UNIVERSITY OF NAIROBI

IMPROVEMENT OF RED VOLCANIC SOIL USING VOLCANIC TUFFS FROM NGURUNGA

BY: MELAU KENNEDY SARONI

F16/21543/2007

A project submitted as a partial fulfillment for the requirement for the award of the degree of

BACHELOR OF SCIENCE IN CIVIL ENGINEERING

MARCH 2014
DECLARATION
I, Melau Kennedy Saroni, do declare that this report is my original work and to the best of my knowledge, it has not been submitted for any degree award in any University or Institution.

Signed______________________________________________ Date __________

Melau Kennedy Saroni

CERTIFICATION
I have read this report and approve it for examination

Signed_______________________________________________Date_____________

Mr. J.R.Ruigu
ABSTRACT
Long-term performance of pavement structures is significantly impacted by the stability of the underlying soils. Roads have to be supported on sound sub-surface and base material. However, in many Kenyan regions, the local sub-grade materials are poor in strength and other properties, for instance red soils may not provide suitable sub-grade requirements.

In situ subgrades often do not provide the support required to achieve acceptable performance under traffic loading and environmental demands. The strength and stiffness of a soil layer can be improved through the use of additives to permit a reduction in design thickness of the stabilized material compared with an un-stabilized or unbound material. The design thickness strength, stability, and durability requirements of a base or sub-base course can be reduced if further analysis indicates suitability.

Although stabilization is an effective alternative for improving soil properties, the engineering properties derived from stabilization vary widely due to heterogeneity in soil composition and due to differences in physical and chemical interactions between the soil and candidate stabilizers. Information on grain size distribution and Atterberg limits must be known to initiate the selection process of a stabilizer.

These variations necessitate the consideration of site-specific treatment options which must be validated through testing of soil-stabilizer mixtures. This project addresses soil treatment with the traditional calcium-based stabilizers. Soil for use in constructing pavement layers in road construction normally requires some improvement of the different characteristics of that particular soil in order to meet certain engineering properties. This improvement is normally achieved by the use of a stabilizing agent, normally cement and lime.

The possibility of including pozzolanic ash in the process of soil improvement/stabilization was carried out during this study with results supporting the same. Results from this study indicate that pozzolanic ash can be used as an additive to soil when improving the engineering properties of the soil used with an impact of reducing the cost of the improvement while still achieving the set standards.

The results from this study have shown that natural pozzolanic ash has an improving characteristic in the strength of soil used as sub-grade.
DEDICATION
This project is dedicated to my parents (Mr. Joseph Melau and Mrs. Brigit Melau) who have sacrificed a lot to get me to this point. Their unwavering support throughout this course has been inspirational.
ACKNOWLEDGEMENTS
Firstly my sincere thanks goes to God Almighty for his care and mercy for this far I have come in life.

I also thank my supervisor Eng. J.R Ruigu for his immense support, encouragement, positive criticism and guidance during the whole research process without whom much would not have been achieved.

Also I thank Civil Engineering staff members and my colleagues who guided and assisted me in accomplishing this research work.

Other sincere thanks goes to my family for the assistance they have offered me as I undertook my studies.

Finally I would like to thank my friends who stood by my side throughout my studies.

Thank you.
Table of Contents

DECLARATION ........................................................................................................... ii
CERTIFICATION ....................................................................................................... ii
ABSTRACT ................................................................................................................ ii
DEDICATION ............................................................................................................ iii
ACKNOWLEDGEMENTS ............................................................................................ iv
LIST OF FIGURES .................................................................................................. viii
LIST OF TABLES ..................................................................................................... ix
CHAPTER ONE ........................................................................................................ 1
  INTRODUCTION ....................................................................................................... 1
    1.1 GENERAL ......................................................................................................... 1
    1.2 PROBLEM STATEMENT .................................................................................. 3
    1.3 JUSTIFICATION .............................................................................................. 3
CHAPTER TWO ........................................................................................................... 5
  2.0 LITERATURE REVIEW ...................................................................................... 5
  2.10 SOIL PROPERTIES .......................................................................................... 5
  2.2 RED COFFEE SOIL ........................................................................................... 6
    2.2.1 FORMATION OF RED SOILS .................................................................... 7
    2.2.2 ENGINEERING CHARACTERISTICS OF KENYAN RED SOILS ........ 10
  2.3 SOIL STABILIZATION ......................................................................................... 14
    2.3.1 Definition .................................................................................................... 14
    2.3.2 POZZOLANA ............................................................................................... 15
    2.3.2.1 Clays and shales ................................................................................... 16
    2.3.2.2 Volcanic Ash .......................................................................................... 19
    2.3.3 POZZOLANA PROPERTIES ..................................................................... 22
    2.3.4 Volcanic Tuff ............................................................................................. 25
    2.3.5 USE OF POZZOLANA IN PORTLAND CEMENT CONCRETE FOR .... 27
      APPLICATIONS IN HIGHWAY CONSTRUCTION .......................................... 27
    2.3.6 USE OF POZZOLANA IN SOIL IMPROVEMENT FOR APPLICATIONS IN 27
      HIGHWAY CONSTRUCTION ............................................................................. 27
    2.3.7 POZZOLANA QUARRY DEPOSITS IN KENYA ......................................... 28
CHAPTER THREE ................................................................................................. 32
  3.0 Research Methodology ...................................................................................... 32
LIST OF FIGURES

Figure 1 - Volcanic ash being excavated in Rwanda.

Figure 2 - Crushed volcanic tuffs at Ramji & sons quarry, Ngurunga.

Figure 3 - Volcanic tuffs crusher at Ramji & sons quarry, Ngurunga.

Figure 4 - Volcanic tuffs crusher at Ramji & sons quarry, Ngurunga.

Figure 5 - A lorry carrying crushed volcanic tuffs to a cement factory being weighed.

Figure 6 - Red Soil at Dagoretti (Southern Bypass).

Figure 7 - Sieve Analysis arrangement.

Figure 8 - Standard proctor test equipment.

Figure 9 - Trimming off excess material after compaction.

Figure 10 - Variation of consistency of fine grained soil in proportion to water content.

Figure 11 - Standard cone penetrometer.

Figure 12 - Plastic limit test.

Figure 13 - CBR moulds soaked in water.

Figure 14 - Manual CBR machine.
LIST OF TABLES

Table 1-Chemical composition of clay suitable for use in calcinated clay pozzolans.

Table 2-Composition of volcanic ash suitable for use as pozzolana.

Table 3-Natural pozzolana and cement demand projections.(2009-2018)

Table 4-Pozzolana deposits and use analysis for the next 100 years.

Table 5-Results.
CHAPTER ONE

INTRODUCTION

1.1 GENERAL

In civil engineering practice road construction is a major concern. It is an undertaking that is important to the economic development of any country. This is because it enables quick and easy movement of goods and delivery of services.

It is in this context that road engineers are involved in trying to obtain materials that are readily available, less costly, and in cases that such materials are not readily available, to device ways of improving deficiencies of the available materials. Improvement of such materials should be aimed at achieving high qualities in terms of strength, workability and durability at minimum construction costs as well as maintenance costs.

Soil affects the route location decision in transport engineering hence the type encountered should be firm enough to serve as a subgrade material. Red soils have montmorillonite ions which absorb water, expand and increase the soil volume on wetting. These soils may cover a depth of up to 1.5 meters from the ground surface in certain areas. This poses a challenge to engineers and hence the need to provide a solution. The current practice has been to remove the red soil and backfill with a material of high bearing capacity and with less swelling characteristics or stabilize the soil using lime, cement, bitumen and grouting.

Removing all these soils during road construction may not be feasible or lead to increased construction costs in excavation and disposal of the red soils. Excavation also leads to degradation in quarry import, borrow pit material and also leaves steep hazardous slopes.

The properties of materials used for the base and sub base of a surfaced road need to be much more closely controlled than those for gravel roads. In its broadest sense soil stabilization implies improvement of soil so that it can be used for sub-bases, bases and in some rare cases, surface courses. Soil stabilization offers one of the best forms of base construction for use in tropical and sub-tropical countries. A wide variety of soils may be
stabilized. Apart from highly organic soils and certain rare soils, the only limiting factor to the use of soils for stabilization is that they must be capable of being broken up to a fine tilth in order to mix the stabilizer. The main properties that may be required to be altered by stabilization are:

1. Strength: to increase the strength and thus stability and bearing capacity;
2. The volume stability: to control the swell-shrinkage characteristics caused by moisture changes;
3. Durability: to increase the resistance to erosion, weathering or deformation due to traffic usage;
4. Permeability: to reduce permeability and hence the ingress and passage of water through the stabilized soil.

Research has shown that the use of bitumen, lime or cement is effective. Unfortunately, the costs of these stabilizers are on the higher side making them economically unattractive as stabilizing agents. Recent trend in research works in the field of geotechnical engineering and construction materials focuses more on the search of cheap and locally available material such as natural pozzolana, bagasse ash, fly ash, blast furnace slag e.t.c. as stabilizing agents for the purpose of full or partial replacement of traditional stabilizers.

