt and sand content obtained in the stabilized soil was above 50%, as a result of modification; the maximum clay content was 24% at 28 days cure, a reduction of 31% which gave a better fill material, with improved swelling characteristics due to reduced

influence of the clay fraction. Silts, sands and gravels are normally non-plastic (Mitchel & Soga, 2005). The plasticity was established from atterberg limits as discussed in section 4.3.6.1(b) below, because grading is not a measure of plasticity

The grading types at the various temperature ranges are shown on Table 4.11.

Table 4.11: Grading Types for Optimal Mix

Curing (Days)	Period	D₁₀	D₃₀	D₆₀	C_u	C_c	Grading Type
7		0.010	0.0045	0.075	7.5	2.7	Well graded
14		0.010	0.055	0.075	7.5	4.0	Gap graded
28		0	0.013	0.075	-	-	Gap graded
56		0.0045	0.045	0.055	12.2	8.2	Gap graded
90		0.0017	0.009	0.025	14.7	1.9	Well graded

The grading type was established as for the neat and heated soil, as detailed in section 4.2.1.1. D₁₀, D₃₀ and D₆₀ are defined in section 2.3.2, and were obtained from Figure 4.30. The coefficient of uniformity (C_u) and the coefficient of (C_c) were determined from Equations 2.3 and 2.4 respectively.

A soil is either well graded, poorly graded, or gap graded as discussed in section 2.8.2. The 7 and 90 days cured soil have C_u greater than 5 and C_c between 1 and 3 and are therefore well graded; The 14 and 56 days cured soil have C_u greater than 5 and a C_c greater than 3. They are therefore gap graded which is apparent from Figure 4.36, since they were missing intermediate sizes, with the 56 days profile having distinct lower and upper sizes, with a “step” in the grading. The soil cured for 28 days had a D₁₀ value of

zero (0), C_u and C_c cannot be defined and is therefore gap graded, which is apparent from Figure 4.36. As the curing process progressed, the stabilized soil started as well graded, and was well graded at 90 days cure, implying it did not deteriorate with long term curing. The long term curing process formed calcium silicate hydrates, calcium aluminate hydrates and calcium aluminium silicate hydrates which gave a strong and durable material (Dang et al., 2016; Das 2016; O'Flaherty, 2002) suitable as a fill material.

(b). Atterberg Limits and Free Swell for Varying Curing Periods

The swelling characteristics of the stabilized soil are shown in Figure 4.37, plotted from the data in Appendix I. The liquid limit (W_L), the plastic limit (W_p), the plasticity index (I_p) and the linear shrinkage (LS) experienced very minor changes during the whole 90 days curing period, while the free swell (FS) decreased steadily. In the curing periods of 7, 14, 28, 56 and 90 days respectively the W_L changed by -4.4%, -1.5%, +11% and -10%; the W_p changed by -2.6%, -10.8%, +12.5% and +2.6%; the I_p changed by -6.7%, -14.2%, +37.5%, and -24.2%; the LS changed by +26.7%, -15.5%, +18.8%, and 0%; the free swell changed by -13.6%, -30%, -20.4% and +20.5% respectively. The key index property of I_p dropped from 30 to 25%, while the W_L was dropped from 68 to 64% from 7 days to 90 days of curing respectively. These results indicate that the swelling properties of the stabilized black cotton improved during long term curing. The lime-HBC-NBC system behaviour is apparently very similar to behaviour of a lime stabilized expansive clay, which causes cation exchange by replacing the monovalent ions adsorbed on the clay surface with the divalent Ca^{2+} in the lime, reducing the diffuse double layer thereby causing attraction between the clay particles forming flocs with reduced swelling properties, during the modification stage.

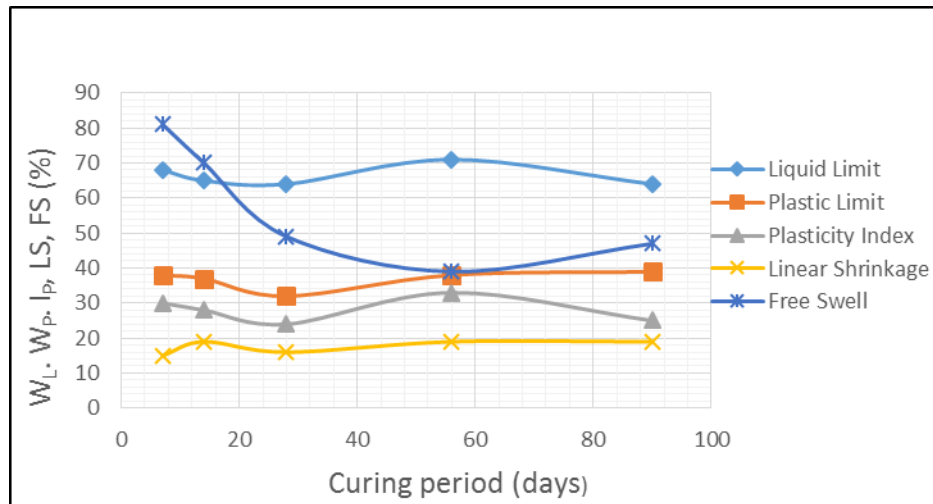


Figure 4.37: Swelling Characteristics

As discussed in section, 2.4.2.2, the desired properties for the soil to be suitable as a fill material for road construction are as indicated in Table 2.7, for marginal swell potential, with W_L ranging between 50 and 60 and I_p between 25 and 35%. However, MOTC (1987) limits the I_p to a maximum of 50, without a qualification on W_L . Based on these two requirements, the stabilized soil can be classified as being of marginal swell potential, and thus suitable as fill material for road construction. The linear shrinkage remained below 20% throughout the curing period, whereas the FS dropped to below 50% after 28 days curing. Lightly loaded structures such road pavements, residential buildings and canal linings suffer most damage when built on expansive soils (Chen,1988). For free swell values below 50%, there is no appreciable volume change during wetting or drying even under light loads (Holtz and Gibbs, 1956). Therefore the stabilized soil in this research is suitable as a fill material for road construction and other light structures such as ground floor slabs.

(c) Activity for Varying Curing Periods

The activity values for the various curing periods is shown on Table 4.12.

Table 4.12: Activity over the curing period.

Curing Period (Days)	Ip (%)	% finer than 0.002mm	Activity	Classification
7	30	4	7.5	Active
14	28	4	7.0	Active
28	24	24	1.0	Normal
56	33	9	3.7	Active
90	25	18	1.4	Active

The activity was computed from Equation 2.6 as the ratio between the I_P and the percentage finer than 0.002, and classified as indicated in section 2.9.2.2 as inactive, normal or active depending on the activity value. The results show that the stabilized soil remained active for the entire curing period, except at 28 days when the activity was normal. The levels are high with values of 7.5 and 7.0 for 7 and 14 days respectively, probably due to the fact that the soil was still in the early stages of curing. It became 1.0 at 28 days rose to 3.7 at 56 days, and dropped to 1.4 at 90 days, indicating that the soil did not deteriorate over long term curing. However, it must be noted the “activity concept” is defined for neat and not stabilized soil and thus the measure should be treated with caution. The results indicate the presence of smectites. But the swelling results above indicate that the stabilized soil is a non-swelling material, suitable as a fill material. Therefore, the activity values being a combination of two experimental results may not be applicable to the stabilized soil in this study.

(d) Classification for varying curing periods

The classification was done as for the neat and heated soil, as detailed in section 4.2.1.1, based on the BSCS illustrated in Table 2.2. The results are shown on Table 4.13, for the various curing periods.

Table 4.13: Variation of Classification with Temperature

Curing Period (Days)	Description	Symbol
7	Gravelly SILT of high plasticity	MHG
14	Very silty SAND of high plasticity	SMH
28	Gravelly SILT of high plasticity	MHG
56	Very clayey SAND of high plasticity	SCH
90	Sandy SILT of high plasticity	MHS

The grading type is shown in Table 4.11 while the particle size distribution is shown in Figure 4.36. For fine soils with at least 35% passing the 63 micron sieve, the classification was carried out using the plasticity chart shown in Figure 2.13. At 7 days cure, the soil had 35 % fine (4% clay and 31% sand) and 65 % coarse (63 % sand and 2 % gravel); the I_p of 30% and W_L of 68%. It is located below the A-line in Figure 2.13 and from Table 4.13, it is well graded and the fines are 35%. It is therefore a gravelly SILT of high plasticity with symbol MHG. The others were classified similarly and summarized in Table 4.11. The soil after stabilizing was either a silt or a sand of high plasticity. As discussed in section 4.3.6.1(a), the modification process in stabilization transformed the clay into a more silt-like or sand-like material which has improved plasticity, swelling, workability and stability resulting into a better fill material (O’Flaherty, 1974; O’Flaherty, 2002; Dang et al., 2016).

Although the plasticity is high, the stabilized soil has marginal swell potential suitable as fill material for road construction as already discussed in section 4.3.6.1 (b). Figure 2.13 gives the plasticity level in terms of the liquid limit only, whereas the swell potential which gives an indication of the performance of the soil considers the plasticity index as well. The illustration in Table 4.13 indicates that the swell did not deteriorate after long term curing.

4.3.6.2 Mechanical Test Results for varying curing periods

These are compaction, CBR and UCS. The long term tests were done for UCS only. The UCS results are discussed in section 4.3.7 under effect of long term curing and different flooding regimes on strength.

(a).Compaction Results

The compaction results for the optimal mix are given illustrated in Figure 4.38

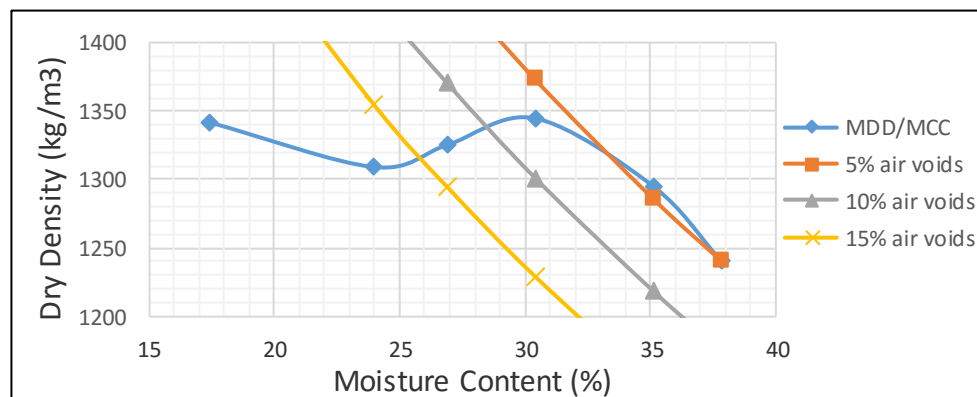


Figure 4.38: Compaction Results

The MDD was 1348 kg/m^3 with an optimum moisture content of 30%, at about 7.5% air voids, compared to NBC values which were 1272 kg/m^3 , 29.5% and 11% respectively. The specific gravity was 2.58 which was an increase of 10% from the value for the neat material which was 2.47. Thus, the stabilizing improved the packing together of the soil particles. As discussed in section 2.12.4.7 (g), a highly permeable lime stabilized material can potentially lose strength through softening and removal of the pozzolanic products (McAllister & Petry, 1992; Runigo et al., 2009). Compaction should therefore achieve a minimum air voids value throughout the fill (Sherwood, 1992). Ingress of water in a highly permeable material causes the calcium silicate hydrates (C-S-H) to de-constitute into $\text{Ca}(\text{OH})_2$ and silicate if the water is of neutral pH and Ca^{2+} levels are low, raising the pH and Ca^{2+} levels. CSH gel continues to de-constitute with the supply of more water

(Taylor, 1990). In this study, the reduction of air voids by 32%, increase of specific gravity and MDD by 10% and 6% respectively gave a less permeable material, with a high durability suitable as fill material in high moisture conditions.

(b).CBR Results

The CBR was 39.6%, and the CBR swell 0.22%, from Appendix F. Stabilizing expansive clays with lime-activated GGBS, fly ash and GGBS combined with fly ash enhances their strength and due to formation of cementitious products of calcium silicate hydrate (CSH), and calcium aluminate silicate hydrate (CASH) and calcium aluminate hydrate (CAH) from pozzolanic and hydration reactions (Saravanan, Udhayakumar, Dinesh, Venkatasubramanian & Muthu, 2017; Kavak & Kigen, 2016; Obuzor, 2012; Wild et al., 1999; Sharma & Sivapullaiah, 2016). According to Wild et al., (1999), the lime in the clay-lime mix provides the required alkaline environment for the slag activation and hydration, at the same time producing modification of the clay. It is after this that the time dependent pozzolanic and hydration reactions occur to form the cementitious products. In this study, the lime in the clay-lime mix similarly provided conditions for activation and hydration of the HBC at 400°C, at the same time causing modification of the clay, after which the cementitious products were formed in a way similar to use of fly ash, which resulted in a very high CBR value and low swell value. The CBR increased from 1% for the NBC to 39.6% for the lime- HBC-NBC mixture, a percentage increase of 3860%.

According to MOTC (1987), the minimum CBR requirement for a fill material is for a class S2 subgrade with CBR in range 5%-10%, with a maximum CBR swell of 3%. However, for a stabilized material, CBR must be a minimum of 10% after 7 days cure, and the CBR swell should be reduced to negligible values, giving a minimum class S4 material, with CBR in the range 10%-18%. The lime-HBC-NBC stabilized soil under test was class S6, with CBR value of 39.6% exceeding the minimum required value of 30% for this class, with a negligible swell value. Thus, the stabilized material is suitable

as a fill and subgrade material. The higher the subgrade strength, the more economical are the sub-base and base layers for any desired pavement structure (MOTC, 1987).

4.3.6.3 Chemical and Electro-chemical properties

The chemical and electro-chemical properties were conducted as detailed in section 3.3.4. The properties that were tested during the curing period were XRD, cation exchange capacity and organic matter content. These properties are discussed hereunder.

(a).XRD Properties

The XRD results charts for the curing periods of 7, 14, 28, 56 and 90 days are shown in the Appendix J, Figures J1-J5. As discussed in section 4.2.3.1 for the neat and heated black soil, the minerals were obtained from the diffraction of X-rays directed on the samples from which the minerals were identified using software inbuilt to the XRD equipment connected to ICDD, which categorizes minerals based on Bragg's Law (Equation 2.1). For each curing period, the minerals, their peaks in terms of d spacing, 2θ and percentage composition are given in Appendix J. The percentages of all the minerals present and the clay minerals were as shown in Tables 4.14 and 4.15 respectively. As in the case of the neat and heated soils, there were three broad peaks for 2θ between 5 and 62°.

Table 4.14: Minerals for Different Curing Periods

Mineral	Curing Period (days)				
	7	14	28	56	90
Quartz	22.9	19.8	20.0	19.0	17.2
Albite	24.9	21.2	21.7	18.6	23.9
Microcline	24.6	27.6	29.0	37.2	34.9
Montmorillonite	6.6	10.1	8.4	4.6	5.9
Calcite	16.0	15.4	11.5	15.1	14.8
Imogolite	5.1	5.9	0	3.4	3.3
Kaolinite	0	0	1.3	2.1	0

Table 4.15: Clay Minerals as a Percentage of all Minerals

Mineral	Curing Period (days)				
	7	14	28	56	90
Albite	24.9	21.2	21.7	18.6	23.9
Microcline	24.6	27.6	29.0	37.2	34.9
Montmorillonite	6.6	10.1	8.4	4.6	5.9
Calcite	16.0	15.4	11.5	15.1	14.8
Imogolite	5.1	5.9	0	3.4	3.3
Kaolinite	0	0	1.3	2.1	0

From Table 4.15 the main clay minerals were albite and microcline, whose percentage sum by weight were 49.5, 48.8, 50.7, 55.8 and 58.8 for the samples cured at 7, 14, 28, 56

and 90 days respectively. The percentages for the NBC are shown in Table 4.5, which show that percentage of albite and microcline were 26.2 and 38.5 respectively, giving a total of 64.7%. The montmorillonite increased from 3.3% to 6.6, 10.1, 8.4, 4.6, and 5.9% for the 7, 14, 28, 56 and 90 days cure respectively. The most significant increase was for the calcite, which increased from 5.8% for the neat soil to 16, 15.4, 19.5, 15.1 and 14.8% for 7, 14, 28, 56, and 90 days cure respectively. Calcite is mainly calcium carbonate, which formed due to cementitious reactions (Solanki & Zaman, 2012). Despite the relative increase in the montmorillonite and the presence of albite, the soil remained non-swelling as indicated by the swelling characteristics in section 4.3.6.1 (b), with a high strength value as measured by CBR, which was 39.6% for the 7 days cure and 7 days soak indicating high strength stability. This was due to the formation of products of lime stabilization, mainly Calcium-Silicate-Hydrate (CSH), Calcium-Aluminate-Silicate-Hydrate (CASH) from hydration and pozzolanic reaction (Das, 2016; Dang et al., 2016). The presence of lime caused formation of aggregations or flocs, which are the more significant contributor to reduction of swell (Al-Rawas, 2002). This resulted in a very strong fill and subgrade material

(b) Cation Exchange Capacity

The exchangeable cation results are shown in Table 4.16. The soil was alkaline throughout the curing period with pH above 9 due to the high percentage of Ca^{2+} . The cation exchange capacity (CEC) in meq/100 was 39.2, 25.9, 43.8, 29.9 and 51.3 for 7, 14, 28, 56 and 90 days curing periods respectively. These values were higher than 36.6 for the neat soil shown in Table 4.7 though they were lower than that of montmorillonite which has CEC ranging between 80-120, as shown in Table 2.1. Just as in the neat material, the main exchangeable cation was calcium, which had a high value exceeding 90% as a percentage of the basic cations of Ca^{2+} , Mg^{2+} and K^+ and Na^+ throughout the curing period.