This project therefore attempts to provide an alternative solution to the current practice. The alternative is to mix the natural pozzolana with the soil used as subgrade material. The principle is that the Natural pozzolana is cementitious in presence of water hence the swelling and shrinkage properties of the red soils are minimized or even eliminated. The Natural pozzolana also performs a separation function thereby restraining the aggregates from mixing at the boundary between the subgrade and the base. Lateral and upward movements of the subgrade soil are also restrained by the Natural pozzolana.

This method provides a cost effective solution to road construction in areas prone to swell pressures. The Natural pozzolana is relatively cheap compared to other stabilizer and this reduces the cost which would otherwise be incurred by getting the fill material of acceptable engineering properties.
Currently, research is conducted worldwide to find road construction material that will be both economically feasible as well as environmentally friendly. The use of in-situ and non-industrial material in road construction was observed to be the best solution especially by including various stabilization techniques and chemicals. These stabilization methods have proved to be useful and more so economical in improving the quality of material initially regarded as being marginal, despite the high costs associated with cement and other chemical stabilizer.

1.2 PROBLEM STATEMENT
A problem continuously facing an engineer is that concerning varied properties of soil from site to site hence procedures by which unsuitable soils may be improved to make it useful as a subgrade material by stabilization. In most cases sub grade soils that are unsatisfactory in their natural state can be altered by admixtures, by addition of aggregates or by proper compaction and thus made suitable for highway subgrade construction.

The problem of cost is a factor since the current materials (cement and lime) are relatively expensive. Therefore the introduction of volcanic pozzolana ash which is readily available and cheap will go a long way into more savings in the construction industry. Environmental pollution (CO$_2$) is greatly reduced due to industrial emissions during processing of cement.

1.3 JUSTIFICATION
One of the ways in which highly urbanized cities cope with rising cost of road construction is seeking cheaper and readily available cementitious material, like natural pozzolana, lime and cement. Red soils have relatively high water absorption, swelling and shrinkage characteristic due to the presence of montmorillonite ions. Swelling and shrinkage of the soil cause cracking of the bituminous surface of paved roads. If possible an engineer should avoid such locations in the road route location. If the route so chosen still has these soils its only best to stabilize the soil with natural pozzolana, lime or cement rather than replacing the entire layer of the soil. Natural pozzolana is a potential source of admixture for soil improvement. The Natural pozzolana exhibits a likelihood of pozzolanic reaction due to its chemical composition and physical characteristics. Use of Natural pozzolana as an admixture in the stabilization of clay silty red soil result in stabilized samples with an improved
strength. Incorporation of Natural pozzolana ash can also improve drainage property and reduce both the plasticity and compression indices. Soil stabilization with natural pozzolana makes it possible to use construct roads on materials which are not directly suitable like red soils.
CHAPTER TWO

2.0 LITERATURE REVIEW

2.10 SOIL PROPERTIES
Materials making up the earth’s crust can be divided into two main groups. These are rocks and soils. Rock is the hard rigid and strongly cemented material and soil is the top loose material deposits normally formed from weathering of the rock and transportation of the weathed rock known as erosion.

When one looks at a soil, colour is the first feature that is observed. Soils have a variety of colours ranging from white through to grey and even to black. The colour of a dry sample of soil is affected by;

- The colour of the mineral grains it contains
- The quantity of organic material and
- The amount of oxidizing compounds in it.

Presence of moisture leads to a darker colour of the soil which when dry may look lighter.

Soils form the principle material for construction of roads, air fields and dams. In this respect it is important to have adequate knowledge of the soil properties. The performance of any type of soil under any given traffic and environmental conditions depend on several factors. Among them are;

- The engineering characteristics of the soils, their selection and method of placing them on the ground.
- Nature, pattern, weight and intensity of the existing and predicted traffic.
- Environmental and drainage conditions.

A specific soil structure is obtained when a soil deposit is formed. Environmental factors such as leaching, temperature cycles and organic activity may produce a very stable structure through natural cementation of particle contacts. These three groups of factors are closely related to field performance and it is important to relate them.
2.2 RED COFFEE SOIL
Red soils derive their name from their red colour. Kenyan red soils are tropical soils that result from weathering of parent rock (any type of parent rock containing iron) which is enriched with oxides of iron and alumina in the form of clay mineral especially kaolinite group and secondary clays like goethite and haematite. These soils are the traditional materials for road and airfield construction in Africa and many parts of the world. They have varied behaviour depending on the origin and formation. Location also determines their behaviour, thus Kenyan red clays present problems to engineers due to their unique properties which vary from site to site.

In any study of engineering properties of red clay, it is very essential to include the geological aspects that are involved in the soil formation. Goethechnical engineering therefore does have an inextricable relationship with geology. The use of geology helps an engineer answer pertinent questions that include:

- What materials are present in the structure and what effects do they have as far as the strength characteristics of the soil are concerned?
- By what process did these materials arise?
- What are the mechanical properties of the materials?
- Are they subject to risk of failure due to future changes within the engineering true scale due to geotechnical effects?

Certain aspects of red soils in Kenya have been studied although no comprehensive, universal and coordinated study has been done. Basically experts involved in various engineering undertakings in different parts of the country tackle the isolated problems as they are encountered. Therefore there arises a need for a well coordinated study that will bring together all that has been done and single out what that has yet to be done. On the other hand, there are certain generalizations as regards red soils that are not true for all cases. For instance the Kenyan red coffee soil (red friable clays) has been taken to have a bearing capacity of 80 KN/M². This may not be true for all cases of red soils across the country.
2.2.1 FORMATION OF RED SOILS

Red clay soils are found as in-situ products of weathering of many types of soils (extrusive and intrusive). Igneous, metamorphic and sedimentary rocks all contribute to the formation of red soils provided the right conditions prevail. Deep weathering profiles develop on volcanic deposits. In most cases most of the original parent rock has been lost but the gravel sized fragments of parent rock usually themselves get weathered and relic discontinuities may be encountered. Red soils are therefore formed from all types of rock:

- Metamorphic rocks
- Igneous rocks (volcanic rocks)
- Sedimentary rocks.

Red soils from metamorphic rocks

When rocks are subjected to heat and pressure, metamorphic rocks are formed which are entirely different from their original parent rocks. These rocks when subjected to weathering decompose to form red soils (if enriched with oxides of iron and alumina). The common types of red soils that result in red soils include;

- Migmatites
- Phyllites
- Silicate Minerals

Migmatites

The introduction of igneous (e.g. granite) into country rock of igneous kinds produces mixed rock or migmatite when subjected to processes of metamorphism. These rocks are found in areas of of high grade metamorphism in many belts. When these rocks are weathered the iron coating present gives it red colour for which red soil is identified.

Phyllites

When rock are subjected to pressure they are coverted into new products. In this case shale is converted into slates which in turn are converted to phyllites. Further subjection of phyllites to greater pressure results in micas and schist from which weathering breaks them
down to red soils that are micaceous in nature. In Kenya, these metamorphic rocks are the other major red soils forming red soils forming rocks other than volcanic. However most Kenyan red soils are also found where volcanic rocks are dominant.

**Silicate minerals**

Metamorphic rocks also result in silicate minerals the most important being the mica group. These are monoclinic minerals whose property of splitting into thin flakes is their main feature and easily recognized. The most commonly occurring micas are muscorites which also occur in granites and other acid rocks. They have silver crystals and are stable minerals. Other metamorphic rocks include quartzite and gneisses with quartz as the main mineral.

The weathering of these rocks leads to soils of various minerals. Silty red soils are also a result of these.

**Red soils from igneous rocks**

These are molten rock material generated from within or below the earth’s crust which reaches the surface from time to time and flows through orifices as lava while some cool within the crust as intrusive. The most significant igneous rocks in formation of red soils include;

- **Pyroclastics**
- **Biotites**

**Pyroclastics**

Pyroclastics are deposits formed by consolidation of fragments ejected during a volcanic eruption. If the ejected mass forms near the vent then we have agglomerates. Small particles of ash and dust may be blown by wind and spread over large areas in layers which harden to form tuffs. Weathered soil containing coarse grained silts and clays are formed.

**Biotites**

Biotites occur in many igneous rocks including granites, then in lavas and dykes. They occur in crystal brown colour. Feldspars and quartz are the other common mineral in igneous
rocks. When feldspars of granites decompose, the clay mineral kaolinite is formed which is the main mineral in red soils.

**Red soils in sedimentary rocks**

In the sedimentary group of rocks the most important group of soils that are responsible for the formation of red soil of the ferruginious deposits. These are the sedimentary rock deposits that contain iron which is either precipitated as primary mineral or in crystalline lattice. The most common types are iron stones, mudstones, micaceous sandstones. However in Kenya these types of soils are not prevalent.

**Red soil forming processes**

The primary mineral weathering through illite or montmorillonite, forming kaolinite, hydrated aluminium and oxide. More resistant quartz and mica particles may remain. As weathering proceeds the contents of kaolinite decreases and hydrated oxides progressively alter to sesquioxides which upon drying harden to form laterites, cementing agents and concentrations.

- Acid igneous rock: mainly sand or gravelly soils with little clay
- Basic rocks: fine grained silts and clays
- Quartz: sand
- Feldspar: kaoline (clay mineral)
- Product of kaolinization: sandy clay

The specific gravity of the soil increases as the weathering proceeds. This is because the specific gravity of iron bearing mineral is high compared to most other rock and tropical soil forming minerals.

Soluble irons are removed leaving iron and aluminium sesquioxides plus kaolinitic clay mineral. Most residual clays contain the mineral kaolinite aggregated with oxides and hydroxides of iron and alumina. The parent rock can therefore be from any of the following sub-groupings;

- Agglomerates basalt
• Trachytes
• Tuffs (Volcanic)
• Phonolite
• Quartzite
• Gneiss and Schist
• Phyllites

In Kenya red clays from volcanic tuff is very common especially around Nairobi (Kabete, Limuru) with high contents of clay. Generally volcanic red soils are the most widespread in Kenya. These soils are free draining and those formed on volcanicash deposits are much younger whereas those containing mineral halloytes are much older.

2.2.2 ENGINEERING CHARACTERISTICS OF KENYAN RED SOILS

General
The engineering characteristics of red soil influence its use and performance as a subgrade in the construction of road pavements. In general the tropical red soils have properties that are unique to the tropics hence it would not be in order to adopt testing techniques developed in the temperate areas. It is thus essential to develop techniques to soils in the tropics.

Texture of the soil
The texture and the structure of the red soils of East Africa differs greatly from those in temperate zones. Tropical red soils have concretionary structure as compared to dispersed structure of temperate zone soil. The clay fraction may vary from 10% to 30%

In terms of texture, laterites for instance can be categorized into the following categories:

• Lateritic clays (diameter<0.002)
• Lateritic silts (diameter=0.002-0.06mm)
• Lateritic sands (diameter=0.06-2.0mm)

• Lateritic gravels (diameter=2.0-60mm)

• Lateritic stones and cuirasses (diameter>60mm)

Lateritic gravels and gravelly soils have high contents of fines. These soils exist as hard medium or weak granules.

**Structure of red soils**
The Kenyan red soil has a concretionary structure which influences its engineering properties. The physical hardening of laterites for example and its minerology affect the genesis of laterite soil and the physio-chemical aspects which involve coating of the soil by iron oxida and alumina. The soil particles later coagulate into clusters with subsequent reduction of specific surface.

In Kenya, Sasumua clay for example, has aggregated particles that formed clusters due to the presence of halloysites with iron coating.

**Colloidal activities**
Colloidal activity is the ratio of plasticity index to the clay mineral contents. The predominant clay mineral in the red soil in Kenya is kaolinite which is inactive hence low colloidal activity for Kenyan red soils. Residual red soils found in Kenya have colloidal activity between 0.4-1.0 while non residual soils have values greater than 1.0. Low colloidal activity can also be due to method of weathering which involves coatings of soil particles with sesquioxides hence surface activity suppressed

**Shrinkage and swelling characteristics**
In general, kaolinite particles swell very little but red soils in Kenya undergo an amount of swell that depends on the initial density and moisture content, the composition and the method of drying. The swell characteristics of the Kenyan red soils are normally found in soils whose permanent rock is of volcanic origin. Weathering transforms volcanic ash as below;

Volcanic ash $\rightarrow$ montmorillonite (black brown) $\rightarrow$ halloysite $\rightarrow$ kaolinite
If the soil is partially weathered it posses properties of black cotton soils (montmorillonite) hence exhibiting the swelling characteristics;

- High unusual absorbtion of moisture leading to increase in index values.
- Increase in index values due to mixing.

The high moisture absorption leads to water being held in porous oxide surface structure. Oven drying leads to water loss hence high index results. On the other hand prolonged mixing of the soil in the laboratory breaks down the surface structure of the soil leading to release of iron oxide particles and destruction of any aggregation of soil particles and hence an increase in the index values of the soil. The breakdown in structure consequently leads increase in clay content. Linear shrinkage test can be performed for tropical red soils and it has been found to be more relaible than Atterberg limits tests hence it has been successfully used in selecting materials for road construction.

The Kenyan experience shows that the soil has the following index characteristics

Plastic limit=35%

Liquid limit =75%

**Strength properties of the soil**
Strength tests on red soils vary mainly with texture. The main tests include penetration test, vane shear test and plate loading test. The difference in strength illustrates the variation in the homogeneity of the soil profile and degree of laterization. The strenth therefore is a function of chemical composition, age and homogenity. Iron rich soils are harder than alumina rich red soils. Homogeneous profiles have less impurities hence stronger soils. Older laterites are stronger than recently formed. The most important strength test is the California bearing ratio test (CBR test) which expresses the strength of subgrades and base courses of pavements.

The CBR test was developed by the California division of highways as a method of classifying soils for suitable use in highway construction especially for the classification of base course as for the support of flexible pavements. CBR measures the shearing resistance of the soil under controlled density and moisture conditions because it has been found that certain
soils are very sensitive to such variables as slight changes in moulding moisture, compactive effort and soaking of the specimen. The CBR is expressed as a percentage of the unit load required to force a piston into the soil divided by the unit load required to force the same piston on the same depth into a standard sample of compacted crushed stone.

Consolidation and permeability characteristics
Permeability and consolidation characteristics of tropical red soils in Kenya have been studied by a few researchers using soil from certain areas as representative of the entire country. In these areas, Kabete, Limuru, Sasumua the parent rock is mainly of volcanic origin. Under normal conditions, clay minerals is expected to have high compressibility and low permeability. The Kenyan red soils have low compressibility and high permeability. The angle of shearing resistance is also usually high.

These phenomena can be due to the coarse grained nature of the soil (high permeability) due to aggregated particles into clusters due to the presence of hallosites with iron oxide coating. Due to this aggregation of particles in red clay soils, there is less change in water content and volume changes in compression is less hence the low compressibility.
2.3 SOIL STABILIZATION

2.3.1 Definition
Soil stabilization may be regarded as the controlled process that alters the soil properties for the purpose of improving the structural properties of the soil. This is done by adding lime, cement and/or other additives (e.g. natural pozzolana) to the soil to boost volume stability, strength and stress-strain characteristics, permeability and durability.

The development of high strength and stiffness is achieved by reduction of void space, by bonding particles and aggregates together, by maintenance of flocculent structures, and by prevention of swelling. The permeability is altered by modification of pore size and distribution. Good mixing of stabilizers with soil is the most important factor affecting the quality of results. The two most commonly used stabilizers are cement and lime.

The chief property of soils with which the construction engineer is concerned is volume stability.

Stabilization should be thought of not only in terms of corrective treatment but also as a preventive measure against adverse conditions. These adverse conditions may develop either in the course of construction or any time during the life of the structure.

Soils that do not possess the desired characteristics for a particular construction reduce pavement life and can be improved by adding stabilizers. Poor subgrade soil conditions can result in inadequate pavement support and reduce pavement life.

Soils may be improved through the addition of chemical or cementitious additives. Such chemical additives range from Natural Pozzolana, waste products to manufactured materials. These additives can be used with a variety of soils to help improve their native engineering properties.

Stabilizer can fulfill one (or at the most two) of the following functions:

- Increase the compressive strength and impact resistance of the soil construction, and also reduce its tendency to swell and shrink, by binding the particles of soil together.
- Reduce or completely exclude water absorption (causing swelling, shrinking and abrasion) by sealing all voids and pores, and covering the clay particles with a waterproofing film.

- Reduce cracking by imparting flexibility which allows the soil to expand and contract to some extent.

- Reduce excessive expansion and contraction by reinforcing the soil with fibrous material

- The choice of the most suitable stabilizer will mainly depend on local availability and costs, but also to some extent on social acceptance.

2.3.2 POZZOLANA

**Definition**

Pozzolanas are materials containing reactive silica and/or alumina which on their own have little or no binding property. However, when mixed with lime, in the presence of water, they will set and harden like cement. Pozzolanas are an important ingredient in the production of alternative binding materials to Portland cement (OPC). Alternative cements provide an excellent technical option to OPC at a much lower cost and have the potential to make a significant contribution towards the provision of low-cost building materials and consequently affordable shelter.

Pozzolanas can be used in combination with lime and/or OPC. When mixed with lime, pozzolanas will improve the properties of lime-based mortars, concretes and renders for use in a wide range of building applications. Alternatively, they can be blended with OPC to improve the durability of concrete and its workability, and reduce its cost considerably.

A wide variety of siliceous or aluminous materials may be pozzolanic, and historically the two most widely used of these are calcined clays and volcanic ash. Calcined clay in the form of crushed fired clay bricks, tiles or pottery has traditionally been used for improving the properties of lime mortars and renders. (It is known as surkhi in India, homra in Egypt and semen merah in Indonesia.) Shales are harder than clays and have similar mineral contents resulting in similar pozzolanic properties.
Volcanic ash was first used as a pozzolana by the Romans from deposits close to the village of Pozzuoli, near Naples, hence the name pozzolana.