Table 4.16: Minerals for Different Curing Periods

Parameter	Curing Period (days)				
	7	14	28	56	90
Soil pH-H ₂ O(1:2:5)	9.7	9.9	9.3	10.6	9.1
Elect. Cond. mS/cm	0.6	0.5	0.6	0.4	0.3
Total Org. Carbon (%)	0.7	1.2	1.0	1.28	0.2
CEC (Meq/100)	39.2	25.9	43.8	29.9	51.3
Calcium (%)	85.6	82.9	75.4	97.5	90.7
Magnesium (%)	1.75	1.70	1.34	2.3	1.3
Potassium (%)	1.59	1.2	1.4	0.6	1.65
Sodium (%)	3.0	2.6	2.8	2.2	4.09
Sum (%)	91.9	88.4	80.4	102.6	97.69
% of Calcium	93	94	94	95	92.8
Base Saturation (%)	100+	100+	100+	100+	100+

The base saturation, which is a measure of the percentage of the exchangeable cations occupied by the basic cations, remained at 100% as in the neat soil, because the stabilized soil was alkaline. In spite of this, the swelling test results indicated a non-swelling material throughout the curing period due to the products of lime stabilization from hydration and pozzolanic action, as illustrated in Figure 4.37.

(c) Organic Matter Content

The organic matter content for the various curing periods shown in Figure 4.39 indicates that it remained below 1.0 for curing of up to 56 days, but was 1.70 for 90 days cure. The limit for fill material is 5.0%, but limited to 3% for subgrades for roads (MOTC, 1987). As discussed in section 2.12.4.5(h) organic matter adsorbs calcium ions in lime, making it unavailable for formation of cementitious products of CSH, CAH and CASH, while

organic compounds in soil react with lime reducing the pH level inhibiting formation of the cementitious products. The low levels in the stabilized soil made it useful both as a fill and subgrade material for road construction. .

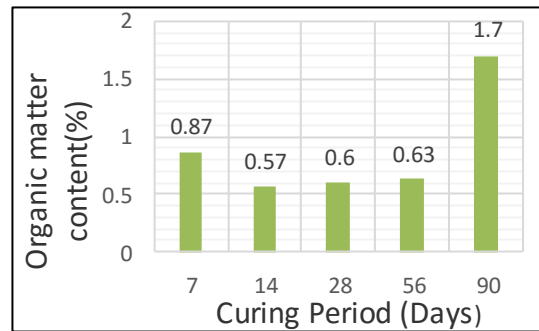


Figure 4.39: Test Results for Organic Matter Content

4.3.7 Effect of long term curing and simulated flooding regimes on strength

As discussed in section 3.3.4, the effect of long term curing and simulated flooding regimes was investigated by soaking the cured samples in water for specified periods, and then taking the UCS value at the end of these periods. The test results are shown in Appendix K. The durability of the stabilized soil is measured in-terms of the strength reduction index (SRI) which is an empirical way of expressing the strength of a soaked material as a percentage of the same material in its un-soaked state. It is also called the durability index (Obuzor, 2012; Hossain, 2006, Thomas, 2001). The partial and full soakage regimes which simulate a normal and a prolonged flood are illustrated in Figures 4.40 and 4.41 respectively for 4 and 10 days soaking periods. The 4 and 10 days soaking periods simulate the effect of a flash flood and a severe flood respectively.

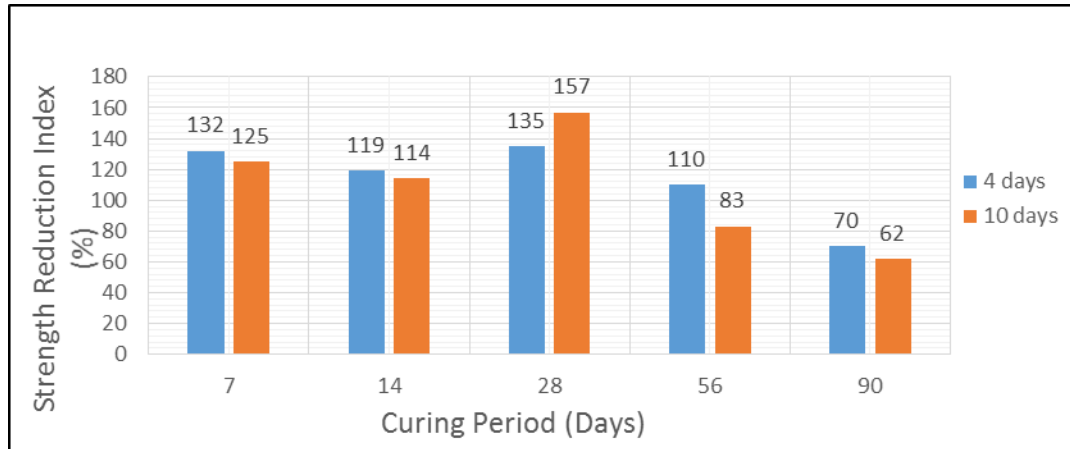


Figure 4.40: SRI values for partial soaking

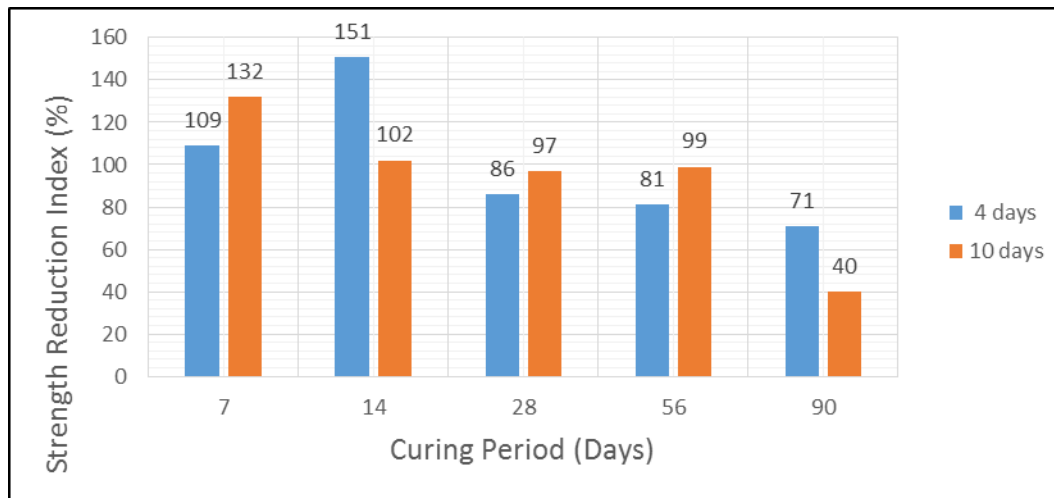


Figure 4.41: SRI values for full soaking

As illustrated in Figure 4.40, the 4 days partial-soak SRI values were above 100%, up-to 56 days of curing, and then dropped to 70% at 90 days. This reflects a very high level of stability of the stabilized soil. The partial soak ensured gradual availability of water for the hydration process, enhancing the curing process. The 10 days partial-soak SRI values were above 100%, up-to 28 days of curing, and then dropped to 83% and 62% for 56 and 90 days curing respectively. They were generally below the values for the 4 days soak,

except for the 28 days cure which was 22% higher. This represents a high level of stability up-to 28 days of curing, with a drop to 83% and 62% for 56 and 90 days respectively. The normal severe flood therefore has a more severe impact than the normal flash flood. Despite this the lowest strength recorded for a normal flood scenario was 75 kN/m² at 56 days curing, which is strength of a Kenyan red soil which has UCS values in the range 75-100 kN/m², and is used for fill and subgrades where it is available. It is worth noting that the strength recovered and became 139 kN/m² at 90 days. Most road construction projects have construction projects exceeding one year and therefore by the time the road is fully loaded, the strength of the fill and subgrade will have recovered even if the road were to experience flooding during the construction period.

Therefore the stabilized soil is suitable for use as a fill and subgrade material for road construction for flash floods, without risk of collapsing. The best curing period for normal flooding is 28 days; after this, flooding increased the strength by 35% and 57% for the 10 days and 4 days partial soak respectively.

As illustrated in Figure 4.41, the 4 days submerged samples had SRI values higher than 100% for 7 and 14 days curing, while SRI values decreased steadily from the 28, 56 and 90 days curing period. Except for the 14 and 90 days cure, the SRI values for the 4 days soak were lower than for the 10 days soak. The latter were basically 100% and over except for the 90 days cured sample which was 40%. The UCS for this latter scenario was 89 kN/m² which is a good strength for fills and subgrade. The lowest strength for full submergence was 73kN/m² for 56 days cure and 4 days soak. However there was good recovery and the 90 days cure, 4 days soak gave a strength of 158 kN/m².

The stabilizing thus produced a strong durable material, useful for fills in roads and buildings and also as a subgrade material for roads. The material withstood the worst simulated environment of severe prolonged flooding.

In both the normal and severe flooding regimes, there is was general trend of decrease in strength as the curing period increased as illustrated in Figures 4.40 and 4.41. The drop in

strength up-to 56 days curing was a maximum of 19% which is considered not too significant, because the stabilized soil needed adequate moisture to satisfy the moisture demand for completion of the lime and lime-heated soil activated pozzolanic and hydration reactions. The greatest reduction was 40% for 90 days cured severe prolonged flooding scenario. Figure 4.34 illustrates that there is a continuous increase in strength as the compaction moisture content increased. According to Obuzor et al (2012) in lime-activated ground granulated blast furnace slag (GGBS) stabilized soil, SRI values increase up-to 56 days, after which they start decreasing because the active period of moisture demand approaches its ultimate at 56 days when the hydration reaction nears completion. Further addition of moisture caused water to enter into the void spaces, causing a deleterious effect of reduced strength. At this point, the free lime and pH are in short supply. In this study, the same trend is apparent. The active period for moisture demand was ultimate at period of 56 days when the strength stabilized, and hydration apparently has reached completion. However it may not be practical to allow for a long curing period on site. Wild, Kinuthia, Jones and Higgins (1998) suggested a moist curing period of 28 days before the stabilized fill material gets saturated with water, which agrees very well with the results of this study where the maximum loss of strength was 14% for the 28 days cured 4 days soak severe flooding regime.

As discussed in section 2.12.4.5(g), the pozzolanic products of lime stabilization are CSH, CAH, and CASH, with CSH being the principal cementitious product. These products contribute to the flocculation process by bonding adjacent soil particles together into larger-sized aggregate which strengthen the soil as curing occurs. However, where free lime and Ca^{2+} and pH are in short supply especially when water enters the voids of stabilized soil particles, the CSH de-constitutes into $\text{Ca}(\text{OH})_2$ and silicate (Taylor, 1990). Although this raises the pH and Ca^{2+} , with the supply of more water the CSH gel will continue to de-constitute, explaining why the strength dropped for curing periods of 56 and 90 days in presence in flooding scenarios. This is what was happened to the lime-activated heated soil-black cotton mixture in this research. The pozzolanic properties of

the heated soil had the same effect as the GGBS, although GGBS is a calcium rich alluminosilicate, compared to the HBC at 400°C which is a low calcium pozzolanic alluminosilicate. Therefore the heated soil at 400°C is effective as a low calcium pozzolanic alluminosilicate precursor in stabilization of black cotton soils, which aided in formation of more CSH gel and change in micro-pore orientation, forming a strong durable material for use as a fill material for roads and ground floor slabs, and also for use as a subgrade. Ideally the stabilized soil should be moist cured for 28days before exposure to flooding. This 28 days period gave the higher SRI values for normal flooding, with hardly any change between the 28 and 56 days cure for the prolonged flooding.

4.3.8 Performance of Model House

As discussed in section 3.3.4, the settlement of a loaded ground floor slab placed on a stabilized fill was measured for various curing periods with 10 days side flooding after 90 days cure to establish the long term behaviour of the stabilized black cotton soil as a fill material for ground floor slabs. The settlement levels are shown in Table 4.17.

Table 4.17: Heave/settlement of the model house

Curing Period (days)	Heave (Settlement) -mm
7	0
14	0
28	(5)
56	(3.1)
90	(3.0)
90 days and 10 days side flooding	(2.1)

7.

The max settlement was 5 mm after 28 days moist curing. This reduced to 3.1 mm and 3.0 mm respectively after 56 days and 90 days moist curing and to 2.1 mm when the

sides were flooded for 10 days after 90 days moist curing. The maximum allowable settlement for spread footings is 25 mm (Bowles, 1996) which can equally apply to slabs on grade; this is a lot higher than the 5 mm attained. There was no heave. Construction procedures for structures in expansive clays is shown in Table 2.9, based on the total predicted heave. For heaves of up-to 6.35 mm no precaution is required in construction where L/H is up-to 1.25. Therefore, the stabilized fill performed very well, and thus the stabilized material can effectively be used as a fill material for ground floor slabs. It is recommended that the slab be isolated from the walling

4.3.9 Summary: Effect of stabilizer mixes on properties of expansive clay soils

This study has established that an a mix comprising HBC at 400°C, lime and molasses at ratios of 14, 6 and 0 respectively gives and optimal mix for stabilizing black cotton soil as discussed in section 4.3.3, which gave high values for the key parameters of UCS and CBR. The UCS value was 398 kN/m² at 28 days cure for a compaction moisture content of 36%, while the soaked CBR was 39.6% and CBR swell 0.22%. The results for this mix for curing periods of 7, 14, 28, 56 and 90 days are discussed in section 4.3.6.1 for grading, swelling characteristics and classification. Results for compaction and CBR are discussed in section 4.3.6.2, while XRD, CEC, pH and organic matter content are discussed in section 4.3.6.3. The results for effect on strength on long term curing and flooding regimes are discussed in section 4.3.7 and for the performance of the model house in section 4.3.8. All these results indicate a strong durable material suitable as a fill for road construction and ground floor slabs. Therefore objective two, effect of mixes of heated soil, lime and molasses on properties of expansive clay soils was achieved.

4.4 Economic viability of Using Improved Soil

The costs assessed were for cutting and replacing and comparing it with cost of heating the soil to 700°C, and stabilizing the soil with mixes of heated soil and lime in both road-works and buildings. The secondary data on costs for road works is given in Appendix L.

The current costs for buildings are as published in MTIH&UD (2017) - Current Construction Costs Handbook 2017/2018.

4.4.1 Cut and Replace

4.4.1.1 Road-works

Table 4.18 shows the cost of cutting and disposing expansive clay sand replacing it with suitable material, for filling with soft and hard materials as alternatives. The cost per cubic metre of fill for soft material is Kshs 701.75, say Kshs 705 while that of filling with hard material is Kshs.1393.95, say Kshs 1400 which is almost twice as much. Therefore, subject to availability soft material should be used

4.4.1.2 Buildings

The cost of cutting and replacing expansive clay in buildings is shown in Table 4.19 for filling in hardcore, murrum and red soil as alternatives. The costs per cubic meter of fill are Kshs 2200, Kshs 4250 and Kshs 1984, say Kshs 1985 respectively. The highest cost is for murrum, probably due to the high cost of excavation compared to red soil which is a soft material, making it the cheapest alternative. The cost of hardcore is just slightly above that of red soil, probably due to the fact that hardcore is sold as a waste material in quarrying for building stone. However, it is more difficult to lay and compact hardcore, explaining why the cost is comparable to that of red soil. Depending on availability of materials, red soil should be used.

Table 4.18: Cutting and replacement (Roadworks)

Item	Description	Quantity	Unit	Rate (kshs)	Amount	
					Kshs	cts
4.01	Cut to spoil in soft material	1.0	m ³	193	193	00
4.02	Compaction of 150mm thick original ground under fill below subgrade to 95% MDD(AASHTOT99)	0.15	m ³	233	34	95
4.03(a)	Fill in soft material to 95% MDD (AASHTO T99) in layers of 150mm each	1.0	m ³	454	454	00
4.03(b) (alternative to 4.03(a))	Fill in hard material to 95% MDD (AASHTO T99) in layers of 150mm each	1.0	m ³	1166	(1166)	00
4.04	Extra over item 4.03(a) for compaction of top 300mm below formation levels in fills (subgrade) 100% MDD (AASHTO T99)	0.3	m ³	66	19	80
Total cost per m³						
(a) – Fill in soft material					701	75
(b)– Fill in hard material					1393	95

Table 4.19: Cutting and replacement (Buildings)

Item	Description	Quantity	Unit	Rate (Kshs)	Amount	
					Kshs	cts
4.10	Excavate over site to reduce levels not exceeding 1.5m deep starting from existing ground level	1.0	m ³	330	330	00
4.11	Cart away from site surplus excavation spoil way from site	1.0	m ³	520	520	00
4.12(a)	Approved hardcore well compacted	1.0	m ³	1350	1350	00
4.12(b)	Approved murrum filling consolidated in layers	1.0	m ³	3400	3400	00
4.12(a)						
4.12(c)	Approved red soil filling consolidated in layers	1.0	m ³	1134	1134	00
4.12(a)						
Total cost per m³						
(a)	Fill in hardcore				2200	00
(b)	Fill in murrum				4250	00
(c)	Fill in red soil				1984	00

4.4.2 Filling with black cotton heated to 700°C.

Filling at this temperature was assessed for road-works and buildings. The cost of heating is based on specific heat capacity of wet soils which is 1480J/kg/°C, higher than that for dry soil is 800J/kg/°C, and the current cost of electricity in Kenya which Kshs 17.00 per kilowatt hour. The research established that the optimum temperature is 700°C for a period of 2 hours from a room temperature of 25°C.

4.4.2.1 Road-works

Table 4.20 shows the cost of filling with the heated soil. The process involves compacting the original ground, heating, processing and compacting 1m deep layer of soil, and compacting the top 300 mm layer as an extra over. Cost is for 1m³ of fill. The cost is Kshs 11,355.75, say Kshs 11,360, which is about 15 times higher than filling with soft material, and about 8 times than filling with hard material due to the high temperature of heating.

Table 4.20: Filling with heated soil (Roadworks)

Item	Description	Quantity	Unit	Rate (Kshs)	Amount	
					Kshs	cts
4.20	Compaction of 150mm thick original ground under fill below subgrade to 95% MDD(AASHTOT99)	0.15		233	34	95
4.21	Heating black cotton in-place	1.0	m3	10,743	10,743	00
4.22	Processing heated black cotton	1.0	m3	325	325	00
4.23	Compaction of heated soil fill to 95% MDD (AASHTOT99)	1.0	m3	233	233	00
4.24	Extra over item 4.22(a) for compaction of top 300mm below formation levels in fills (subgrade) 100% MDD (AASHTO T99)	0.3	m3	66	19	80
Total cost per m³					11,355	75

4.4.2.1 Buildings

The cost for filling buildings with heated soil is shown on Table 4.21. It totals kShs. 10,933.00, say 10,940 which is slightly lower than for in roads. which is about 15 times higher than filling with soft material, and about 8 times than filling with hard material due to the high temperature of heating.