2.3.2.1 Clays and shales
Clays or shales suitable for use as a pozzolana are very widespread and are readily available in almost all regions of the world. They have been used as cement replacement materials on large-scale construction programmes in a number of countries, particularly the US, Brazil, Egypt and India. For example, in Egypt, a lime-calcined clay mortar was used in the core of the first Aswan dam built in 1902 and an OPC-calcined clay mixture was used in the construction of the Sennar dam in Sudan.

However this large-scale utilization has declined in the last three decades, due to the availability of pozzolanas which require less processing and are therefore cheaper, such as volcanic ash and pulverized fuel ash. Where these are not available, the use of calcined clay still has considerable potential. Although sandy clays are often used as a pozzolana, frequently in the form of crushed fired clay bricks, the coarser sand is not reactive. The pozzolanic activity resides in the finer clay mineral fraction, and sandy clays may not produce the best pozzolanas. Despite their variable pozzolanic performance, the use of ground underfired or reject bricks and tiles as a pozzolana is likely to continue on a small scale due to the low cost of these waste materials. Plastic clays, as used in tile manufacture or for pottery, can produce better pozzolanas, although the composition of good pozzolanic clays is variable. Table 1 gives the chemical composition (on an oven-dry basis) of some clays in India which produce pozzolanas conforming to the Indian Standard for calcined clay pozzolana (IS 1344.1981).
Table 1. Chemical composition of clay suitable for use in calcined clay pozzolanas

<table>
<thead>
<tr>
<th>Constituents</th>
<th>Contents by weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silica + Alumina + Iron Oxide</td>
<td>Not less than 70%</td>
</tr>
<tr>
<td>Silica</td>
<td>Not less than 49%</td>
</tr>
<tr>
<td>Calcium Oxide</td>
<td>Not more than 10%</td>
</tr>
<tr>
<td>Magnesium Oxide</td>
<td>Not more than 3%</td>
</tr>
<tr>
<td>Sulphur trioxide</td>
<td>Not more than 3%</td>
</tr>
<tr>
<td>Water soluble alkali</td>
<td>Not more than 0.1%</td>
</tr>
<tr>
<td>Water soluble material</td>
<td>Not more than 1%</td>
</tr>
<tr>
<td>Loss on ignition</td>
<td>Not more than 10%</td>
</tr>
</tbody>
</table>

Raw materials for calcined clay pozzolanas should be free from coarse sand or gravel greater than about 0.6mm in diameter. In tropical climates, clay deposits are often subjected to a form of chemical weathering which leaches out the silica and alkalis resulting in an accumulation of ferric and aluminium hydroxides. The soils produced are bauxitic (aluminium bearing) and lateritic (iron bearing). Although these soils are low in silica, normally considered essential for a pozzolanic reaction, both will exhibit some pozzolanic reactivity when calcined. In general, the reactivity of laterite is low but bauxite can show reasonable results and its use as a pozzolana should be considered if silica-bearing clays are not available.
Processing of calcined clays and shales

Calcining

The early stages of processing calcined clay pozzolanas are similar to the moulding and firing process for clay bricks, tiles or pottery, and traditionally the rejects from these industries have been used as pozzolanas. The optimum calcining temperatures for clay pozzolanas are slightly below those for clay bricks or tiles and therefore better results are likely to be obtained if the moulding and firing process are designed specifically for pozzolana production. The Indian Standard (IS1344 1981) gives the following range of temperatures for different types of clays:

Montmorillonite type 600 to 800°C

Kaolinite type 700 to 800°C

Illite type 900 to 1000°C

In practice, most clay soils consist of a mixture of minerals, and a calcination temperature of 700-800°C is normally considered suitable. The optimum period of calcination will vary with clay type but is normally around one hour or less.

Rotary kilns have been the most common means of calcining clays and have been extensively used in the US and Brazil. Natural gas or oil is normally used as a fuel and outputs vary from 12.5 to 100 tonnes per day.

Several Indian institutions, including the Central Road Research Institute and the National Building Organization, have undertaken studies to design and test kilns specifically for clay pozzolana production with production rates of between 5 and 20 tonnes per day. Two of the kilns are coal fired and are designed on the natural down-draught and forced air vertical shaft concepts similar to those commonly used, respectively, in the ceramics and lime industries through a series of fins against an updraft of hot gases generated by oil burners. Although the contact time is extremely short, of the order of a few minutes, it is apparently sufficient to calcine the ground clay feed. The cost of production, despite the cost of using oil as a fuel, is reported to be competitive with other methods.
Grinding

The second step in processing is the grinding of the calcined clay to a fine powder. On a small scale, this has traditionally been performed with human or, more commonly, animal-powered methods. Ball mills are more suited to large-scale applications. Some calcined clays, such as kaolin, will be softer than others and will therefore require less grinding in order to achieve the desired fineness.

2.3.2.2 Volcanic Ash

Deposits of volcanic ash are likely to be found wherever there are active or recently active volcanoes, for example in the Mediterranean, the Pacific region, and central and eastern Africa. The physical condition of volcanic ashes may range from loose fine material to coarse deposits containing quite large particles. Deposits may be loose, with an appearance and texture similar to a compacted coal or wood ash. Other deposits are cemented, sometimes with appearance and properties similar to stone, and in this form they are normally referred to as tuffs or trassy. The colour of deposits can vary from off-white to dark grey.

<table>
<thead>
<tr>
<th>Constituents</th>
<th>Contents by weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silica</td>
<td>45-65%</td>
</tr>
<tr>
<td>Alumina + Iron Oxide</td>
<td>15-30%</td>
</tr>
<tr>
<td>Calcium + Magnesium Oxide + alkalis</td>
<td>Up to 15%</td>
</tr>
<tr>
<td>Loss on ignition</td>
<td>Up to 12%</td>
</tr>
</tbody>
</table>

Table 2: Composition of volcanic ash suitable for use as a pozzolana
The pozollanic reactivity of ash deposits can vary considerably. The quality of material may also vary within a single deposit or a single geologically consistent stratum, with variations in depth being common.

Regular testing is therefore required if volcanic ash is to be used as a pozzolana and this has been a restraint on its commercial exploitation. However, volcanic ash is, or has recently been, successfully used as a pozzolana in many countries including the US, Germany, Japan, Italy, Kenya and Indonesia, with pilot plants tested in Tanzania and Rwanda. For example, 200,000 tonnes of volcanic pozzolana were used in the construction of the Glen Canyon dam in the US, completed in 1964. Volcanic pozzolanas usually have chemical composition within the limits shown on table 2.

![Volcanic ash pozzolana being excavated – Rwanda.](image)

**Figure 1: Volcanic ash pozzolana being excavated – Rwanda.**

**Processing of volcanic ash**

Once the deposits have been excavated most volcanic ashes will require only minor processing before use as a pozzolana. Many ashes are only loosely cemented and can easily be excavated by hand, although others may need mechanical or pneumatic equipment. Some lithic tuffs may require blasting with explosives. The ash may require drying, and in dry sunny climates this can simply be achieved by spreading the ash in a thin layer on a specially
prepared drying floor, similar to those commonly used to dry crops. Alternatively, in wet climates, and for large quantities, inclined rotary driers are normally used.

If the ash is cemented it will need to be crushed before entering the dryer. Some volcanic ashes will already be in a very fine, loose powdered form and may not require crushing or grinding. Other ashes may be of sufficient fineness but be cemented together. These will require milling or crushing. Coarse ashes and lithic tuffs will need to be ground in a ball mill or similar.

**Utilization of pozzolanas**

A fineness similar or slightly greater than that of OPC is usually recommended for pozzolanas although some have been ground considerably finer. The minimum fineness recommended by the Indian standards for pozzolana (IS 1344. 1981. Calcined clays) is 320 and 250m²/kg for grade 1 and 2 pozzolanas respectively, measured by the Blaine air permeability test.

Once the pozzolana has been ground, it must be blended with lime and/or OPC to produce a pozzolanic cement. This can be accomplished by human or animal-powered methods but full homogeneity is unlikely to be achieved and the strength and consistency of cements blended in this manner will be variable. Mechanical techniques, preferably intergrinding in a ball mill or, as a second option, dry blending in a pan or concrete mixer, will give better results in terms of both strength and consistency.

Pozzolanas can be used with either lime and/or OPC. With the latter, replacement of up to 25-30% is common, although research has suggested that for non-structural purposes replacement of up to 50% can be used. With lime pozzolana cements mixtures of 1:1 to 1:4(lime:pozzolana) by weight are used. The addition of 5-10% of OPC will improve strength and decrease setting times. A larger percentage of OPC may be required if only poor quality pozzolanas are available. The exact ratio of the ingredients will depend upon the quality of the respective raw materials and on the required characteristics of the concrete or mortar made from the cement.