Table 4.21: Filling with heated soil (Buildings)

Item	Description	Quantity	Unit	Rate (Kshs)	Amount	
					Kshs	cts
4.30	Excavate over site to reduce levels not exceeding 1.5m deep starting from existing ground level	1.0	m ³	330	330	00
4.31	Heating black cotton soil	1.0	m ³	10473	10473	00
4.32	Return fill and ram selected excavated material	1.0	m ³	130	130	00

4.4.3 Filling with Stabilized Soil.

The stabilized soil consisted 80% of NBC, 14% of HBC at 400°C and 6% lime. The specific capacity of the soil was taken as 1480J/kg/°C and cost of electricity at Kshs 17.0 per kilowatt hour in Kenya

4.4.3.1: Road-works

The cost for road-works is shown on Table 4.21, and it totals Kshs 3885.75, say Kshs 3900.00.

Table 4.22: Filling with stabilized black cotton soil (Roadworks)

Item	Description	Quantity	Unit	Rate (Kshs)	Amount	
					Kshs	cts
4.40	Compaction of 150mm thick original ground under fill below subgrade to 95% MDD(AASHTOT99)	0.15	m ³	233	34	95
4.41	Heating black cotton in place	145.20	kg	5.24	760	85
4.42	Supply lime for stabilization	63	kg	26.78	168	15
4.43	Processing the soil in place	1.0	m ³	317	317	00
4.44	Mixing lime with soil	1.0	m ³	317	317	00
4.45	Compaction of stabilized fill in layers not exceeding 180mm to 95% MDD(AASHTOT99)	1.0	m ³	233	233	00
4.46	Curing and protection of treated layers	6	m ²	86	516	00
4.47	Extra over item 4.4 for compaction of top 300mm below formation levels in fills (subgrade) 100% MDD (AASHTO T99)	0.3	m ³	66	19	80
Total cost per m³					3,885	75

4.4.3.2 Buildings

The cost of filling with stabilized black cotton soil is shown in Table 4.23. The cost is Kshs. 3844.00, say Kshs 3850.00 which is very close to the cost of filling in road-works due to use of equipment. It is cheaper than filling with murrum, by about 9% which is significant.

Table 4.23: Filling with stabilized black cotton soil (Buildings)

Item	Description	Quantity	Unit	Rate (Kshs)	Amount	
					Kshs	cts
4.50	Excavate over site to reduce levels not exceeding 1.5m deep starting from existing ground level	1.0	m ³	330	330	00
4.51	Heating soil in place	145.20	kg	5.24	760	85
4.52	Supply lime for stabilization	63	kg	26.78	1,687	15
4.53	Mixing lime with soil	1.0	m ³	317	317	00
4.54	Compaction of stabilized fill in layers not exceeding 180mm to 95% MDD(AASHTOT99)	1.0	m ³	233	233	00
4.55	Curing and protection of treated layers	6	m ²	86	516	00
Total cost per m³					3844	00

4.4.4: Summary-cost of filling.

The cost of filling is summarized in Table 4.24, which clearly demonstrates that the specific objective 3, to evaluate the economic viability of using heated soil, lime and molasses as stabilizers for expansive clay was achieved.

Table 4.24: Summary- Cost of filling (in Kenya Shillings)

Type of Fill	Road-Works	Buildings
Cut and Replace	SM - 705	HC - 2200
	HM - 1400	MRM- 4250
		RS - 1985
Heated Soil at 700°C	11,360	10,940
Stabilized Fill with 14% HBC at 400°C and 6% lime	3900	3850

Legend: SM - Soft material; HM - Hard material; HC - Hardcore; MRM – Murram
RS – Red soil

The highest cost is for filling with heated soil at 700°C, which is Kshs. 11,360 for road-works and Kshs. 10,390 for buildings. Though it is of the same order, the cost in road-works is 9% higher than in buildings mainly because in buildings there is no provision for compacting the original ground and for a higher compaction of the top of the fill. The lowest costs are for cutting and replacing in both cases except in buildings where the cost of filling in murrum is fairly high. The cost for filling with stabilized material is Kshs. 3900 for road-works, and Kshs. 3850 for buildings which is basically the same. The aim of the research was to establish the alternative to cutting and replacing expansive clays, with a material which can withstand harsh environmental conditions. The research has established that the stabilized material is strong and durable. Prusinski and Bhattacharja (1999) and Yang and Zheng (2006) state that excavation and importing material for replacement has huge cost implications due to increase in carbon footage resulting from burning fossil fuel or cost due to damage to the environment resulting in increase in greenhouse gases, responsible for global warming. Therefore, the cost of stabilizing is

viable considering that other suitable materials and spoil sites may not be available and the environmental implications. However carbon foot print and costs on the environment were not part of this study. Therefore, the third specific objective, evaluating the economic viability of stabilization has been achieved.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The following conclusions were made for the three objectives in sequence.

1. Properties of Heated Black Cotton Soil

Heating the black cotton soil under study improved its physical, mechanical and chemical properties and made it suitable as a fill and subgrade material for roads, as well as a suitable fill material for ground floor slabs. The optimum heating temperature was established as 700°C. The key swelling parameters of liquid limit, plastic limit, plasticity index, linear shrinkage reduced to zero, and the free swell became 9%, making the soil non-swelling. The soaked CBR increased from 1% to 16% making it a class S4 subgrade, while the CBR swell reduced from 4.95% to 0.6%, compared to MOTC(1987) requirements of minimum CBR of 5%, and maximum CBR swell of 2%. The organic matter content was 1.62%, against the MOTC (1987) requirement of 3% maximum. The heated soil became a very strong, non-swelling material meeting MOTC (1987) requirements for fill and subgrade materials for roads and fill for ground floor slabs.

2. Properties of Stabilized Black Cotton Soil.

An optimal lime-activated stabilizer with low calcium pozzolanic allumino-silicate heated black cotton soil heated at 400°C as the precursor was obtained, comprising 14% heated black cotton soil at 400°C, 6% lime. The stabilized black cotton soil resulted into a very strong, non-swelling material with a very high soaked CBR of 39.6% giving a class S6 subgrade, and a CBR swell 0.22% suitable as a fill and subgrade material for roads, and as a fill for ground floor slabs. During the 90 days curing period, the plasticity index and the organic matter content were a maximum of 33% and 1.7% against the

MOTC(1987) requirements of 50% and 3% maximum respectively. This stabilized soil was able to withstand harsh full soaking period of 10 days, representing a prolonged severe flood. Though the durability decreased with increasing curing period, the final product was acceptable as a fill and subgrade material at 90 days curing, and 10 days soaking.

The ground floor slab experienced no heave for the long term curing period of 90 days and 10 days side flooding. The maximum settlement of 5 mm experienced in these conditions was negligible. Mixes including molasses were found to be of low strength because the sugars produced are water soluble, and causing the stabilized mix to have low strength.

3. Economic Viability

For road works, the cost of filling with heated soil at 700°C was Kshs 11,360 compared to Kshs 3,900 for filling with black cotton soil stabilized with heated black cotton soil at 400°C and 6% lime. The cost for cutting and replacing with suitable material was Kshs 705 and Kshs. 1400 for filling with soft and hard materials respectively. For ground floor slabs, the cost of filling with the heated soil was Kshs 10,940; the cost of filling with the stabilized soil was Kshs 3850 while cutting and replacing was Kshs 1,985, Kshs 2,200 and Kshs 4,250 for filling with red soil, hardcore and murrum respectively. Thus the cost of filling with the heated soil was very high in both cases, and of the same order.

Although the stabilized material was generally more expensive than cutting and replacement (except for filling with hardcore in buildings which was slightly more expensive), the benefits of using in-place materials and saving on imported materials from arable land, reduction in depletion of natural materials, the reduction in waste materials, the avoidance of largely uncontrollable legislated costs and taxes of importing and spoiling materials, the benefits to the environment in decreasing carbon footage due to reduction in burning of fossil fuel during haulage and reduction in emission of

greenhouse gases leading to global warming far outweigh this higher cost. In locations where suitable fill material is not readily available such as in Homa Bay and Tana River Counties in Kenya, stabilization as established in this research is the most viable option, with heating the soil as the last option. However, environmental costs were not in the scope of this study, and therefore their monetary implications were not investigated.

5.2 Recommendations

1. Black cotton soil heated to 700°C can be used as a very strong and non- swelling in-place fill and subgrade material in road construction and as a fill for ground floor slabs, where lime is not readily available.
2. Lime activated heated black cotton soil at 400°C should be developed as a viable strong and durable in-place stabilizer for black cotton soil for use as a fill and subgrade material in road construction and as a fill material in ground floor slabs. This will convert a waste into riches and contribute to Kenya's vision 2030 as well as contribute to UN agenda on Sustainable Development Goals, particularly goals 9 on resilient infrastructure and 11 on resilient and sustainable cities and human settlements. It will also reduce negative environmental consequences of borrow and dumping sites which are very high (Prusinski & Bhattacharja, 1999; Yang & Zheng, 2006) as well as avoid use of arable materials.

5.3 Recommendations for Further Research

1. There is need to develop a cost effective in-situ thermal-stabilization method
2. It is recommended that a physical model of the stabilized and the heated black cotton soil be constructed for a trafficked road and the effects of long term curing and various flooding regimes be investigated.
3. It is recommended that long term field trials be done with the stabilized and the heated black cotton soil before use on a large scale.

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APPENDICES

Appendix A: Particle size distribution for NBC & HBC

Particle Size distribution for NBC & HBC										
Sieve Size(mm)	NBC	HBC at temperatures indicated in °C								
		200	300	400	500	600	700	800	1000	1200
5	100.0	100.	100.0	100	100	93	77	99.3	100	99.5
3.35	99.25	100.0	100.0	100	100	75	70	83.5	73.3	98
2.0	98.50	100.0	100.0	98.8	100	63	59	63.8	45.5	66.5
0.6	97.00	96.00	95.00	96.0	75	41	45	34.3	10.8	27
0.3	95.25	93.00	91.50	89	64.0	32.0	39	27.0	4	20
0.212	94.25	92.00	90.00	87.3	61.0	30.0	38	27	2.75	19
0.15	92.75	90.00	87.75	83.5	57	26.0	36	27	2	17
0.0750	89.50	73.70	75.00	68.8	33	21.0	35	27	2	17
0.0350	62.30	56.50	60.75	41.3	22	13	0	0	0	0
0.0260	56.70	53.70	59.00	35.8	20	11.8	0	0	0	0
0.0190	53.30	52.00	58.00	34.3	17	10.3	0	0	0	0
0.0090	47.00	48.70	55.70	24.8	12	5.7	0	0	0	0
0.0070	43.00	48.00	52.70	23	9	4.25	0	0	0	0
0.0050	39.70	45.70	51.00	22.3	4.5	3.75	0	0	0	0
0.0020	35.00	42.30	48.30	21	4	0.80	0	0	0	0

Appendix B: Swelling properties for NBC and HBC

Heating Temperature	Atterberg Limits				Free Swell (%)
	W_L	W_P	I_P	LS	
Neat (unheated)	65	35	30	15	74
200 ⁰ C	66	27	39	19	85
300 ⁰ C	61	28	33	17	85
400 ⁰ C	55	29	27	14	59
500 ⁰ C	43	20	23	12	38
600 ⁰ C	44	24	20	10	46
700 ⁰ C	Non Plastic	Non Plastic	Non Plastic	Non Plastic	9
800 ⁰ C	Non Plastic	Non Plastic	Non Plastic	Non Plastic	Non Plastic
1000 ⁰ C	Non Plastic	Non Plastic	Non Plastic	Non Plastic	Non Plastic
1200 ⁰ C	Non Plastic	Non Plastic	Non Plastic	Non Plastic	Non Plastic

Appendix C: Specific Gravity for NBC and HBC

Soil Type	HBC	HBC Temperature(°C)					
		200	300	400	500	600	700
Specific Gravity	2.47	2.76	2.63	2.62	2.78	2.60	2.78

Appendix D: CBR, CBR Swell & UCS for NBC & HBC

Reference	CBR	CBR Swell (%)	UCS (kN/m ²)
NBC	1	4.95	44.1
HBC at 200°C	1	2.2	17
HBC at 300°C	2	1.0	27.7
HBC at 400°C	2	2.4	35.3
HBC at 500°C	3	0.6	55.2
HBC at 600°C	3	1.9	-
HBC at 700°C	16	0.6	-

Appendix E: XRD For NBC & HBC

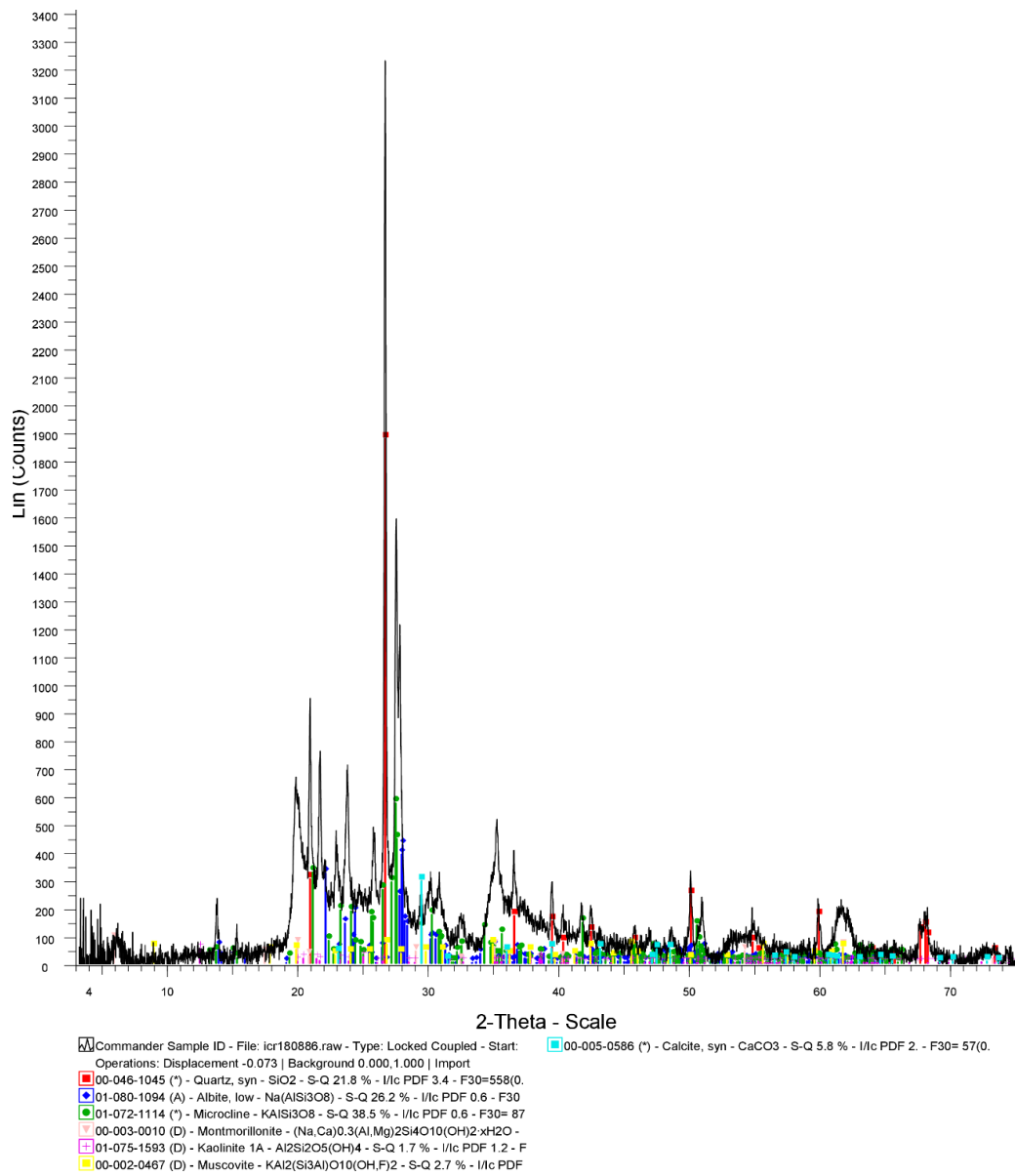
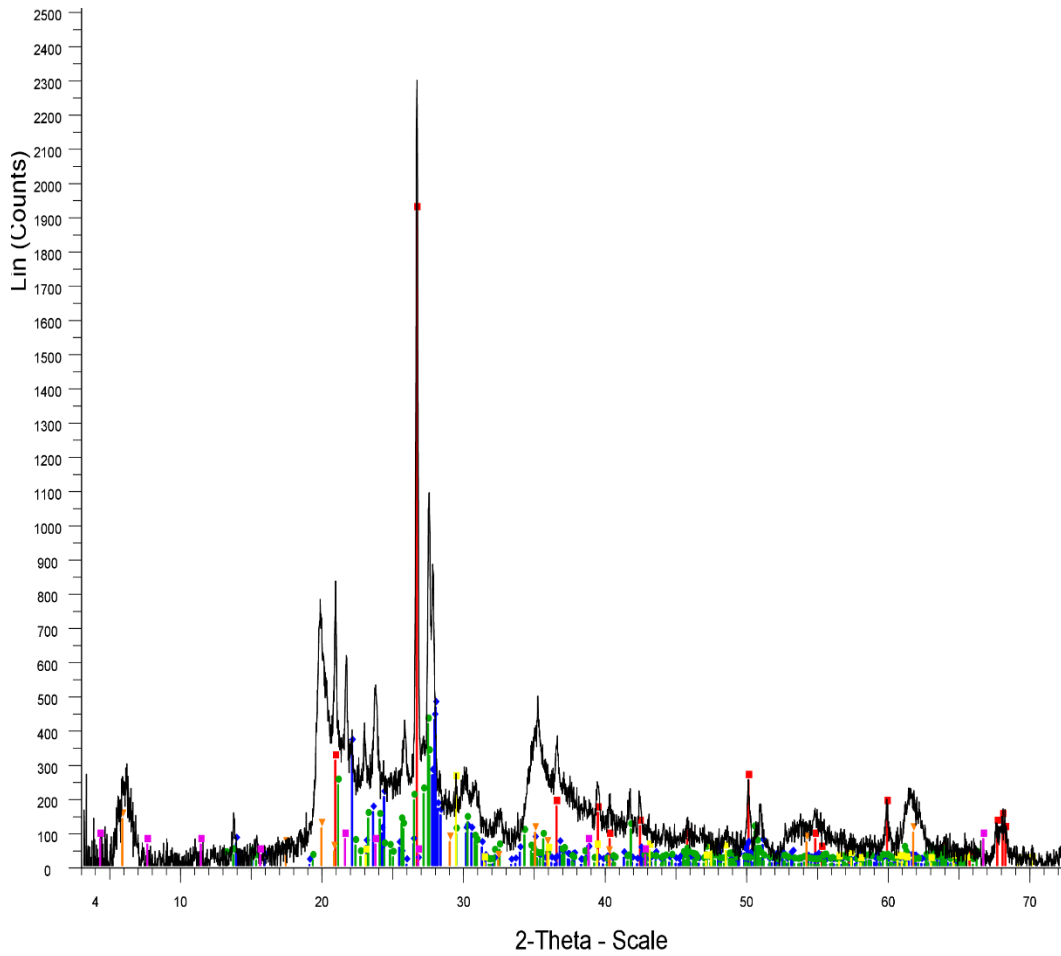


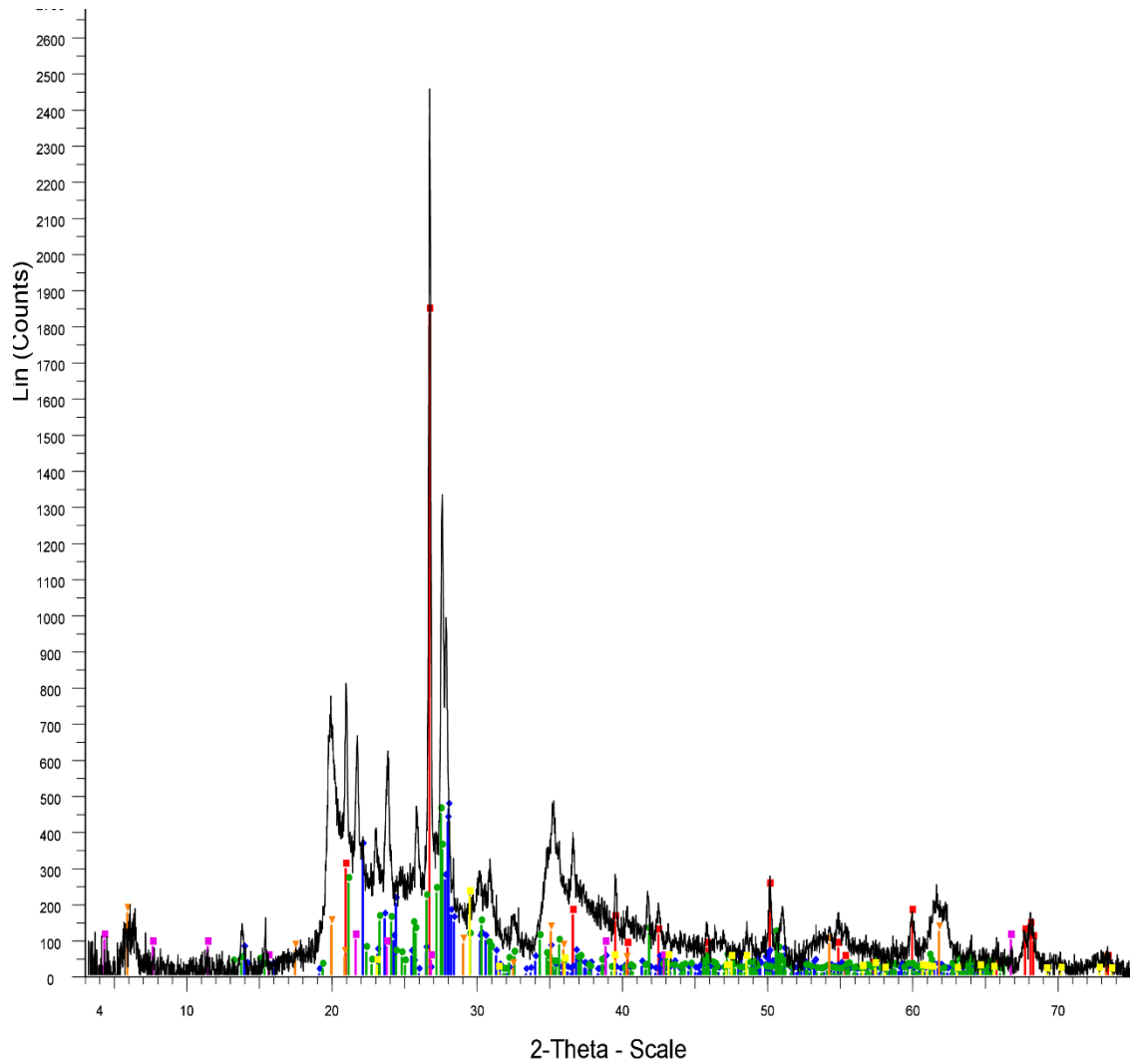
Figure E1: NBC



Commander Sample ID - File: icr180231.raw - Type: Locked Coupled - Start: 3.060 ° - End: 75.055 ° - Step: 0.020 ° - Step time: 96. s - Temp.: 25 °C (Room) - Ti
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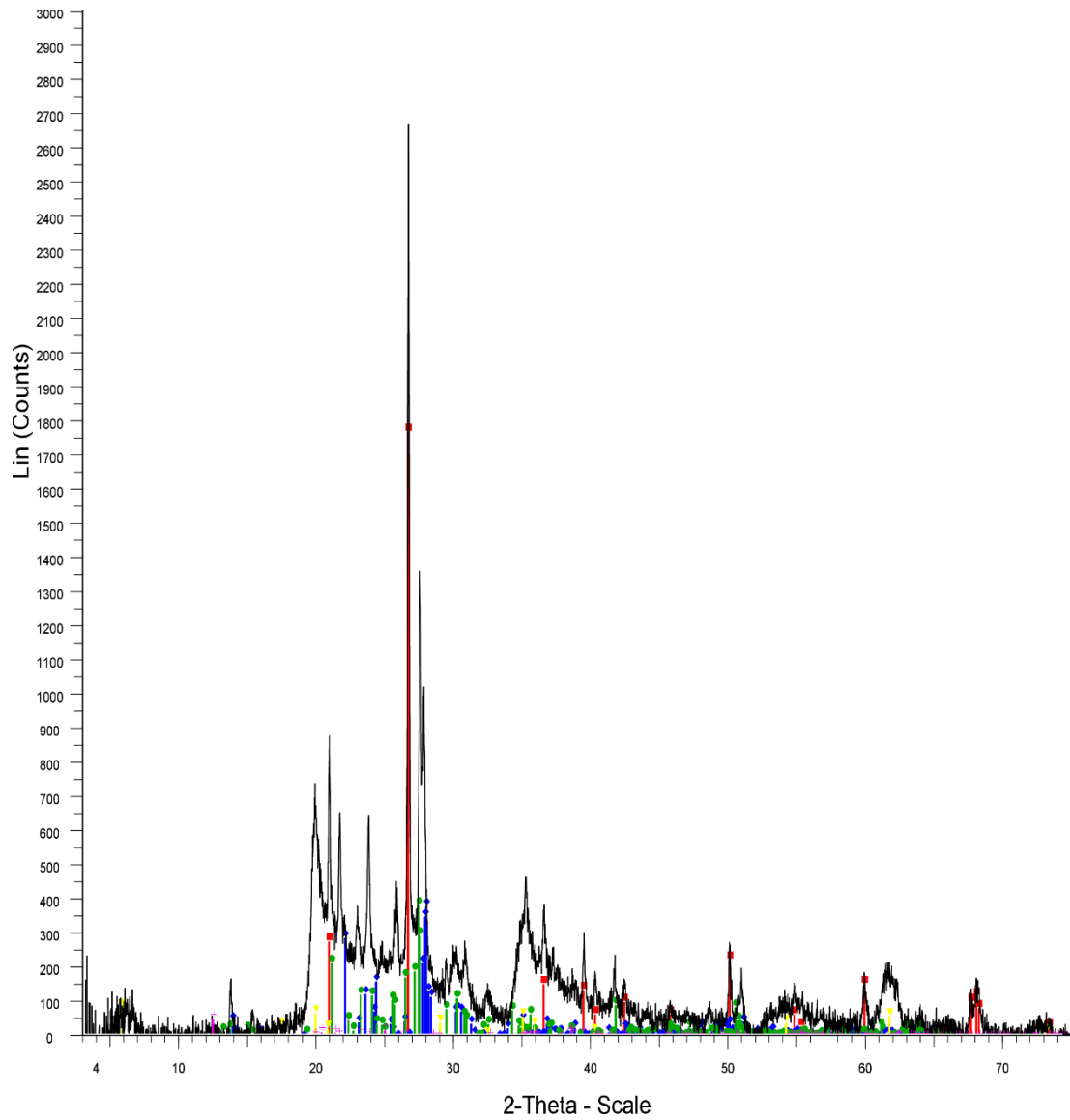
- 00-046-1045 (*) - Quartz, syn - SiO₂ - S-Q 24.2 % - I/Ic PDF 3.4 - F30=558(0.0011,31)
- 01-080-1094 (A) - Albite, low - Na(AlSi₃O₈) - S-Q 31.2 % - I/Ic PDF 0.6 - F30=169(0.0040,37)
- 01-072-1114 (*) - Microcline - KAlSi₃O₈ - S-Q 30.2 % - I/Ic PDF 0.6 - F30= 87(0.0084,42)
- 00-003-0010 (D) - Montmorillonite - (Na,Ca)0.3(Al,Mg)2Si₄O₁₀(OH)₂·xH₂O - S-Q 5.8 % - I/Ic PDF 1. -
- 00-005-0586 (*) - Calcite, syn - CaCO₃ - S-Q 5.3 % - I/Ic PDF 2. - F30= 57(0.0158,33)
- 00-025-1493 (D) - Imogolite - Al₄Si₃O₇(OH)₁₀·xH₂O - S-Q 3.3 % - I/Ic PDF 1. -

Figure E2: HBC at 200°C



Commander Sample ID - File: icr180232.raw - Type: Locked Coupled - Start: 3.060 ° - End: 75.055 ° - Step: 0.020 ° - Step time: 96. s - Temp.: 25 °C (Room) - Tim
 Operations: Displacement -0.073 | Background 0.000,1.000 | Import
 00-046-1045 (*) - Quartz, syn - SiO₂ - S-Q 22.5 % - I/Ic PDF 3.4 - F30=558(0.0011,31)
 01-080-1094 (A) - Albite, low - Na(AlSi₃O₈) - S-Q 30.1 % - I/Ic PDF 0.6 - F30=169(0.0040,37)
 01-072-1114 (*) - Microcline - KAlSi₃O₈ - S-Q 31.7 % - I/Ic PDF 0.6 - F30= 87(0.0084,42)
 00-003-0010 (D) - Montmorillonite - (Na,Ca)_{0.3}(Al,Mg)₂Si₄O₁₀(OH)₂·xH₂O - S-Q 7.1 % - I/Ic PDF 1. -
 00-005-0586 (*) - Calcite, syn - CaCO₃ - S-Q 4.5 % - I/Ic PDF 2. - F30= 57(0.0158,33)
 00-025-1493 (D) - Imogolite - Al₄Si₃O₇(OH)₁₀·xH₂O - S-Q 4.0 % - I/Ic PDF 1. -

Figure E3: HBC at 300°C



Commander Sample ID - File: icr180882.raw - Type: Locked Coupled - Start: 3.060 ° - End: 75.055 ° - Step: 0.020 ° - Step time: 96. s - Temp.: 25 °C (Room) - Tim
 Operations: Displacement -0.073 | Background 0.000,1.000 | Import
 00-046-1045 (*) - Quartz, syn - SiO₂ - S-Q 27.0 % - I/Ic PDF 3,4 - F30=558(0.0011,31)
 01-080-1094 (A) - Albite, low - Na(AlSi₃O₈) - S-Q 31.4 % - I/Ic PDF 0.6 - F30=169(0.0040,37)
 01-072-1114 (*) - Microcline - KAlSi₃O₈ - S-Q 34.3 % - I/Ic PDF 0.6 - F30= 87(0.0084,42)
 00-003-0010 (D) - Montmorillonite - (Na,Ca)_{0.3}(Al,Mg)₂Si₄O₁₀(OH)₂·xH₂O - S-Q 4.8 % - I/Ic PDF 1. -
 01-075-1593 (D) - Kaolinite 1A - Al₂Si₂O₅(OH)₄ - S-Q 2.5 % - I/Ic PDF 1.2 - F30=1000(0.0007,33)

Figure E4: HBC at 400°C

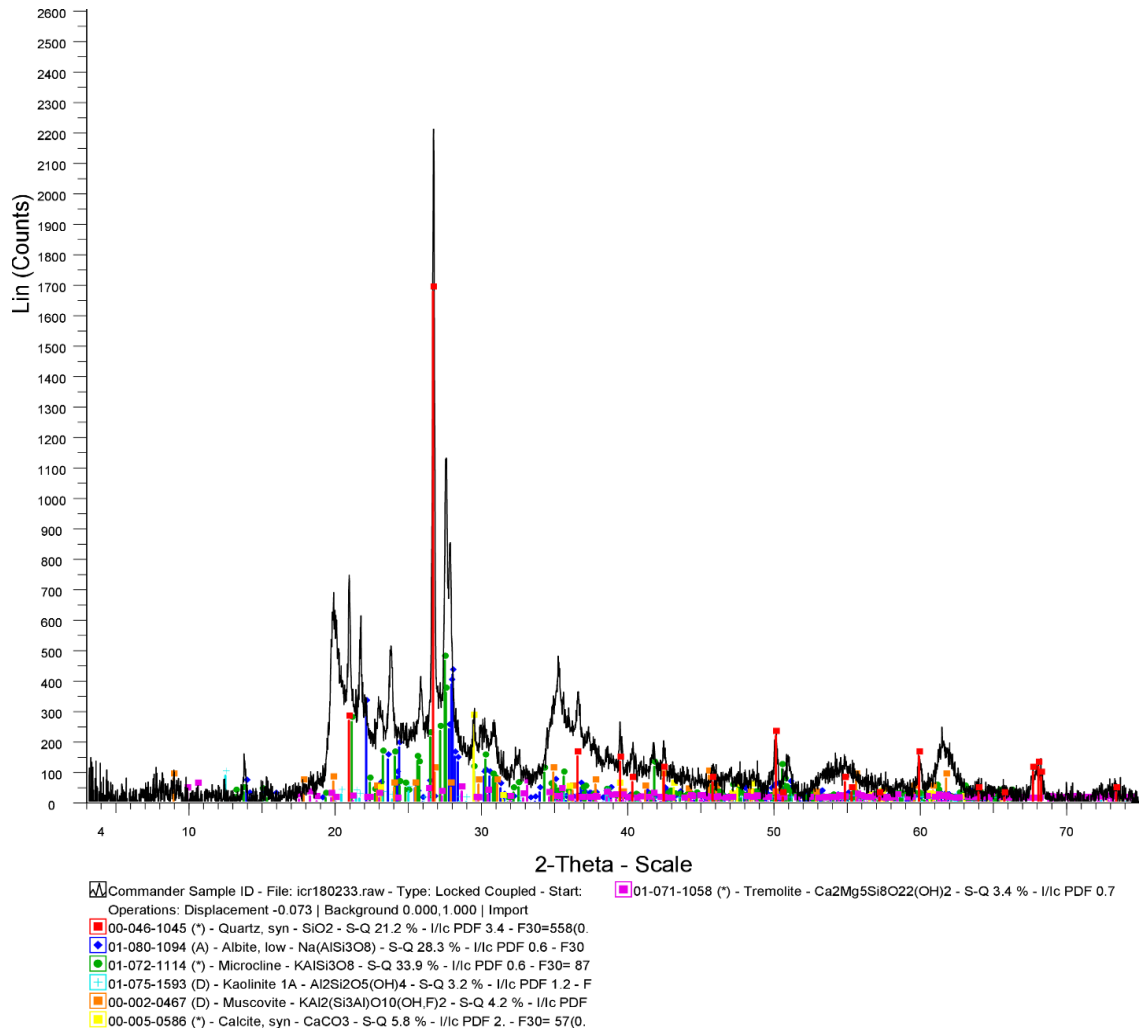
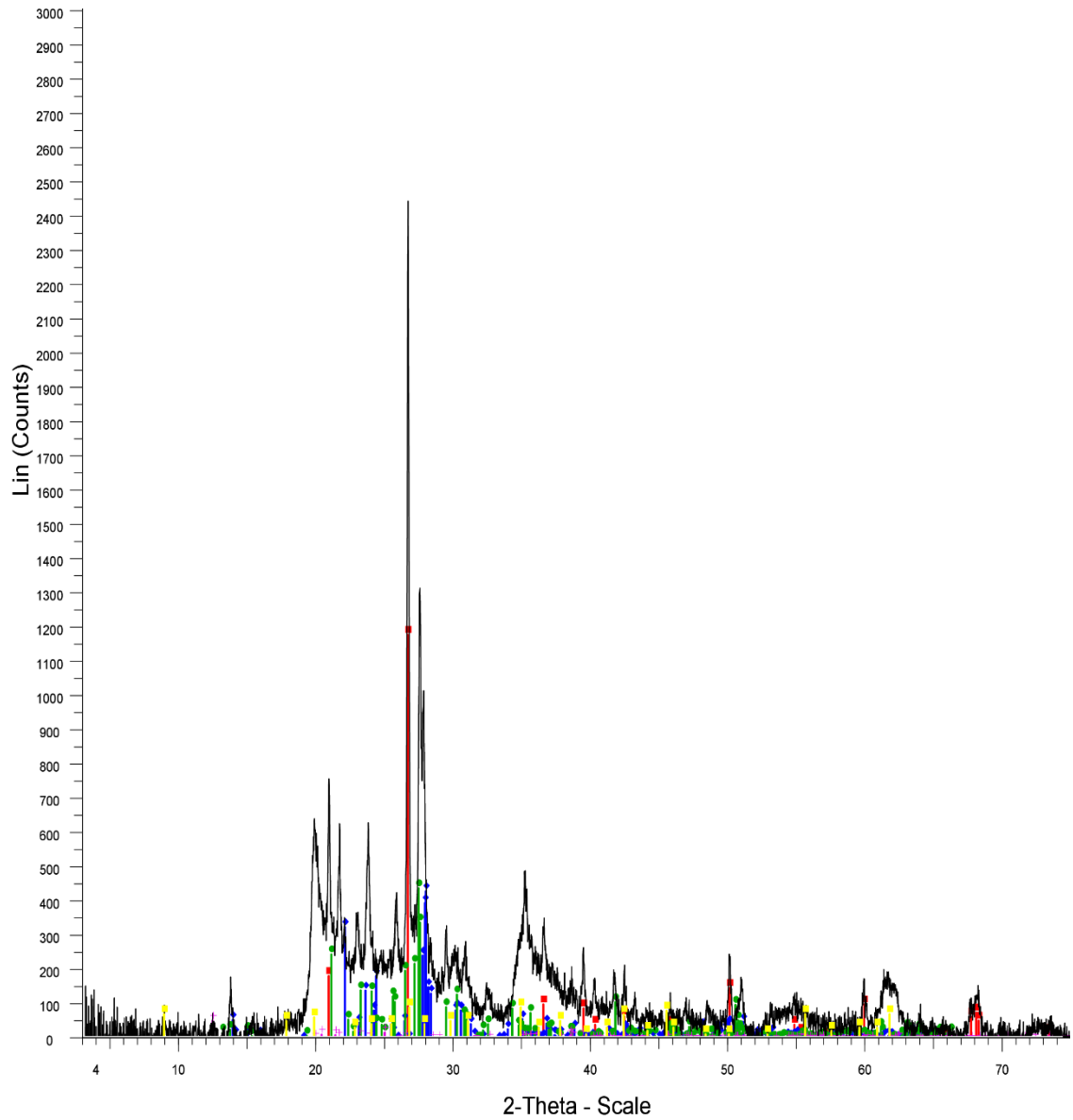