A good calcined clay or volcanic ash pozzolana should produce a cement, when mixed with a good quality lime, capable of producing concretes and mortars with 7 and 28 days strengths in excess of 2 and 4 mega pascals (MPa) respectively. Pozzolana strength development is slow and long-term strengths should be considerably higher, as much as 15MPa by 2 years.
Strength development can be accelerated if up to 4 per cent of fine gypsum is added to the lime-pozzolana mix.

**Origin**

Pozzolana, also known as pozzolanic ash (pulvis puteolanus in Latin), is a fine, sandy volcanic ash. Pozzolanic ash was first discovered and dug in Italy, at Pozzuoli. Pozzolana is a constructive residue collected from the Biosphere system which is often used as a road base and as a material for producing concrete block for commercial uses. Hence it is also referred to as a fine volcanic ash. It was later discovered at a number of other sites as well. Vitruvius speaks of four types of pozzolana: black, white, grey, and red, all of which can be found in the volcanic areas of Italy, such as Naples.

**Types of pozzolana**

a. black

b. white

c. grey

d. red

2.3.3 POZZOLANA PROPERTIES

The pozzolanic reaction may be slower than the rest of the reactions that occur during cement hydration, and thus the short-term strength of concrete made with pozzolans may not be as high as concrete made with purely cementitious materials; conversely, highly reactive pozzolans, such as silica fume and high reactivity metakaolin can produce "high early strength" concrete that increase the rate at which concrete gains strength.

A pozzolan is a siliceous or aluminosiliceous material, which is highly vitreous (one of numerous species of siliceous sponges having glassy spicules). This material independently has few/fewer cementitious properties, but in the presence of a lime-rich medium like calcium hydroxide, shows better cementitious properties towards the later day strength (>
28 days). The mechanism for this display of strength is the reaction of silicates with lime to form secondary cementitious phases (calcium silicate hydrates with a lower C/S ratio) which display gradual strengthening properties usually after 7 days.

The extent of the strength development depends upon the chemical composition of the pozzolan: the greater the composition of alumina and silica along with the vitreous phase in the material, the better the pozzolanic reaction and strength display.

Many pozzolans available for use in construction today were previously seen as waste products, often ending up in landfills. Use of pozzolans can permit a decrease in the use of Portland cement when producing concrete, this is more environmentally friendly than limiting cementitious materials to Portland cement. As experience with using pozzolans has increased, current practice may permit up to a 40 percent reduction of Portland cement used in the concrete mix when replaced with a carefully designed combination of approved pozzolans. When the mix is designed properly, concrete can utilize pozzolans without significantly reducing the final compressive strength or other performance characteristics.

**Pozzolanic reaction**

At the basis of the Pozzolanic reaction stands a simple acid-base reaction between calcium hydroxide, (Ca(OH)$_2$). Simply, this reaction can be schematically represented as follows:

\[
\text{Ca (OH)}_2 + \text{H}_4\text{SiO}_4 \rightarrow \text{Ca}^{2+} + \text{H}_2\text{SiO}_4^{2-} + 2 \text{H}_2\text{O} \rightarrow \text{CaH}_2\text{SiO}_4 \cdot 2 \text{H}_2\text{O}
\]

Calcium hydroxide a constituent in natural Pozzolana when mixed with the soil, the calcium causes the clay particles to flocculate into a sand-like structure, reducing the plasticity of the soil.

This reduction in plasticity reduces the shrink/swell characteristics of the soil.

The compounds formed possessing cementitious properties at room temperature which have the ability to set underwater. It transformed the possibilities for making concrete...
structures, although it took the Romans some time to discover its full potential. Typically it was mixed two-to-one with lime just prior to mixing with water. The Roman port at Cosa was built of Pozzolana that was poured underwater, apparently using a long tube to carefully lay it up without allowing sea water to mix with it. The three piers are still visible today, with the underwater portions in generally excellent condition even after more than 2100 years.

Modern pozzolanic cements are a mix of natural or industrial pozzolans and Portland cement. In addition to underwater use, the high alkalinity of pozzolana makes it especially resistant to common forms of corrosion from sulfates. Once fully hardened, the Portland cement-Pozzolana blend may be stronger than Portland cement, due to its lower porosity, which also makes it more resistant to water absorption and spalling.

Limitations

As the density of CSH is lower than that of portlandite (calcium hydroxide) and pure silica, a consequence of this reaction is a swelling of the reaction products. This reaction may also occur with time in concrete between alkaline cement pore water and poorly-crystalline silica aggregates. This delayed process is also known as alkali silica reaction, or alkali-aggregate reaction, and may seriously damage concrete structures because the resulting volumetric expansion is also responsible for spalling and decrease of the concrete strength.

Advantages

- Reducing green-house gases from business oriented firms and industrialized emissions
- Reducing CO₂ emission, which is certainly one of the primary contributors to global warming
- Is actually used as supplemental cementitious materials
- Many commercial and gardening by-products are pozzolanic
- Widely-used to lessen the measure of cement used within concrete
- Pozzolana cement blend may perhaps be stronger than Portland cement, because of its low porosity, that also can make it more protected from water absorption and spalling.
2.3.4 Volcanic Tuff

Tuff is an igneous rock that forms from the products of an explosive volcanic eruption. In these eruptions the volcano blasts rock, ash, magma and other materials from its vent. This ejecta travels through the air and falls back to Earth in the area surrounding the volcano. If the ejected material is compacted and cemented into a rock that rock will be called "tuff".

Tuff is usually thickest near the volcanic vent and decreases in thickness with distance from the volcano. Instead of being a "layer" a tuff is usually a "lens-shaped" deposit. Tuff can also be thickest on the downwind side of the vent or on the side of the vent where the blast was directed.

Some tuff deposits are hundreds of meters thick and have a total eruptive volume of many cubic miles. That enormous thickness can be from a single eruptive blast or more commonly from successive surges of a single eruption - or eruptions that were separated by long periods of time.

**Tuff Rings**

A "tuff ring" is a small volcanic cone of low relief that surrounds a shallow crater. These craters, known as maars, are formed by explosions caused by hot magma coming in contact with cold ground water. The explosion blasts fragments of bedrock, tephra, and ash from the crater. The tuff ring forms as these ejected materials fall back to Earth. Tuff rings range in size from several hundred meters across to several thousand meters. They are typically less than a few hundred meters in height and have a very gentle slope of less than ten degrees.

**Welded Tuff**

Sometimes the ejecta is hot enough when it lands that the particles are soft and sticky. These materials "weld" together upon impact or upon compaction. The rock formed from this hot ejecta is known as a "welded tuff" - because the ejected particles are welded together. Some deposits might contain welded tuff near the vent and unwelded tuff at a distance where smaller, cooler particles fell to the ground.
**Many Types of Tuff**

"Tuff" is a name that is used for a broad range of materials. The only requirement is that the materials are air fall produced by a volcanic eruption. Tuff can contain fragments of dust-size particles to boulder-size particles and be composed of many different types of material.

Many tuff deposits contain fragments of bedrock that are unrelated to volcanic activity. These materials are involved when the volcanic explosion occurs below the ground. The subsurface explosion crushes the overlying bedrock and launches it into the air mixed with tephra and volcanic ash produced from the magma source below.

Different volcanoes are supplied with magma of different compositions. Many tuff deposits form from magma with a rhyolitic composition but andesitic, basaltic and other types of magma might contribute to the tuff.

Tuff also varies by particle size. Near the vent a tuff might consist mainly of large blocks of material in a volcanic ash matrix. With distance from the vent the clasts will be smaller in size. At the edges of the rock unit the tuff might be mainly composed of very fine ash.
2.3.5 USE OF POZZOLANA IN PORTLAND CEMENT CONCRETE FOR APPLICATIONS IN HIGHWAY CONSTRUCTION

Pozzolana is used in concrete admixtures to enhance the performance of concrete roads and bridges. Portland cement contains about 65 percent lime. Some of this lime becomes free and available during the hydration process. When pozzolana is present with free lime, it reacts chemically to form additional cementitious materials, improving many of the properties of the concrete. There are many advantages of incorporating pozzolana into a Portland cement concrete which have been demonstrated through extensive research and countless highway and bridge construction projects. Benefits to concrete vary depending on the type of pozzolana, proportion used, other mix ingredients, mixing procedure, field conditions and placement.

2.3.6 USE OF POZZOLANA IN SOIL IMPROVEMENT FOR APPLICATIONS IN HIGHWAY CONSTRUCTION

Pozzolana is an effective agent for chemical and/or mechanical stabilization of soils. The properties of soil which can be change by using of Pozzolana are density, water content, plasticity, strength and compressibility performance of soils, hydraulic conductivity, and so on. Typical applications include: soil stabilization, soil drying, and control of shrink-swell. Pozzolana provides the following advantages when used to improve soil conditions:

- Eliminates need for expensive borrow materials
- Expedites construction by improving excessively wet or unstable sub grade
- By improving sub grade conditions, promotes cost savings through reduction in the required pavement thickness
- Can reduce or eliminate the need for more expensive natural aggregates in the pavement cross-section.
- Higher ultimate strength
• Improved workability

• Reduced bleeding

• Reduced heat of hydration

• Reduced permeability

• Increased resistance to sulphate attack

• Increased resistance to alkali-silica reactivity (ASR)

• Lowered costs

• Reduced shrinkage

• Increased durability

2.3.7 POZZOLANA QUARRY DEPOSITS IN KENYA

Kenya has large deposits of Pozzolana in the form of volcanic tuffs in the Rift Valley due to previous volcanic activities. The natural state of these deposits varies considerably as does their pozzolanic reactivity. Pozzolana is mined and used as a binding agent in cement production. The pozzolana used in this project is from quarries in Ngurunga area which is close to cement factories in the Athi River area. The factories include East African Portland Cement Company, Bamburi Cement Company (Lafarge), Mombasa Cement Company (A.R.M.) and a new company Savannah Cement Company.

Pozzolana is mined and some times crushed at the quarries located about 6 kms from the cement factories. Crushed or uncrushed Pozzolana is supplied and transported to the factory in trucks by subcontracted suppliers.

Pozzolana is received at the factory from suppliers either crushed or uncrushed. Any uncrushed Pozzolana will is further crushed in the factories and blended.

Since there is no alternative application of the Pozzolana, which exist in plenty in the rift valley, no shortage of Pozzolana is expected over the next 100 years as shown in Tables 3 and 4 below.
### Table 3: NATIONAL POZZOLANA AND CEMENT DEMAND PROJECTIONS (2009-2018)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement output in</td>
<td>2.4</td>
<td>2.64</td>
<td>2.90</td>
<td>3.19</td>
<td>3.51</td>
<td>3.87</td>
<td>4.25</td>
<td>4.68</td>
<td>5.14</td>
<td>5.66</td>
<td>38.25</td>
</tr>
<tr>
<td>Kenya with annual growth</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>rate of 10%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(million tonnes)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pozzolana requirements</td>
<td>0.84</td>
<td>0.92</td>
<td>1.02</td>
<td>1.12</td>
<td>1.23</td>
<td>1.35</td>
<td>1.49</td>
<td>1.64</td>
<td>1.80</td>
<td>1.98</td>
<td>13.39</td>
</tr>
<tr>
<td>at 35% blend(</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>million tonnes)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table 4: POZZOLANA DEPOSITS AND USE ANALYSIS FOR THE NEXT 100 YEARS

<table>
<thead>
<tr>
<th>POZZOLANA DEPOSIT LOCATION</th>
<th>AMOUNT OF DEPOSITS (MILLION TONES)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lukenya</td>
<td>170</td>
</tr>
<tr>
<td>Athi River/ kitengela</td>
<td>465</td>
</tr>
<tr>
<td>South of kitengela/ Isenya</td>
<td>1,571</td>
</tr>
<tr>
<td>Total mineable deposits</td>
<td>2,206</td>
</tr>
<tr>
<td>Pozzolana requirement in the</td>
<td><strong>1,517</strong></td>
</tr>
<tr>
<td>next 100 yrs (growth in demand</td>
<td></td>
</tr>
<tr>
<td>of 10% in the next 2 decades</td>
<td></td>
</tr>
<tr>
<td>and 5 % next 3 decades and</td>
<td></td>
</tr>
<tr>
<td>thereafter constant demand) at</td>
<td></td>
</tr>
<tr>
<td>35% blend</td>
<td></td>
</tr>
</tbody>
</table>

* the deposits are estimated
Improvement of red coffee soil using volcanic tuffs from ngurunga

Figure 2: Crushed Volcanic Tuff at Ramji & Sons Quarry Ngurunga

Figure 3: Volcanic Tuff Rock Crusher at Ramji & Sons Quarry Ngurunga
Improvement of red coffee soil using volcanic tuffs from Ngurunga

Figure 4: Volcanic Tuff Crusher at Ramji & Sons Quarry site in Ngurunga

Figure 5: A lorry carrying the crushed volcanic tuff to the factory being weighed
CHAPTER THREE

3.0 Research Methodology

3.1 Introduction

The methodology employed involved samples collection and laboratory tests. Each test was conducted and the results analyzed.

The natural pozzolana ash was crushed from the volcanic tuff sourced from Ngurunga which is the main supplier for cement factories in the Athi River region.

The soil used in this study is red coffee soil (red silty clay) obtained from Dagoretti. In terms of extent of deposit, red silty clays are not restricted to the area of study but are wide spread throughout the central province.

In this study, experimental design was employed and deductions were derived purely from the obtained results. The tests were conducted for neat material and the material after stabilization.

Figure 6: Red Soil at Dagoretti(Southern Bypass)
3.2 Sieve analysis
This procedure is suitable for coarse grained soils. This was aimed at determining the particle size distribution or gradation of the gravel used. This was done through wet sieving to 75µm sieve. The data obtained was presented in form of graph plotted on grading chart.

![Sieve analysis arrangement](image)

Fig 7: Sieve analysis arrangement.

The soil can either be well-graded soil if no soil particle size is lacking or is in excess or poorly-graded soil. In the case of poorly graded soil it can either be gap/skip-graded where the soil has large percentage of its bigger and smaller particles and only a smaller percentage of the intermediate sizes, or closely/uniformly-graded if the soil has a particle size distribution extending over a limited range with most particles tending to be about the same size:

- $D_{10}$, $D_{30}$, $D_{60}$ = Diameter at 10%, 30% and 60% respectively.

- Coefficient of Uniformity,

$$c_u = \frac{D_{60}}{D_{10}}$$

- Coefficient of Curvature,
\[ C_C = \frac{D_{25}^2}{D_{10} \times D_{60}} \]

**Test apparatus**
- Wire brush
- Balance
- Drying oven
- Tray
- Receiving pan
- Sieve series (50.8mm, 38.1mm, 25.4mm, 19.1mm, 9.52mm, 4.76mm, 2.00mm, 0.84mm, 0.42mm, 0.25mm, 0.105mm, 0.074mm)

**Procedure**
Soil sample was initially air dried and then oven dried at 105°C for 24 hours. A sample weighing 100g was then taken. The taken sample was then soaked in water for 24 hours after which it was thoroughly washed a little at a time on a 2mm sieve rested on top of 0.074mm. The washed sample was then oven dried at 105°C for 24 hours. The sample was then passed in a series of sieves starting with the largest mesh unit and then proceeding with decreasing mesh sizes up to the receiving pan at the bottom. The weighed out gravel was then transferred to the top sieve and passed through by hand shaking onto a paper. A soft brush was used to clean the sieve. The material received on the paper was poured into the next sieve, while the retained fraction was weighed. The procedure was repeated with each sieve and the material passing through the last one collected in the pan at the bottom. From the obtained results a grading chart was drawn.

**3.3.0 Compaction test**

**Objective**
This was conducted with an aim of determining the moisture content at which the dry density is maximum i.e. the optimum moisture content (OMC).

**Apparatus**
- 2.5Kg metal rammer.
- A cylindrical mould
- A straight edge – steel strip 300mm long, 25 mm wide and 3mm thick with bevelled edge.
- A tray (for mixing of soil)
- Trowel.
- Moisture content tins
- A balance
- Mortar and pestle
- 20mm test sieve with a receiver

![Standard proctor test equipments.](image)

**Fig 8: Standard proctor test equipments.**
- Steel rods for extracting the specimen from the mould
Procedure
From the air dried soil sample that passed 20mm sieve, 2500g was measured and thoroughly mixed with suitable amount of water in the tray provided using a trowel until it formed a uniform paste. The weight of empty mould with its detachable base but excluding the collar was taken. The mould with its collar attached and the base plate was placed on a firm solid base and the moist soil placed in approximately three equal layers and on each layer 25 blows were applied by the use of 2.5kg rammer falling through a height of 300mm and distributing the blows uniformly all over the soil surface. The collar was then removed and compacted soil struck level with the mould using the straight edge. The mould and the soil with base plate in place were then weighed and the compacted soil removed using steel rod. The extruded soil specimen from the mould was mixed thoroughly and a sample taken for moisture content determination. Further samples of 2500g were taken and the whole process repeated using increasing amounts of water until the weight of compacted moist soil, which filled the mould, reached a peak and started to decrease i.e. the material falls in the wet zone. The dry density obtained in the determinations was plotted on a linear scale against corresponding moisture content and the best smooth curve joining the points drawn and the maximum point (apex) on the curve noted to give the maximum dry density (MDD) and optimum moisture content (OMC).
3.4.0 Atterberg Limit

These are moisture content limits at transition from one phase to another i.e. solid-plastic-liquid state. These moisture content values are used in determination of consistency of the material, basically for soil to be used in road construction. This involved determination of parameters such as; liquid limits (LL), plastic limit (PL), linear shrinkage limit (LS) and the plasticity index (PI) and modulus (PM).