Figure E5: HBC at 500°C



Commander Sample ID - File: icr180883.raw - Type: Locked Coupled - Start: 3.060 ° - End: 75.055 ° - Step: 0.020 ° - Step time: 96. s - Temp.: 25 °C (Room) - Tim
 Operations: Displacement -0.073 | Background 0.000,1.000 | Import
 ■ 00-046-1045 (*) - Quartz, syn - SiO₂ - S-Q 18.0 % - I/Ic PDF 3.4 - F30=558(0.0011,31)
 ◆ 01-080-1094 (A) - Albite, low - Na(AlSi₃O₈) - S-Q 35.3 % - I/Ic PDF 0.6 - F30=169(0.0040,37)
 ● 01-072-1114 (*) - Microcline - KAlSi₃O₈ - S-Q 39.1 % - I/Ic PDF 0.6 - F30= 87(0.0084,42)
 ⊕ 01-075-1593 (D) - Kaolinite 1A - Al₂Si₂O₅(OH)₄ - S-Q 2.5 % - I/Ic PDF 1.2 - F30=1000(0.0007,33)
 ■ 00-002-0467 (D) - Muscovite - KAl₂(Si₃Al)O₁₀(OH,F)₂ - S-Q 5.1 % - I/Ic PDF 1. - F27= 5(0.0343,148)

Figure E6: HBC at 600°C

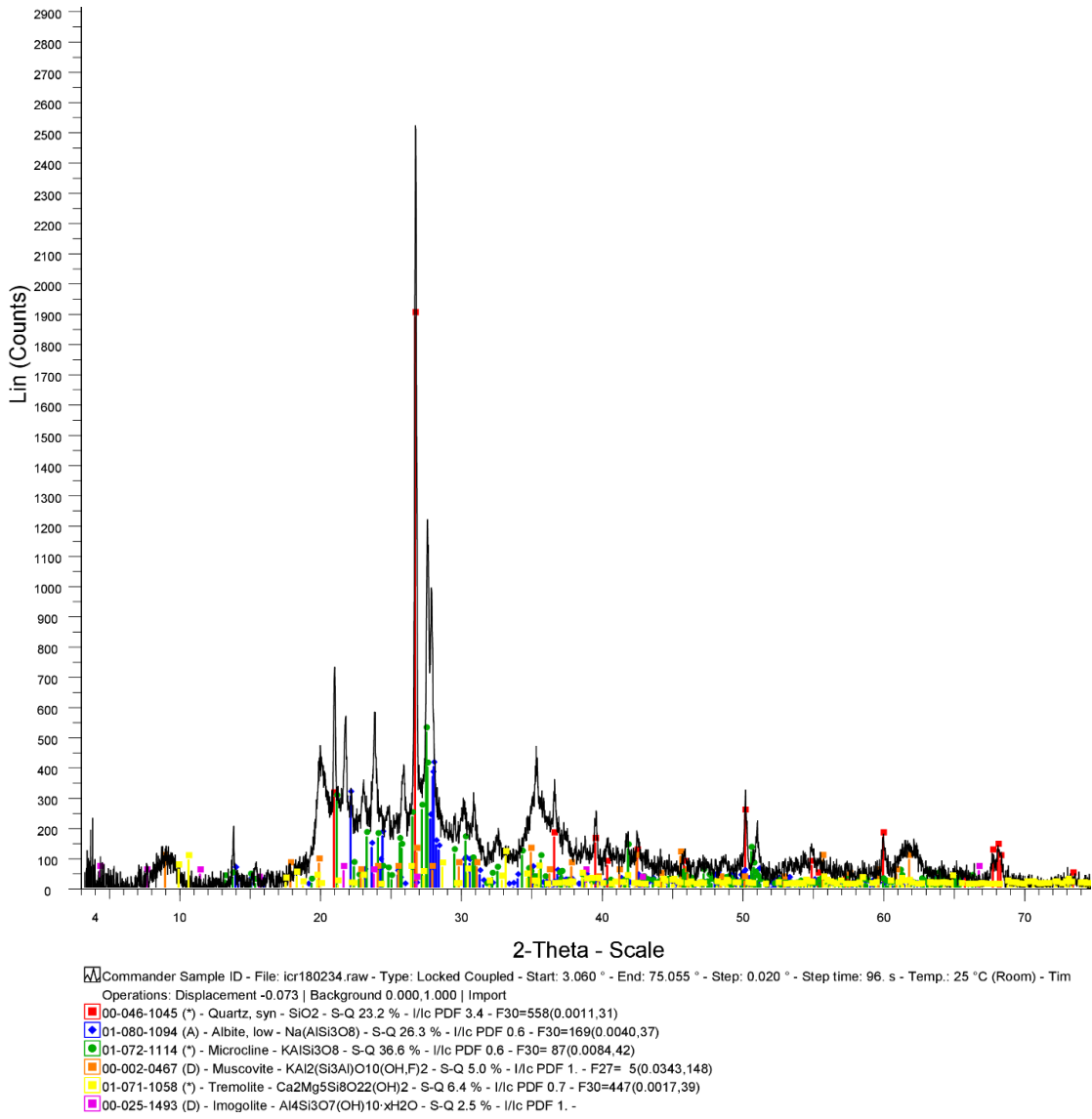


Figure E7: HBC at 700°C

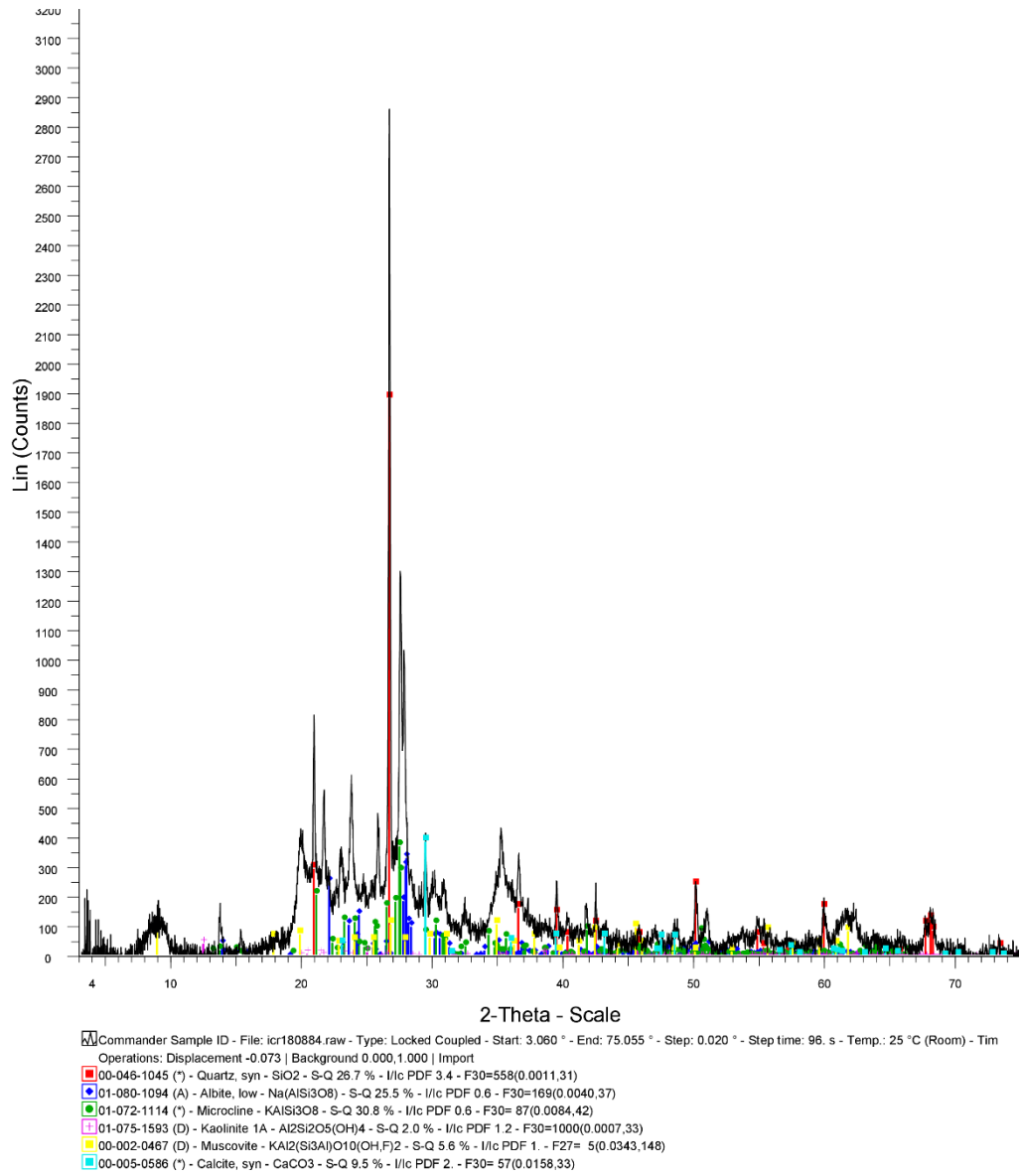
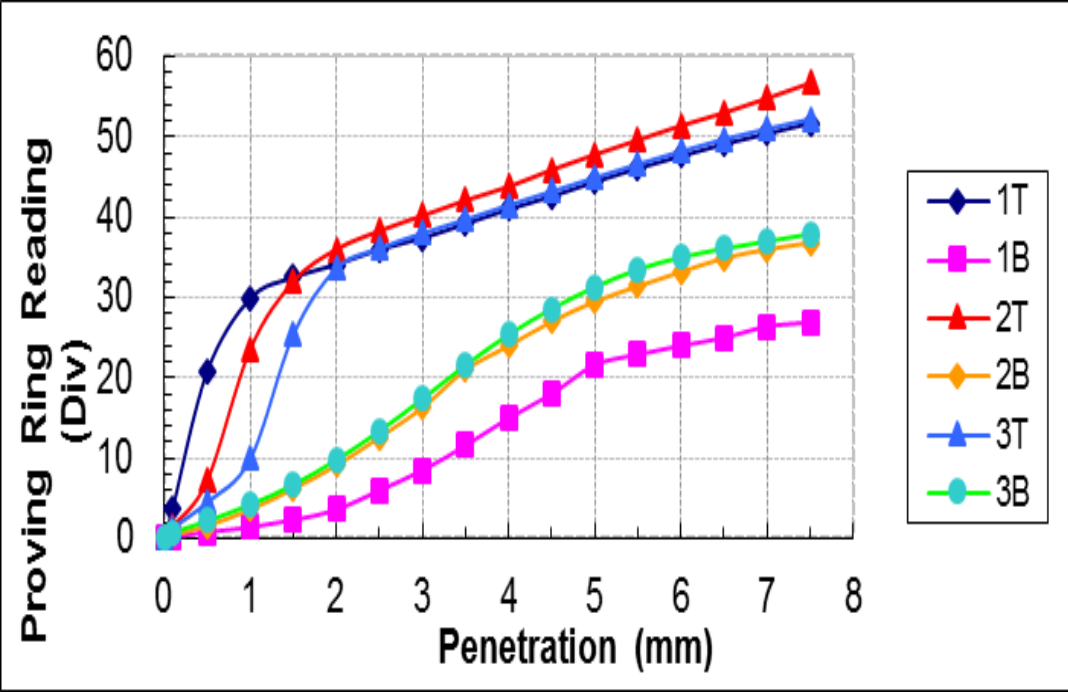


Figure E8: HBC at 800°C

Appendix F: CBR for optimal mix



Appendix G: Particle Size Distribution for varying curing periods

Sieve Size (mm)	7 days	14 days	28 days	56 days	90 days
9.52	100	100	100	100	100
5	99	99	100	91	98.59
3.35	99	98	100	84	98.18
2	98	96	99	74	97.76
1.18	97	93	97	65	97.76
0.6	96	91	95	58	92.3
0.425	94	90	94	55	91
0.3	91	89	91	53	88.2
0.212	89	88	88	51	85
0.15	85	86	85	50	84.01
0.075	36	36	60	33	73.11
0.05	33	29	45	30	69.08
0.038	22	21	43	27	65.62
0.025	21	17	40	23	61.47
0.019	14	14	37	20	43.2
0.013	12	12	34	17	40
0.01	10	11	32	14	33
0.0067	8	8	29	11	24
0.0044	6	6	26	8	18.84
0.002	4	4	24	4	11.57

Appendix H

Selected Trial Results (kN/m² x 100)

Specimen code 200-12-4-4

MC	AGE(DAYS)			
	7	14	21	28
24	0(B)	0(B)	0(B)	0(B)
28	0.322	0.656	0	0.666
32	0.432	0.877	0	1.146
36	0.107	0.165	0	0.054

Specimen code 200-10-6-4

MC	AGE(DAYS)			
	7	14	21	28
24	0	0	0	0
28	0.674	1.022	1.244	1.406
32	0.685	0.726	0.938	1.239
36	0.779	1.055	1.392	1.381

Specimen code 200-16-4-0

MC	AGE(DAYS)			
	7	14	21	28
24	0	0	0	0
28	0.779	0.869	0	1.326
32	0.341	1.049	0	0.951
36	0.579	1.389	0	1.355

Specimen code 200-14-6-0

MC	AGE(DAYS)			
	7	14	21	28
24	0	0	0	0
28	1.034	0.772	1.428	1.995
32	1.456	1.153	1.619	1.91
36	0.978	0.855	1.899	1.496

Specimen code 300-18-2-0

MC	AGE(DAYS)			
	7	14	21	28
24	0.748	0.823	1.368	1.136
28	1.017	0.723	0.851	1.024
32	0.504	0.556	0.681	0.675
36	0.672	0.66	1.181	1.285

Specimen code 300-16-4-0

MC	AGE(DAYS)			
	7	14	21	28
24	0	0	0	0
28	0.461	0.536	1.042	1.253
32	0.414	0.674	0.815	1.15
36	0.625	0.62	0.966	0.96

Specimen code 300-4-6-10

MC	AGE(DAYS)			
	7	14	21	28
24	0	0	0	0
28	0.387	0.639	0.438	1.336
32	0.384	1.088	1.483	0.786
36	0.471	0.763	1.192	1.088

Specimen code 300-6-8-6

MC	AGE(DAYS)			
	7	14	21	28
24	0	0	0	0
28	0.548	0.657	0.82	1.148
32	0.419	0.521	0.743	0.699
36	0.32	0.584	0.954	1.014

Specimen code 400-14-6-0

MC	AGE(DAYS)			
	7	14	21	28
24	0.39	0.27	0.35	0.64
28	0.44	0.22	0	0.16
32	2.04	1.46	1.83	1.13
36	6.6	6.97	4.78	7.98

Specimen code 400-12-8-0

MC	AGE(DAYS)			
	7	14	21	28
24	0	0	0	0
28	0.23	0.22	0	0.24
32	2.38	2.18	1.7	1.71
36	5.76	6	6.46	6.65

Specimen code 400-10-6-4

MC	AGE(DAYS)			
	7	14	21	28
24	0	0.043	0.03	0.04
28	0.15	0.38	0	0.29
32	0.88	1.3	1.8	1.71
36	1.44	1.97	2.23	3.27

Specimen code 400-10-8-2

MC	AGE(DAYS)			
	7	14	21	28
24	0	0	0	0
28	0	0.37	0.27	0.31
32	0.85	0.97	0.91	0.84
36	1.34	2.25	2.66	2.89

Specimen code 500-10-6-4

MC	AGE(DAYS)			
	7	14	21	28
24	0	0	0	0
28	0.68	0.77	1.24	0.83
32	0.65	1.01	1.11	1.46
36	1.2	1.47	1.53	1.89

Specimen code 500-10-8-2

MC	AGE(DAYS)			
	7	14	21	28
24	0	0	0	0
28	0.47	0.61	0.66	0.42
32	0.67	1.28	1.33	1.02
36	1.03	1.37	1.47	1.68

Specimen code 500-14-6-0

MC	AGE(DAYS)			
	7	14	21	28
24	0	0	0	0
28	0.58	0.71	0.64	0.48
32	1.21	1.97	0.94	0.92
36	2.97	3.07	1.68	1.94

Specimen code 500-12-8-0

MC	AGE(DAYS)			
	7	14	21	28
24	0	0	0	0
28	0.34	0.61	0.65	0.77
32	1.23	1.54	0.93	0.99
36	1.28	2.18	1.56	1.68

Specimen code 600-15-0-5

M C	AGE(DAYS)			
	7	14	21	28
			0.6	0.5
24	0.63	0.63	2	2
			0.9	1.0
28	0.67	0.96	9	5
32	1.07	1.11	1.3	1.7
			1.0	1.6
36	0.9	1.12	2	5