3.4.1 Significance of test.

The atterberg limits test consists of liquid limit and plastic limit. A value frequently used in conjunction with these limits is the plasticity index. The engineering properties of soil vary with the amount of water present and results of the atterberg tests, expressed as moisture contents are used to differentiate between various states of the material. The liquid limit is the water content above which the soil behaves as a viscous fluid. The plastic limit represent the moisture content at which soil changes from plastic to brittle state. The plastic index is the arithmetic difference between the liquid and plastic limit, it can also be obtained by multiplying the percentage linear shrinkage with a constant value of 2.13.

The most common application of the result to highway sub grade is in the soil classification with those soils with comparable limits and indices classed together. As low plasticity index...
indicates a granular soil with little or no cohesion. Both the liquid limit and plasticity index are used to some degree as a quality measuring device for pavement materials, in order to exclude those granular materials with too many fine grained particles that have cohesive plastic qualities.

3.4.2 Liquid limit (LL)

**Definition**
LL is the moisture content of a sample at which a standard cone penetrometer penetrates a depth of 20mm into the sample OR;

Is the moisture content at which two sides of groove cut in the soil sample contained in the cup of casagrande apparatus would touch over a 13mm length after 25 blows.

![Standard Cone Penetrometer](image)

*Figure 11: Standard Cone Penetrometer*

**Procedure**
The liquid limit of a soil can be determined using the cone penetrometer according to BS 1377:1990: part 2, clauses 4.3, 4.5. The cone penetrometer is considered a more satisfactory
method than the alternative because it is essentially a static test which relies on the shear
strength of the soil, whereas the alternative Casagrande cup method introduces dynamic
effects.

**Apparatus.**
- Flat glass plate
- Cone penetrometer
- Two spatulas
- Cone cap
- A wash bottle
- Moisture content dishes.

### 3.4.3 Plastic limit (PL)

#### Definition
PL refers to moisture content of the soil expressed as a percentage of the dry weight at which the
soil, rolled into 3mm diameter treads, first crumbles.

![Fig 12 plastic limit test](image)

**Test materials and equipments**
- Moisture content tins
- Distilled water
• 2 palette knives
• Drying oven
• Balance
• A glass plate

**Procedure**
Approximately 20g of air-dry soil that passes 425µm sieve was put into a container and mixed thoroughly with distilled water to form a thick uniform paste. The paste was rolled into a ball using hands and then rolled on a glass plate using an open hand until it attained a threadlike form of 3mm diameter. The rolled specimen was then kneaded together and the process repeated until the thread shows signs of crumbling at 3mm diameter. The crumbled threads were then for moisture content determination. The results were tabulated as shown in the standard tables attached.

**3.4.4 Linear shrinkage**

**Definition**
Is a measure of how a soil sample will reduce in length upon drying expressed as a percentage of the original length. The sample for linear shrinkage determination should be taken from sample close to the moisture content of liquid limit.

**Apparatus**
• Brass moulds,
• Petroleum jelly,
• Distilled water,
• Two palette knives,
• Drying oven,
• A pair of vernier calipers,
• Flat glass plate
Procedure
The inner walls of the brass mould were cleaned and internal length measured accurately. A thin film of petroleum jelly was applied to the inner walls to prevent any soil from adhering onto the sides. The proportion of soil passing 2.00mm sieve was obtained from which an air dried sample passing 425µm sieve was used to prepare a soil paste on the glass plate using the palette knives. The consistency of the soil paste was at approximately that of liquid limit.

The process of placing the soil paste into the mould was done with great care avoiding inclusion of any air pockets. The mould was thus tapped lightly onto a hard surface and levelled to the top. The specimen was allowed to dry out in the air first for about 1hr until it detached itself from the walls of the mould. It was then transferred into the drying oven and dried at 105°±5 °C for 24hrs. The mould was then removed from the oven and allowed to cool. The mean length of the soil bar was then measured and recorded. For the specimens which cracked badly or broke such that the measurements were difficult, the test was repeated. The results were tabulated as shown in the standard tables.

3.4.5 Plasticity index (PI)
This was obtained by getting the difference between liquid limit and plastic limit.

3.5 California bearing ratio (CBR) test
This is a relationship between standard penetration of a cylindrical plunger (of a cross sectional area=1963mm²) penetrating the red soil at a given rate. At any value of penetration, the ratio of force to the standard force is defined as the CBR.

The ratio seeks to establish the strength hence the bearing capacity of the soil and hence know how to increase its strength e.g. by stabilization (mechanical, chemical or physical)

3.5.1: Objective.
To determine the CBR value of four samples of red soil sub-grades soils at varying dosages of 0%, 2%,4%,6%,8%,10% and 12% of volcanic tuff ash.

Sample care and Pre-treatment Storage.
In order to prevent moisture loss, or premature oxidation, care was taken to protect the bulk soil samples. Bulk samples were placed inside thick-gauge plastic bags and the bags were tightly sealed after air removal. This was done to prevent the sample from drying.
Improvement of red coffee soil using volcanic tuffs from ngurunga

**Obtaining a representative sample.**
After obtaining enough soil from the large bulk samples to create one batch of specimens (approximately 1800g), any particles larger than 5mm were removed. This was typically accomplished by hand, but a wire mesh screen can also be used. The screened soil was then blended together by hand until uniform.

**Initial Moisture Content.**
Representative samples were taken to determine the moisture content of the sample. If the natural moisture content of the sample was higher than desired for mixing, the sample was air dried to moisture content just below the target value. Special care was taken to frequently mix the soil to promote uniform drying throughout the sample.

**Mixing of Soil and Ash.**
The calculated amount of ash was admixed with raw or untreated material thoroughly to a uniform mix before the optimum water content was added.

**Dosage rates.**
Dosage rates can be specified in many different ways, but the most common way to define the dosage rate is based on the dry weight of the soil to be treated.

**Curing of the specimens.**
After completion of the compaction effort, the specimens were cured for seven (7) days in a suitable curing room, exposed to air. At no stage whatsoever during the seven day period was the compacted specimens submerged under water. The compacted specimens were covered with hessian bags to prevent the compacted specimen from drying out too quickly and thus preventing adequate strength gain from taking place.

After the seven day curing period, the specimen were removed from the hessian bags and submerged in water for seven (7) days soaking period, for the determination of the CBR.
3.5.2: General.
CBR is a measure of the shearing resistance of soil under controlled density and moisture conditions. It is obtained by measuring the relationship between load and penetration when a cylindrical plunger of cross-sectional area of 19.35cm² is made to penetrate the soil at a given rate. The CBR is a test conducted to the strengths of sub grades, sub-bases and bases of roads and runway pavements by determining the penetration resistance of the soil.

It is defined as the force per unit area required to penetrate a soil mass with a standard circular piston at the rate of 1.25mm/minute to that for corresponding penetration of a standard material.

CBR = %

It has been proven that there is good relationship between the CBR and the elastic modulus of elasticity of soils. Since the sub grades soils in Kenya are grouped according to their CBR values, then this is a very important test on soils to be used as sub-grade material.

3.5.3: Scope.
The test was done to evaluate in terms of improvement of soil strength the performance of volcanic tuff ash on red coffee soil. The tests were performed on soaked samples so as to simulate field conditions.

3.5.4: Apparatus.
- CBR machine
- A cylindrical metal mould with detachable base plate, top plate and collar.
- Spacer
- Rammer
- Surcharge weights
- Gauges
- Filter paper

![Figure 14: Manual CBR Machine](image)

### 3.5.5 Procedure
Water was added to the sample until the optimum moisture content was attained as determined in standard proctor penetration test.

Porous base plate and collar were connected with the mould and spacer disk was inserted at the base together with the filter paper.

The sample was compacted in mould in three equal layers.
3.5.6 Swell test
The collar was removed and soil trimmed to surface of the mould. Filter paper was placed on top of the specimen and a swell plate placed on top of the filter paper.

The whole assembly was soaked in water tripod and arranged for expansion test and an expansion gauge was attached. Dial gauge reading was taken every day for 4 days.

Tripod was removed, mould taken out of water and inclined slowly to drain excess water for 15 minutes.

3.5.7 Penetration test
- Mould with the soil was placed on CBR testing machine without the swell plate.

- Penetration piston and specimen were connected with loading device ensuring the piston was attached to the centre of the specimen.

- Load and penetration gauge were set to zero.

- CBR machine was adjusted to rate of penetration of 1 mm/minute and test commenced. Load and pressure gauges were read at 0.5mm, 1.0mm, 1.5mm, 2.0mm, 2.5mm, 3.0mm, 3.5mm, 4.0mm, 4.5mm and 5.0mm

- Penetration was also conducted on bottom surface of the sample.
CHAPTER 4

4.0 DATA COLLECTION

The results for the tests conducted are shown below and their graphical presentation shown in the appendix.