Specimen code 600-12-4-4

MC	AGE(DAYS)			
	7	14	22	29
24	0B	0B	0B	0B
				0.2
28	0.23	0.24	0.27	1
				0.8
32	0.81	0.87	1.1	5
36	1.69	1.82	1.55	2.5

Specimen code 600-20-0-0

M C	AGE(DAYS)			
	7	14	21	28
				0.5
24	0.54(6)	0.61	0.5	7
			0.5	0.6
28	0.78(6)	0.65	9	4
			0.5	0.6
32	0.52(6)	0.51	4	2
			0.8	1.1
36	0.59(6)	0.9	8	3

Specimen code 600-16-4-0

MC	AGE(DAYS)			
	7	14	21	28
24	0	0	0	0
				0.1
28	0.18	0.17	0	3
			1.11	0.8
32	0.76	0.83	B	1
	2.47		2.52	3.6
36	B	2.15B	B	8

Specimen code 700-10-6-4

MC	AGE(DAYS)			
	7	14	21	28
24	0	0	0	0
	0.71	0.96		
28	1	7	1.215	1.288
	0.95	1.22		
32	4	4	1.35	2.005
	1.43	1.37		
36	9	7	1.734	2.224

Specimen code 700-10-8-2

MC	AGE(DAYS)			
	7	14	21	28
24	0	0	0	0
		0.77	0.91	
28	0.434	3	8	0.733
		1.25	1.15	
32	0.75	4	8	1.414
		1.56		
36	1.144	1	1.96	2.23

Specimen code 700-14-6-0

MC	AGE(DAYS)			
	7	14	21	28
24	0	0	0	0
	0.36			
28	7	0.667	0.617	0.515
	0.89			
32	5	1.149	0.81	0.737
	3.35			
36	1	2.625	2.106	2.029

Specimen code 700-12-8-0

MC	AGE(DAYS)			
	7	14	21	28
24	0	0	0	0
		0.97		1.01
28	0.96	6	0.873	8
	1.86			1.61
32	5	2.32	1.281	2
	3.31	3.05		3.88
36	8	8	3.629	7

Appendix I: Swelling Characteristics for various curing periods

Curing Period (days)	Atterberg Limits				Free Swell (%)
	WL	WP	IP	LS	
7	68	38	30	15	81
14	65	37	28	19	70
28	64	32	24	16	49
56	71	38	33	19	39
90	64	39	25	19	49

Appendix J: XRD results for long term curing

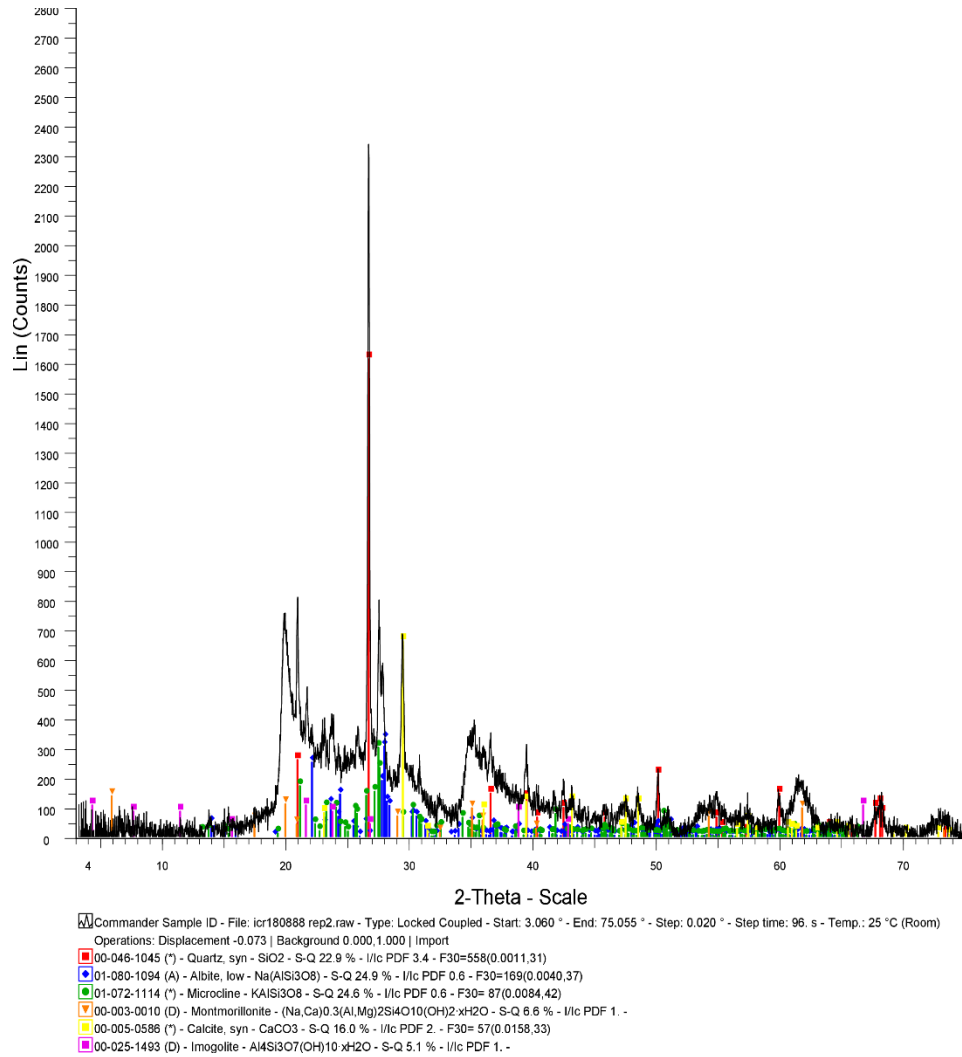
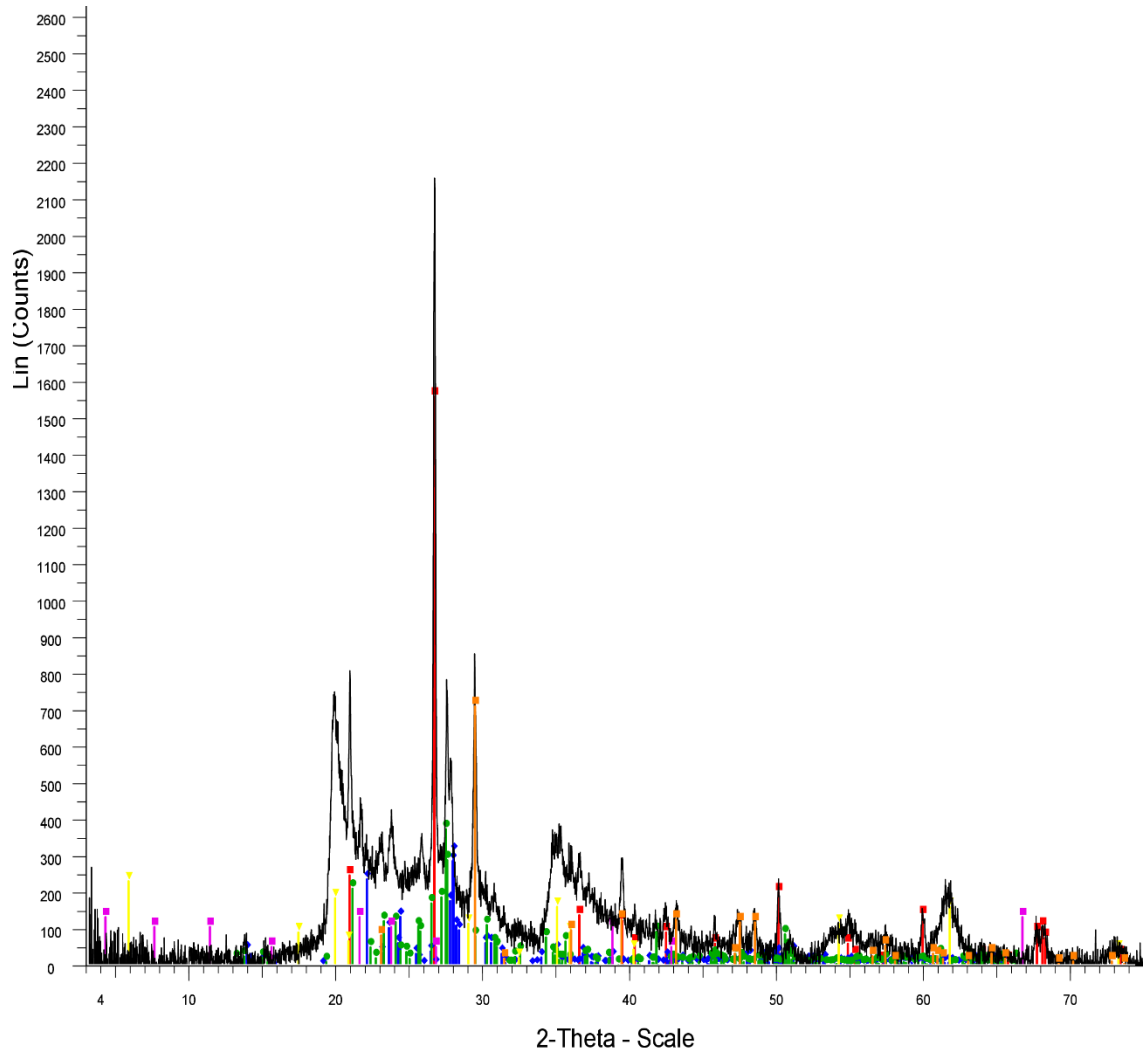


Figure J1: Stabilized, 7 days Cure



Commander Sample ID - File: icr180889 rep2.raw - Type: Locked Coupled - Start: 3.060 ° - End: 75.055 ° - Step: 0.020 ° - Step time: 96. s - Temp.: 25 °C (Room)
 Operations: Displacement -0.073 | Background 0.000,1.000 | Import
 00-046-1045 (*) - Quartz, syn - SiO₂ - S-Q 19.8 % - I/Ic PDF 3.4 - F30=558(0.0011,31)
 01-080-1094 (A) - Albite, low - Na(AlSi₃O₈) - S-Q 21.2 % - I/Ic PDF 0.6 - F30=169(0.0040,37)
 01-072-1114 (*) - Microcline - KAlSi₃O₈ - S-Q 27.6 % - I/Ic PDF 0.6 - F30= 87(0.0084,42)
 00-003-0010 (D) - Montmorillonite - (Na,Ca)0.3(Al,Mg)2Si₄O₁₀(OH)₂·xH₂O - S-Q 10.1 % - I/Ic PDF 1. -
 00-005-0586 (*) - Calcite, syn - CaCO₃ - S-Q 15.4 % - I/Ic PDF 2. - F30= 57(0.0158,33)
 00-025-1493 (D) - Imogolite - Al₄Si₃O₇(OH)₁₀·xH₂O - S-Q 5.9 % - I/Ic PDF 1. -

Figure J2: Stabilized, 14 days Cure

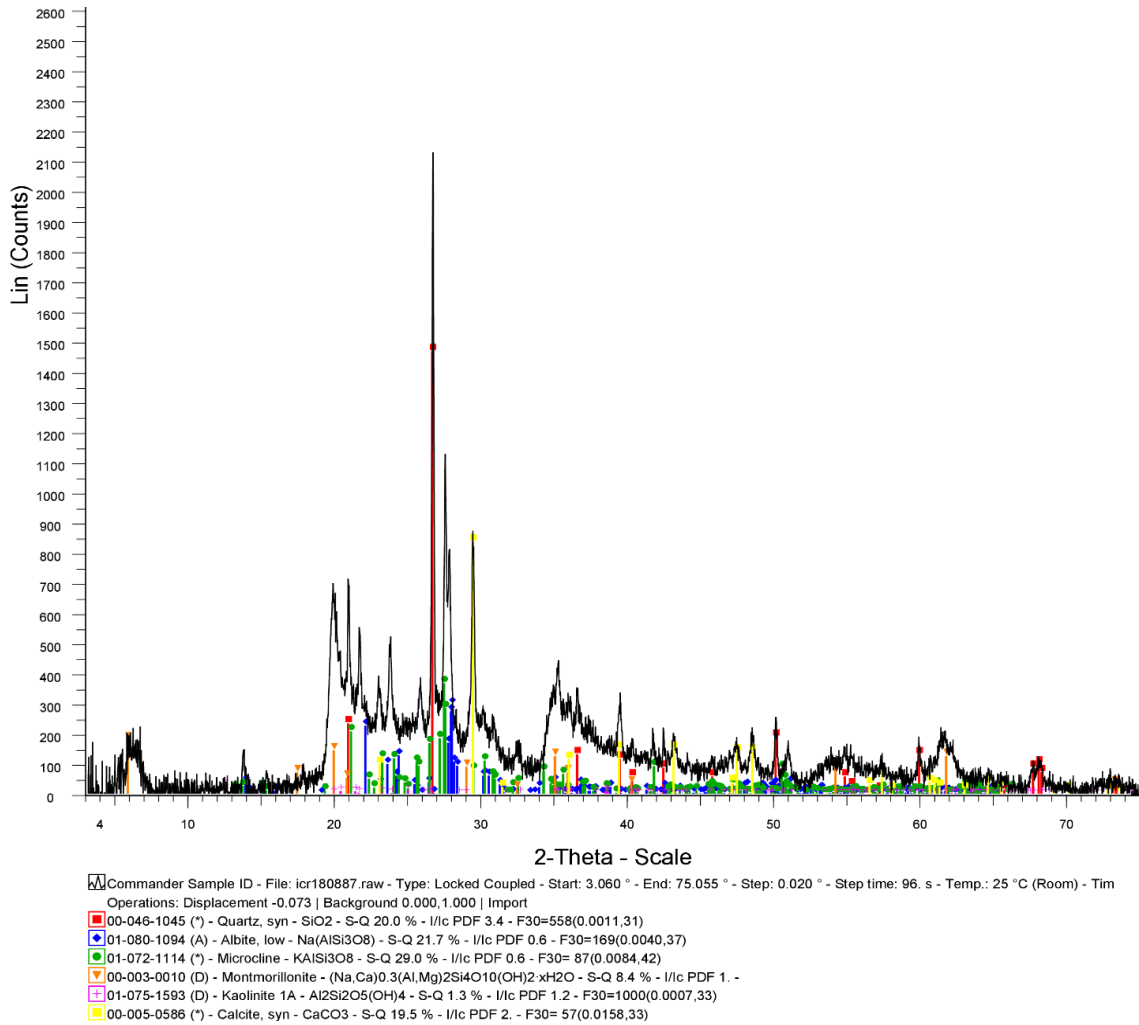


Figure J3: Stabilized, 28 days Cure

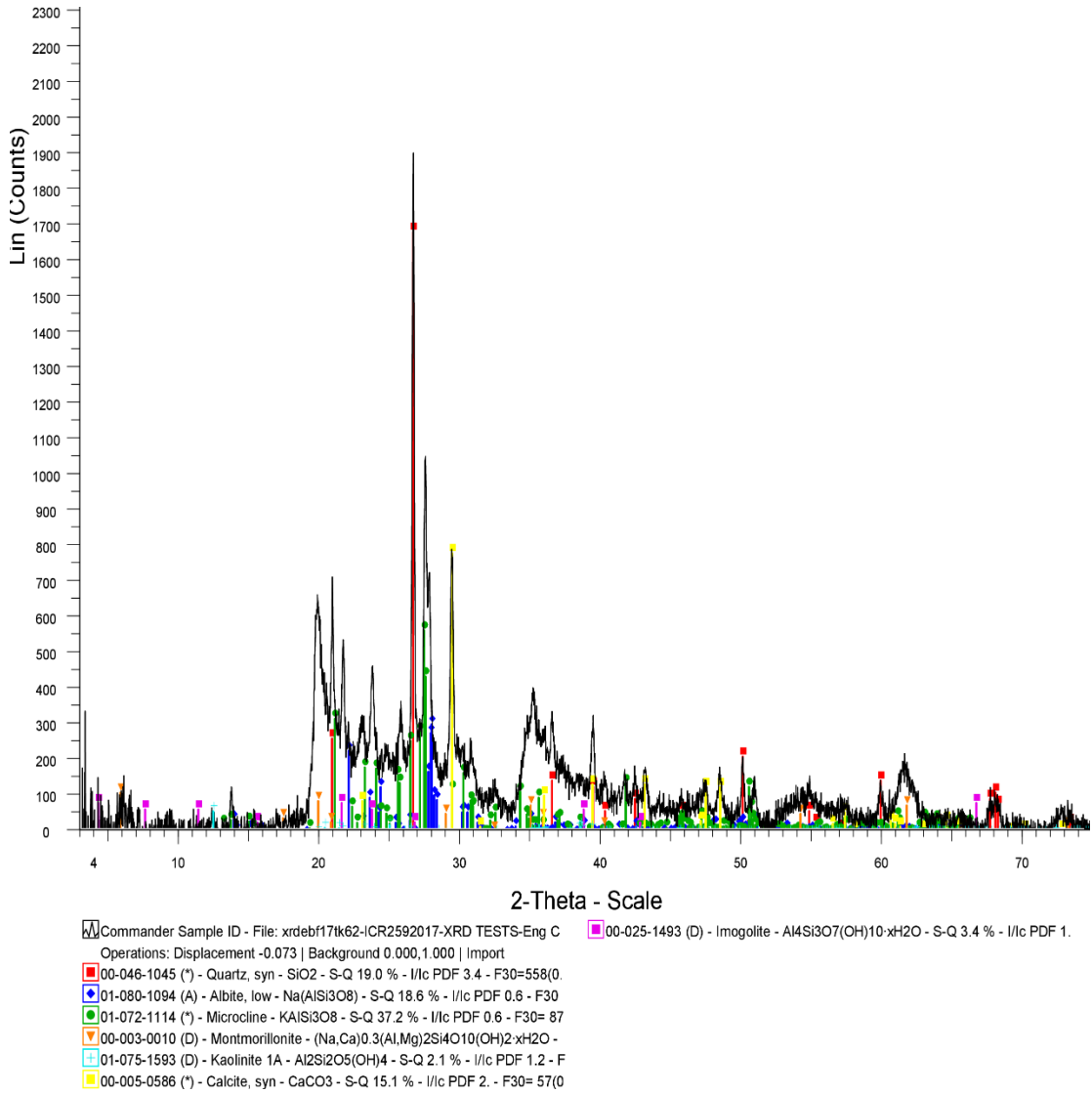
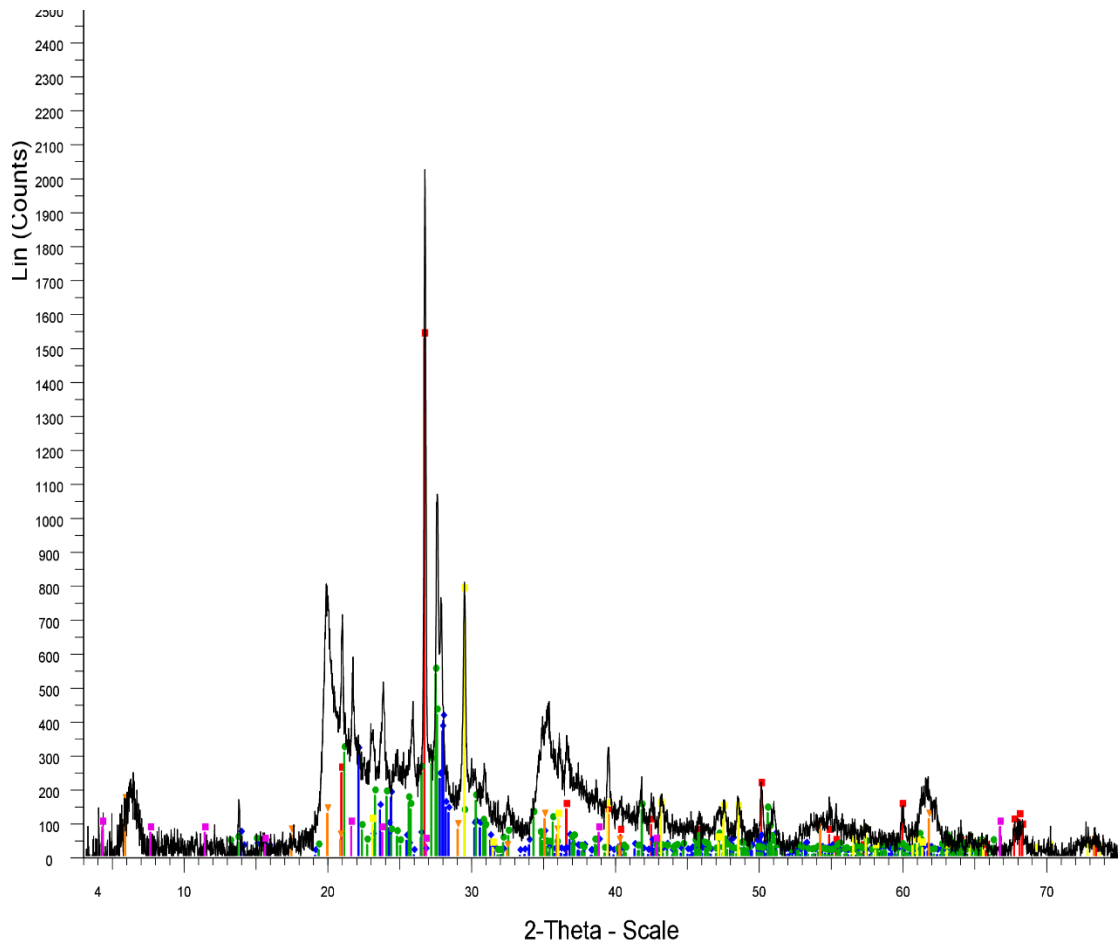


Figure J4: Stabilized, 56 days Cure



Commander Sample ID - File: xrd eod89idv25-ICR2772017-XRD TESTES-Eng Kabubo-Mwihoko-E.AFRICA-Kenya-Soil-ndfsf-1-90----Mwihoko--ICRAF.raw - Type:
 Operations: Displacement -0.073 | Background 0.000,1.000 | Import
 ■ 00-046-1045 (*) - Quartz, syn - SiO₂ - S-Q 17.2 % - I/Ic PDF 3.4 - F30=558(0.0011,31)
 ◆ 01-080-1094 (A) - Albite, low - Na(AlSi₃O₈) - S-Q 23.9 % - I/Ic PDF 0.6 - F30=169(0.0040,37)
 ● 01-072-1114 (*) - Microcline - KAlSi₃O₈ - S-Q 34.9 % - I/Ic PDF 0.6 - F30= 87(0.0084,42)
 ▼ 00-003-0010 (D) - Montmorillonite - (Na,Ca)0.3(Al,Mg)2Si₄O₁₀(OH)₂·xH₂O - S-Q 5.9 % - I/Ic PDF 1. -
 ▲ 00-005-0586 (*) - Calcite, syn - CaCO₃ - S-Q 14.8 % - I/Ic PDF 2. - F30= 57(0.0158,33)
 ■ 00-025-1493 (D) - Imogolite - Al₄Si₃O₇(OH)₁₀·xH₂O - S-Q 3.3 % - I/Ic PDF 1. -

FigureJ5: Stabilized, 90 days Cure

Appendix K: UCS Results for long term cure and simulated flooding regimes, at OMC

Legend:

4P- 4 days partial soak, 4F – 4 days full soak (submerged), SRI – Strength reduction index.

Table K1: 7 days cure UCS results

Reference	Peak Stress Readings (kN/m ²)x10 ²				Average Stress (kN/m ²)x10 ²	UCS (kN/m ²)x10 ²	SRI (%)
Un-soaked	2.1	2.748	2.135	2.521	2.12	1.06	100
4P	1.59	3.286	3.395	2.787	1.79	1.40	132
4F	3.037	1.444	1.78	2.316	2.32	1.16	109
10P	4.578	3.275	2.659	2.673	2.67	1.33	125
10F	2.799	2.161	3.129	4.319	2.8	1.40	132

Table K2: 14 days cure UCS results

Reference	Peak Stress Readings (kN/m ²)x10 ²				Average Stress (kN/m ²)x10 ²	UCS (kN/m ²)x10 ²	SRI (%)
Un-soaked	2.283	3.162	2.457	2.502	2.48	1.24	100
4P	2.586	2.916	2.895	2.998	2.94	1.47	119
4F	2.64	1.745	2.000	1.437	3.75	1.87	151
10P	2.76	2.898	4.386	4.438	2.83	1.41	114
10F	3.478	2.541	2.554	2.207	2.55	1.27	102

Table K3: 28 days cure UCS results

Reference	Peak Stress Readings (kN/m ²)x10 ²				Average Stress (kN/m ²)x10 ²	UCS (kN/m ²)x10 ²	SRI (%)
Un-soaked	2.045	2.343	2.326	3.25	2.33	1.17	100
4P	2.357	3.012	3.326	4.441	3.17	1.58	135
4F	1.831	2.013	2.725	2.033	2.02	1.01	86
10P	3.722	2.796	3.694	3.955	3.68	1.84	157
10F	2.454	3.344	3.304	2.155	2.28	1.14	97

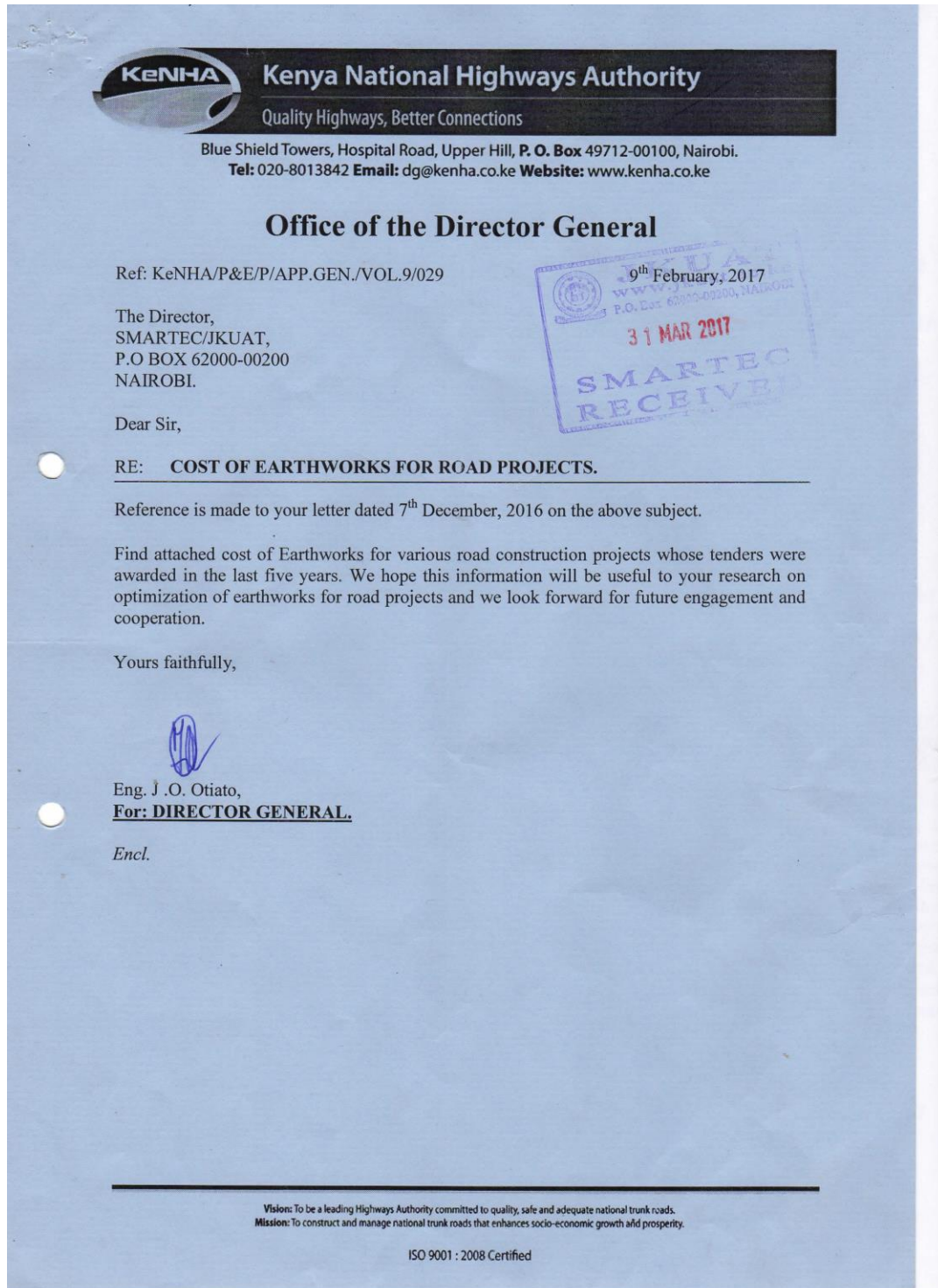
Table K4: 56 days cure UCS results

Reference	Peak Stress Readings (kN/m ²)x10 ²				Average Stress (kN/m ²)x10 ²	UCS (kN/m ²)x10 ²	SRI (%)
Un-soaked	1.357	1.948	1.971	1.787	1.79	0.9	100
4P	2.023	2.564	2.007	1.947	1.99	1.0	110
4F	1.512	1.456	1.46	1.33	1.46	0.73	81
10P	2.797	1.440	1.578	2.351	1.51	0.75	83
10F	1.756	1.799	2.595	1.575	1.78	0.89	99

Table K5: 90 days cure UCS results

Reference	Peak Stress Readings (kN/m ²)x10 ²				Average Stress (kN/m ²)x10 ²	UCS (kN/m ²)x10 ²	SRI (%)
Un-soaked	2.935	4.462	5.299	5.688	4.46	2.23	100
4P	3.717	3.192	3.554	3.105	3.15	1.57	70
4F	3.113	2.241	3.796	3.198	3.16	1.58	71
10P	2.909	2.663	3.174	2.885	2.77	1.39	62
10F	2.589	4.210	1.728	1.827	1.78	0.89	40

Appendix L: Earthwork rates.



SUMMARY - AVERAGE VALUES OF RATES FOR DIFFERENT WORK ITEMS.			
ITEM	DESCRIPTION	UNIT	RATE
1.01	Fill in soft material to 95% MDD (AASHTO T99) in layers of 150mm each in accordance with Cl 502, 504, 505, 508, 511, 512 & 513 of the specs.	m3	454
1.02	fill in hard material to 95% MDD (AASHTO T99) in layers of 150mm each in accordance with Cl 502, 504, 505, 508, 511, 512 & 513 of the specs.	m3	1166
1.03	Cut to spoil in soft material in accordance with CL. 502, 504, 510, 511 & 513 of the specs. Includes excavation in swampy areas.	m3	193
1.04	Cut to spoil in hard material in accordance with CL. 502, 504, 510, 511 & 513 of the specs. Includes excavation in swampy areas.	m3	892
1.05	Compaction of 150mm thick original ground under fills and in cuts below subgrade to 95% MDD. (AASHTO T99) in accordance with Cl. 502,504,508 & 512 of the specs	m3	233
1.06	Extra over item 5.01 for compaction of top 300mm below formation levels in cuts and fills (subgrade) to 100% MDD. (AASHTO T99)	m3	65.8
1.07	Provide geotextile fibre to under, over or around rockfill as directed by the Engineer	m2	283
1.08	Top soiling over side slopes, cuts and fills. Cl.514 of the specs	m2	25.5
1.09	Grass over side slopes in fills and cuts	m2	37.7
1.10	Provide and place rockfill to swampy areas as directed by the Engineer, Cl.507 of the specs.	m3	1183
1.11	Mixing on cement/lime stabiliser into natural gravel or GCS	m3	317
1.12	Curing and protection of treated layers	m2	86

Appendix M: Sample Test Certificates

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MINISTRY OF TRANSPORT AND INFRASTRUCTURE
STATE DEPARTMENT OF INFRASTRUCTURE

Telegraphic Address: "MINWORKS", Nairobi
Telephone: Nairobi 554950/3/4 Fax: 554877
E-mail: chief.engineer@materials.go.ke
If calling or telephoning ask for
When replying please quote

MATERIALS TESTING AND RESEARCH DIVISION
MACHAKOS ROAD
INDUSTRIAL AREA
P. O. Box 11873-00400
NAIROBI

Ref No:
Date:

M.4936./35/N/134
6 October 2014

**JKUAT SMARTEC
P.O BOX 6200
NAIROBI.**

JOB CARD NO: 00218/F/14

SOIL TEST RESULTS- JKUAT SMARTEC (JBC)

SAMPLE NO	REF	PLASTICITY					GRADING				
		LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTIC INDEX (%)	LINEAR SHRINKAGE (%)	GRAVEL %>=2mm	SAND % 0.006-2MM	SILT % 0.002-0.063MM	CLAY <0.002MM	FREE SWELL %	SPECIFIC GRAVITY
76/14/F	A	63	35	28	14	2	29	33	36	70	2.46
"	B	64	34	30	15	2	24	32	42	75	2.50
"	C	62	34	27	14	2	31	32	35	80	2.46
"	D	62	33	29	14	2	28	34	36	70	2.47

CHIEF ENGINEER MATERIALS
MINISTRY OF TRANSPORT AND INFRASTRUCTURE
P. O. Box 11873-00400
CHIEF ENGINEER (MATERIALS)

Sampled and Submitted by: Client





**MINISTRY OF TRANSPORT & INFRASTRUCTURE
STATE DEPARTMENT OF INFRASTRUCTURE**

Telegraphic Address: "MINWORKS", Nairobi
Telephone: Nairobi 554950/3/4
Fax: 554877
E-mail: chief.engineer@materials.go.ke

Materials Testing & Research Division
Machakos Road, Industrial Area
P.O. Box 11873 – 00400
NAIROBI

Ref No. M.7682/35/L/1

Date: 21 March, 2017

Laboratory Test Report

- | | |
|---|---|
| 1. Customer's Address: JKUAT/SMARTTECH | |
| 2. E-mail: | Phone: |
| 3. Customer Contact: BOX 62000 NBI | 6. Job Card No. 002217/S/15 |
| 4. Sample Description: Black Cotton Soil
(Subjected to 600° C Temp) | 7. Date fee paid: 19/01/2016 |
| 5. Sample Submitted by: Engineer Kabubo | 8. GOK MR No. 5250171 |
| 6. Date of Sample received: January 2016 | 9. Date Analysis started: January 2016 |

10. Additional information provided by the customer:

TEST RESULTS																
Sample No	Reference	Grading						Atterberg Limits					Compaction T 99		CBR 4 days Soak (%)	Swell (%)
		% Passing BS Sieve Size (mm)						LL %	PL %	PI %	LS %	PM	MDD Kg/m3	OMC (%)		
		20	10	5	2	0.425	0.075									
	Black Cotton (M6 600°C)	100	100	100	77	53	52	46	26	20		1060	1460	29.0	3	1.8
	Black Cotton (M6 600°C)	100	100	100	77	54	53	47	24	23		1242	1460	29.0	3	2.2
	Black Cotton (M6 600°C)	100	100	100	78	54	53	47	27	20		1080	1460	29.0	4	1.4
	Black Cotton (M6 600°C)	100	100	100	77	53	52	46	26	20		1060	1460	29.0	3	1.8