<table>
<thead>
<tr>
<th>Red Soil Status</th>
<th>Volcanic Tuff Ash%</th>
<th>Liquid Limit</th>
<th>Plastic Limit</th>
<th>Plasticity Index (Pl)</th>
<th>Linear Shrinkage%</th>
<th>C.B.R.%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Neat</td>
<td>0</td>
<td>61.7</td>
<td>26.0</td>
<td>35.7</td>
<td>16.8</td>
<td>6</td>
</tr>
<tr>
<td>Improved</td>
<td>2</td>
<td>68.4</td>
<td>33.3</td>
<td>35.1</td>
<td>16.5</td>
<td>7.63</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>69.5</td>
<td>35.5</td>
<td>34.0</td>
<td>16.0</td>
<td>8.53</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>68.6</td>
<td>36.5</td>
<td>32.1</td>
<td>15.1</td>
<td>9.47</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>52.0</td>
<td>25.0</td>
<td>27.0</td>
<td>12.7</td>
<td>10.4</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>57.5</td>
<td>35.0</td>
<td>22.5</td>
<td>10.6</td>
<td>13.0</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>55.3</td>
<td>35.1</td>
<td>20.2</td>
<td>9.5</td>
<td>15.26</td>
</tr>
</tbody>
</table>

*Table 5: Results*

4.1 PARTICLE SIZE DISTRIBUTION

According to BS 1377 and from the wet sieving and hydrometer analysis, the red coffee soil sample from Dagoretti showed considerable larger amounts of fines, very small percentage of fine and almost none on coarse as shown in Appendix A.

The red coffee soil sampled from Dagoretti was classified as fine grained soil that is very suitable to be stabilized with volcanic tuff ash of cement fineness. All the data from wet sieving and hydrometer analysis is shown in Appendix A.

4.2 ATTERBERG LIMITS

The data and analysis for the red coffee soil from Dagoretti for Atterberg limits are shown in Appendix C.

*Liquid Limit*

The data obtained was plotted for the neat soil sample and the stabilized soil. The liquid limit for the sample was measured at 20mm cone penetration. The liquid limit can however be defined as the moisture content above which the soil behaves as a viscous fluid.
**Plastic Limit**

It is defined as the moisture content at which soil changes from plastic to brittle state. The plastic limits for the sample are shown in appendix C and were obtained from oven dried test samples.

**Linear Shrinkage**

\[
\text{Linear Shrinkage} = \frac{\text{Change in Length}}{\text{Original Length}} \times 100\%
\]

**Plasticity Index**

Plasticity Index is defined as the range of water content where the soil is plastic. The plasticity of the samples were evaluated as per the calculation below.

\[
\text{PI} = \text{LL} - \text{PL}
\]

The percentage shrinkage multiplied by a constant value of 2.13

i.e. Initial length of specimen = 140mm

Final length of specimen = 125mm

Linear shrinkage = 15mm

Then PI = % Shrinkage * 2.13

The answer above must be equal to the answer obtained from LL – PL

**4.2 COMPACTION**

**Standard Proctor Test**

In order to determine the optimum moisture content and maximum dry unit weight of the soil being tested, the standard proctor compaction test was conducted. The optimum moisture content of the soil is the water content at which the soils are compacted to a maximum dry unit weight. Samples exhibiting a high compaction unit weight are preferred for supporting civil engineering infrastructure since the void spaces are minimum and settlements will be less.

Compaction tests were conducted on the neat soil sample to determine the moisture content vs. Dry density relationship.
The quantity of material used for compaction and the amount of volcanic tuff ash added were as follows:

For a standard proctor test the material is usually 3000 grams. Since the amount of additive added is mainly based on the dry weight of the sample to be improved, the following method is used:

Amount of additive to be added = 2%

$$\frac{3000}{1 + NMC} = X$$

Then additive = 2% of X usually in grams.

The results obtained for the neat soil were graphed and the values of MDD and OMC were also obtained as shown in appendix B.

**4.3 CALIFORNIA BEARING RATIO**

The results obtained from compaction were used in finding the amount of soil required to fill the CBR mould and the amount of water to be added e.g.

MDD = 1300 kg/m³

OMC = 30%

NMC = 15%

From the above data the amount of water and material for the neat sample was calculated as follows:

Amount of material = MDD × V × (1 + OMC) × 100%

= 1300 × 2.315 × (1 + 0.30) × 1

= 3912.35 grams.

Amount of water = (OMC - NMC) × 6000g

= (0.30 – 0.15) × 6000

= 900 ml.
The CBR test data and results were tabulated and graphed and are shown in Appendix D.
CHAPTER 5

5.0 DATA ANALYSIS AND DISCUSSION

5.1 PARTICLE SIZE DISTRIBUTION
The particle size distribution has been seen from the wet sieving and hydrometer methods. It is clear that the sample contains approximately 95% fines hence classified as fine grained soils according to the Unified Soil Classification System.

5.2 ATTERBERG LIMITS
This is a measure of the plasticity of the soil obtained from the liquid and plastic limits. The atterberg limits of the soil sample was obtained, the liquid limit being 61.7, plastic limit 26.0 and the plasticity index obtained as 35.7 for the neat sample.

As per the plasticity chart, the soil can be classified as MH; high silt soil. On adding the additive the soil’s plasticity index reduced from 35.1 for the 2% stabilized to 20.2 for the 12% stabilized as shown in the table 5 above.

The neat sample had a linear shrinkage of 16.8% and the stabilized samples had shrinkage of 9.5% which is a decrease of 7.3%.

The plasticity index of the red soil decreased from 35.7 to 20.2. This is a clear indication that the additive has improved the physical properties of the red soil. Generally, a soil having high plasticity index is undesirable in pavement construction since it would lead to failure of the pavement due to the frequent shrinkage and expansion of the material which would consequently lead to fatigue. Having lowered the plasticity index and the atterbergs limits in general, the stabilizer exhibits good stabilization properties.

However, since the red soil is to be used as a sub grade material much lower value of atterber’s limit, linear shrinkage and plasticity indices are required. This is so because the material is subjected to adverse conditions. The plasticity index of 20.2 obtained by administering 12% of the additive is not satisfactory enough. This can be attributed to the high amounts of clay materials in the soil as indicated by the large amount of material passing sieve number 36. This should hence be lowered to much sufficient values. This can be achieved by blending the volcanic tuff ash with lime since lime is very effective as a stabilizer in soils containing high amounts of fines.
5.3 CALIFORNIA BEARING RATIO

The California bearing ratio (CBR) value, of the stabilized soils is an important parameter in gauging the suitability of the stabilized soils. Thus, it gives an indication of the strength and bearing ability of the soil; which will assist the designer in recommending or rejecting the suitability of the soil for base, sub-base or sub-grade material.

For the soaked condition the peak CBR values obtained was at 12% Volcanic tuff ash content with a CBR value of 15.26. These values were higher above those of the neat sample which were 6%.

The above results therefore prove that, the hydration process of volcanic tuff ash is slow and requires considerable longer time than the hydration of cement. The main part of the reaction does not start until a couple of days after the mixing of pozzolan, (Assarson et al. 1974). As a rule, it is not finished one to five years later (Diamond and Kinter, 1965).

Volcanic tuff ash admixture has long time strength improving capability, which implies that the progressive increase in strength will enhance the stability of the pavement. The trend of increased compressive strength with curing period can be attributed to time dependent strength gain action of pozzolans.
CHAPTER 6

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS
The natural red coffee soil was classified as A-7 or fine-grained in the AASHTO and Unified Soil Classification System (USCS), respectively. Soils under these groups are of fair to poor engineering benefit.

Treatment of the natural soil with Volcanic tuff ash showed a drop in the plasticity index from 35.7 to 20.2 at 12% volcanic tuff ash content. The linear shrinkage was also reduced by 4mm i.e. from 18-14mm.

The peak soaked CBR values of 12% stabilized were given as 15.26%. These values met the requirements to be used as sub-grade materials as per RN 31, RN 21 and the Kenya Road Design manual. The minimum CBR value for sub-grade materials should not be less than 2%. With the CBR values obtained, the treated soil can be classified as class S4 from the Road Design Manual.

The objective of the research has been achieved with the soil being an S4 as per the Kenya Road Design Manual with a reduced plasticity index of 20.2 hence can be used as a road sub-grade material in road construction.

6.2 RECOMMENDATIONS
- The red soil to be stabilized for use in road base construction should be of the best quality possible in order to minimize the amount of the stabilizer that will be used.
- A detailed analysis with increase of curing time should be performed on the soil samples. When a soil is treated with pozzolanas (Volcanic tuff ash), the strength gain due to the addition of stabilizer would be increasing with increase in time. It is therefore important to come up with a way of improving the early strength gain by enhancing the chemical properties of the ash.
- It is also of importance if a detailed analysis is carried out on the optimum Volcanic tuff ash content that would give the best results in terms of the maximum dry density and CBR strength.
• Further studies should be carried out in order to identify the long term effects that pozzolana ash has on the durability of the road pavement structures.
REFERENCES


APPENDICES
APPENDIX A

Particle size distribution for the neat sample
APPENDIX B

Results from compaction of the neat soil sample
APPENDIX C

Results for Atterberg limits for both the neat and treated soil
APPENDIX D

CBR results for both the neat and treated soil