Eng Charles M. Muriuki
For: CHIEF ENGINEER (MATERIALS)
OIC Soils Laboratory





**MINISTRY OF TRANSPORT, INFRASTRUCTURE, HOUSING & URBAN DEVELOPMENT
STATE DEPARTMENT OF INFRASTRUCTURE**

Telegraphic Address: "MINWORKS", Nairobi
Telephone: Nairobi 554950/3/4
Fax: 554877
E-mail: chief.engineer@materials.go.ke

Materials Testing & Research Division
Machakos Road, Industrial Area
P.O. Box 11873 – 00400
NAIROBI

Ref No. M.12400 /35/L/1

Date: 08th May, 2017

Laboratory Test Report

- | | |
|---|---|
| 1. Customer's Address: Eng. Charles K. Kabubo
E-mail: kabubo200@gmail.com | Phone: 0722 825927 |
| 2. Customer Contact: Box 1004 – 00618, Nairobi | 6. Job Card No. 1255/S/2017 |
| 3. Sample Description: Pre-Heated Soil (700°C) | 7. Date fee paid: 03/01/2017 |
| 4. Sample Submitted by: Client | 8. GOK MR No. 0082702 |
| 5. Date of Sample received: 03/01/2017 | 9. Date Analysis started: 09/01/17 |

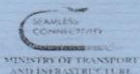
10. **Additional information provided by the customer:** Pre -Heated Soil to 700°C

TEST RESULTS																
Sample No	Reference	Grading						Atterberg Limits					Compaction T 99		CBR 4days Soak (%)	Swell (%)
		% Passing BS Sieve Size (mm)						LL (%)	PL (%)	PI (%)	LS (%)	PM (%)	MDD (kg/m ³)	OMC (%)		
		20	10	5	2	0.425	0.075									
529/S/17	Heated Soil 700°C							Non - Plastic					1400	22.0	16	0.5
								Non - Plastic							18	0.2
								Non - Plastic							23	0.1
								Non - Plastic							13	0.8

CHIEF ENGINEER MATERIALS
MINISTRY OF TRANSPORT AND INFRASTRUCTURE
P. O. Box 11873 / 00400
NAIROBI

Eng. Charles M. Muriuki

For: CHIEF ENGINEER (MATERIALS)



3



**MINISTRY OF TRANSPORT AND INFRASTRUCTURE
MATERIALS TESTING & RESEARCH DEPARTMENT**

P.O.Box 11873 - 00400 NAIROBI. Tel 5549503/4, Fax 554877, e-mail chief.engineer@materials.go.ke

**Test Certificate for Soil samples
(Testing as per BS 1377:1990 & 1924)**

Ref: M.6489/35/L/1
Date: 19th June, 2015

NOTIFICATIONS/MS/MD 038 (Rev 2)

CLIENT: JKUAT/SMARTTECH
ADDRESS: P.O BOX 62000 NAIROBI
Email:

Phone:

Sample No	Reference	Grading						Atterberg Limits				Compaction T 99				CBR (%)		
		20 mm	10 mm	5 mm	2mm	425 µm	75 µm	LL (%)	PL (%)	PI (%)	LS (%)	PM (%)	MDD (Kg/m ³)	OMC (%)	MDD (Kg/m ³)		OMC (%)	
804/S/15	M8-800°C	100	100	100	100	48	15	6	NON PLASTIC				Collapsed					
805/S/15	M8-800°C	100	100	100	100	44	13	5										
806/S/15	M8-800°C	100	100	100	100	43	13	7										
807/S/15	M8-800°C	100	100	100	100	43	13	6										
<p align="center">  CHIEF ENGINEER MATERIALS MINISTRY OF TRANSPORT AND INFRASTRUCTURE P.O BOX 11873 - 00400 NAIROBI CHIEF ENGINEER (MATERIALS) </p>																		
SPSE (PDC)																		

PROJECT NAME :
Date of sampling : Not provided
Sampling Location : Not indicated
Sampled & Submitted By : Client

JOB CARD No.002217/S/15
RECEIPT No. 5250171 of 23/03/15
AMOUNT PAID: KSh 99,000/=



**MINISTRY OF TRANSPORT, INFRASTRUCTURE, HOUSING AND URBAN DEVELOPMENT.
STATE DEPARTMENT OF INFRASTRUCTURE**

Telegraphic Address: "MINWORKS", Nairobi
Telephone: Nairobi 554950/3/4
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Materials Testing & Research Division
Machakos Road, Industrial Area
P.O. Box 11873 – 00400
NAIROBI

Ref No. **M.234/35/N/141**

Date: **25 July, 2017**

Laboratory Test Report

- | | |
|---|---|
| 1. Customer's Address: ENG CHARLES K.KABUBO, P O Box 62000 - 00200 | 6. Job Card No. 6125/F/15 |
| E-mail: N/A | Phone: - |
| 2. Customer Contact: 07222825927 | |
| 3. Sample Description: Soil sample – (Stabilised 7 day cure) | 7. Date fee paid: 12/06/17 |
| 4. Sample Submitted by: Client | 8. GOK MR No.- 0105840 |
| 5. Date of Sample receipt: 12/06/ 2017 | 9. Date Analysis started: 5/7/2017 |

SPECIFIC GRAVITY OF FINE GRAINED SOILS

Container No.		2622	B552	B554	B552	2610	2604
Mass of Bottle (m_1)	g	27.9	33.8	33.0	34.7	29.1	28.8
Mass of Bottle + Dry soil (m_2)	g	37.9	44.1	43.0	44.8	39.0	38.9
Mass of dry soil ($m_s = m_2 - m_1$)	g	10.0	10.3	10.0	10.1	9.9	10.1
Mass of Bottle + Soil + Water (m_3)	g	84.70	138.80	139.80	140.30	84.10	84.60
Mass of Bottle + Water (m_4)	g	78.70	132.50	133.60	134.20	78.00	78.40
Mass of water replaced ($m_w = m_s - m_3 + m_4$)	g	4.0	4.0	3.8	4.0	3.8	3.9
Temperature	T1°C	20	20	20	20	20	20
S.G. of Soil at T1°C ($G_s(T1) = \frac{m_s}{m_w}$)		2.50	2.58	2.63	2.53	2.61	2.59
C1 at T1°C	C1	1.00	1.00	1.00	1.00	1	1
S.G. of Soil at 20°C, ($G_s(20) = C1 \times G_s(T1)$)		2.50	2.58	2.63	2.53	2.61	2.59
Average	(Gs)	2.54		2.58		2.60	

CHIEF ENGINEER MATERIALS
MINISTRY OF TRANSPORT AND INFRASTRUCTURE
P. O. Box 11873 - 00400
NAIROBI

Eng. Charles M. Muriuki

For: CHIEF ENGINEER (MATERIALS)

Page 1 of 1





Received
14/01/2016
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**MINISTRY OF TRANSPORT AND INFRASTRUCTURE
STATE DEPARTMENT OF INFRASTRUCTURE**

Telegraphic Address: "MINWORKS", Nairobi
Telephone: Nairobi 554950/3/4 Fax: 554877
E-mail: chief.engineer@materials.go.ke
If calling or telephoning ask for
When replying please quote

MATERIALS TESTING AND RESEARCH DIVISION
MACHAKOS ROAD
INDUSTRIAL AREA
P.O. Box 11873
NAIROBI

Ref. No. M.7529/35/F/82/937
And date

30 December, 2015

Jomo Kenyatta University of Agriculture & Technology
P.O. Box 62000-00100

NAIROBI

Attn: Eng. C. K. Kabubo

CHEMICAL ANALYSIS OF SOIL (REPEAT)
JOB CARD No.001091/CH/15

The samples you submitted on 2/12/2015 gave the results shown below:-

Our Ref	Your Ref	Organic Content, % m/m
1056/Ch/15	400°C A	0.95
1057/Ch/15	400°C B	1.26
1058/Ch/15	400°C C	0.95
1059/Ch/15	400°C D	1.26
1060/Ch/15	600°C A	0.64
1061/Ch/15	600°C B	0.96
1062/Ch/15	600°C C	0.63
1063/Ch/15	600°C D	0.96
1064/Ch/15	800°C A	0.64
1065/Ch/15	800°C B	0.64
1066/Ch/15	800°C C	0.65
1067/Ch/15	800°C D	0.64
1068/Ch/15	1,000°C A	0.32
1069/Ch/15	1,000°C B	0.64
1070/Ch/15	1,000°C C	0.31
1071/Ch/15	1,000°C D	0.32
1072/Ch/15	1,200°C A	0.32
1073/Ch/15	1,200°C B	0.63
1074/Ch/15	1,200°C C	0.32
1075/Ch/15	1,200°C D	0.32

Your account of Kshs.10, 000/= has been settled vide receipt No. MR.7927537 dated 3/12/2015.
Yours faithfully,

CHIEF ENGINEER MATERIALS
MINISTRY OF TRANSPORT AND INFRASTRUCTURE
P.O. Box 11873-00400
NAIROBI

Rebecca K. Angwenyi
For: CHIEF ENGINEER (MATERIALS)



2483



**MINISTRY OF TRANSPORT AND INFRASTRUCTURE
STATE DEPARTMENT OF INFRASTRUCTURE**

Telegraphic Address: "MINWORKS", Nairobi
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If calling or telephoning ask for
When replying please quote

MATERIALS TESTING AND RESEARCH DIVISION
MACHAKOS ROAD
INDUSTRIAL AREA
P. O. Box 11873
NAIROBI

Ref. No. M.6092/35/F/82/801

And date

21 April, 2015

Eng. C.K. Kabubo
JKUAT/SMATECH
P.O. Box 62000
NAIROBI

Dear Sir,

**CHEMICAL ANALYSIS OF SOIL SAMPLES
JOB CARD NO.001729/CH/15**

The samples you submitted on 23/03/2015 gave the results shown below:-

Our Ref	Your Ref	pH Value	Chlorides as Cl, % m/m	Organic Content % m/m	SiO ₂ % m/m	Al ₂ O ₃ % m/m	Fe ₂ O ₃ % m/m	K ₂ O % m/m	MnO % m/m	CaO % m/m	TiO ₂ % m/m
169/Ch/15	M4a	7.28	0.012	0.612	39.39	25.76	19.56	8.80	2.69	1.98	1.24
170/Ch/15	M4b	7.33	0.011	0.489	39.14	27.13	19.03	8.45	2.71	1.90	1.10
171/Ch/15	M4c	7.38	0.011	0.856	39.30	26.14	19.19	9.44	2.25	1.95	1.11
172/Ch/15	M4d	7.40	0.009	0.734	39.34	25.30	19.57	9.10	2.77	2.07	1.19
173/Ch/15	M6a	7.14	0.009	1.222	40.74	26.47	17.79	8.17	3.12	2.12	1.07
174/Ch/15	M6b	7.33	0.012	0.245	40.01	25.75	18.83	9.08	2.74	1.99	1.02
175/Ch/15	M6c	7.10	0.012	0.856	41.14	24.78	18.47	8.87	3.56	1.87	0.80
176/Ch/15	M6d	7.10	0.011	0.611	39.57	25.69	18.33	10.32	2.92	1.78	0.91
177/Ch/15	M8a	7.37	0.012	0.367	40.93	25.92	17.56	8.42	3.34	2.54	0.81
178/Ch/15	M8b	6.96	0.014	0.245	41.09	25.53	17.90	7.86	4.49	1.77	0.92
179/Ch/15	M8c	7.18	0.039	0.367	39.75	24.72	18.38	9.62	3.43	2.34	1.09
180/Ch/15	M8d	7.43	0.009	0.245	39.92	24.81	18.93	9.74	2.82	2.21	1.13
181/Ch/15	M10a	9.56	0.011	0.857	40.58	26.48	17.18	9.04	2.81	2.40	0.99
182/Ch/15	M10b	9.78	0.011	0.978	41.54	24.19	18.12	9.31	2.89	2.20	1.13
183/Ch/15	M10c	9.22	0.021	0.489	40.95	27.71	16.49	8.56	3.11	1.69	1.00
184/Ch/15	M10d	8.79	0.014	0.612	40.68	25.91	17.67	9.62	2.73	1.92	1.04
185/Ch/15	M12a	10.50	0.014	0.367	41.92	23.64	18.37	8.99	3.39	2.02	1.12
186/Ch/15	M12b	9.03	0.007	0.245	41.73	26.41	16.88	8.57	3.13	1.65	1.08
187/Ch/15	M12c	9.63	0.011	0.489	40.50	25.73	17.48	9.89	2.98	1.74	1.07
188/Ch/15	M12d	9.93	0.012	0.366	40.79	25.77	17.48	9.64	2.67	2.04	1.14

Key:

SiO₂-Silicon (IV) Oxide Al₂O₃- Aluminium (III) Oxide Fe₂O₃-Iron (III) Oxide K₂O-Potassium Oxide
MnO- Manganese (II) Oxide CaO-Calcium Oxide TiO₂-Titanium (IV) Oxide



O.K.

MINISTRY OF MINING



e-mail:cg@mining.go.ke
When replying please quote ref No & date
Ref. No.ORIGINAL CERT NO.6334-6342/16 & 464-68/17

MINISTRY OF MINING
MACHAKOS ROAD
P.O. Box 30009-00100 GPO
NAIROBI

Date...17th February, 201

ASSAY CERTIFICATE

SENDER'S NAME : ENG. KABUBO
DATE : 22.11.2016 & 20-01-201
SAMPLE TYPE : CLAYS
SAMPLE NO : 6334,6338, 6342/16 & 464-68/17

RESULTS

Lab No.	Sender's Ref.	% LOI AT (1000°C)
6334/16	200 ^o C 1	11.83
6338/16	300 ^o C 1	10.18
6342/16	500 ^o C 1	9.20
464/17	NEAT	12.80
465/17	400 ^o C	10.05
466/17	600 ^o C	7.22
467/17	700 ^o C	6.45
468/171	800 ^o C	5.18

The results are expressed in percentages (%) unless otherwise indicated.

Joram W. Katweo

JORAM W. KATWEO
FOR: DIRECTOR OF GEOLOGICAL SURVEYS

FOR DIRECTOR OF
GEOLOGICAL SURVEYS
17 FEB 2017
P.O. Box 30009-00100

The results are based on test sample only.



**MINISTRY OF TRANSPORT AND INFRASTRUCTURE
STATE DEPARTMENT OF INFRASTRUCTURE**

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Telephone: Nairobi 554950/3/4 Fax: 554877
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When replying please quote

MATERIALS TESTING AND RESEARCH DIVISION
MACHAKOS ROAD
INDUSTRIAL AREA
P.O. Box 11873
NAIROBI

Ref. No. M.7370/35/F/82/930

3 December, 2015

And date

Jomo Kenyatta University of Agriculture & Technology
P.O. Box 62000-00100

Received 17/12/2015

NAIROBI

Attn: Eng. C. K. Kabubo

Dear Sir,

**CHEMICAL ANALYSIS OF LIME AND SOIL
JOB CARD No.001022/CH/15**

The samples you submitted on 19/11/2015 gave the results shown below:-

I. Lime

Our Ref		1006/Ch/15
Your Ref		Rhino Lime
Calcium as CaO, % m/m		56.6
Iron as Fe ₂ O ₃ , % m/m		0.4
Aluminium as Al ₂ O ₃ , % m/m		1.8
Magnesium as MgO, % m/m		0.65
Bulk Density		0.54

II. Soil

Our Ref	Your Ref	Sulphates as SO ₃ , % m/m
1007/Ch/15	N1	0.003
1008/Ch/15	N2	0.024
1009/Ch/15	N3	0.066
1010/Ch/15	N4	0.009

Your account of K.shs.4, 500/= has been settled vide receipt No. MR.5864420 dated 19/11/2015.

Yours faithfully,

CHIEF ENGINEER MATERIALS
MINISTRY OF TRANSPORT AND INFRASTRUCTURE
P.O. Box 11873-00100
NAIROBI

Rebecca K. Angwenyi
For: CHIEF ENGINEER (MATERIALS)





Laboratory Test Report

Report Ref: KEBS/TES/1723/IN/14

Date: 24 September 2014

- | | |
|---|--|
| 1. Description of Sample: Molasses | 6. KEBS Sample Ref.No: BS201424950 |
| 2. Sample Submitted by: ENG. CHARLES K. KABUBO | 7. Date of Receipt: 04 September 2014 |
| 3. Customer Contact: Eng. Charles K. Kabubo | 8. Date Analysis Started: 08 September 2014 |
| 4. Customer's Ref. No: Private | 9. Sample Submission Form No: 107690 |
| 5. Customer's Address: P.O.Box 62000 -002000 NAIROBI. | |
| 10. Additional information provided by the customer:
MS (Sony sugar) | |
| 11. Acceptance criteria-title and number of specification against which it is tested:
Kenya Standard Specification for Molasses KS 01-953 | |

12. Parameters tested and Method(s) of test: as listed in the report below

LABORATORY TEST REPORT					
No.	Parameters	Results	Requirements	Test Method No	
1.	Brix	^o Brix	86.52	78Min	KS 953
2.	Calcium	% m/m	1.01	2.1Max	KS 05- 58
3.	Density	g/cm ³	1.40	1.33Min	KS 05- 58
4.	Magnesium as Mg	ppm	0.05	500Max	KS 01- 63
5.	PH		6.04	5-6	KS 953
6.	Phosphorus as P	% m/m	0.003	0.5Max	KS 05- 58
7.	Potassium as K	% m/v	1.77	5000Max	KS 01- 63
8.	Sulphated Ash	% m/m	12.10	15Max	KS 05- 58
9.	Total Ash	% m/m	7.83	10Max	KS 05- 58

COMMENTS/REMARKS:

The sample performed as shown

Tom O. Okumu - Manager Inorganic Chemistry Laboratory
FOR: MANAGING DIRECTOR

24 September 2014
Date of Issue

The results contained herein apply only to the particular sample(s) tested whose sample submission form serial number is herein quoted, and to the specific tests carried out, as detailed in this Test Report. No extract, abridgement or abstraction from a Test Report may be published or used to advertise a product without the written consent of the Managing Director, KENYA BUREAU OF STANDARDS.