Suitability of Lateritic Soils as Construction Material in Sustainable Housing Development in Africa: A Geological Perspective

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DECLARATION

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Charles Adebayo, Oyelami

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The author, whose name appears on the title page of this dissertation, has obtained, for the research described in this work, the applicable research ethics approval.

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DEDICATION

To God the Father, God the Son and God the Holy Spirit –

The Alpha and Omega of this Thesis.

and

In memory of my very dear father,

Luke Oyelami Ogundiran

1937 – 2015

who fought so hard to witness this day.

"Remembering you is easy; I do that every day.

Missing you is heartache that never goes away."

and

to my Mum

Margaret Olawumi Ogundiran

I am what you made me. I love you.

and

to my Wife

Helen Funmilayo Oyelami

I found in you a good wife, far more precious than jewels.

You are my sanity, my peace. I love you.

and

to my Son

Michael Ayotomiwa Oyelami

You have been a blessing from the start; I love you with all my heart.

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ABSTRACT

Lateritic soils which have been described as highly weathered tropical or sub-tropical residual soils with varying proportions of particle sizes ranging from clay size to gravel, were studied in an attempt to establish its suitability or otherwise as sustainable material in building bricks and housing developments that will meet the present challenge of sustaining the environment without costing too much and maintaining a high standard of strength, durability and aesthetics. Index properties of the lateritic soils tested revealed them as mostly well-graded, comprising both cohesive (silt and clay) and cohesionless (sand and gravel) soil fraction. The mineralogical analysis shows the presence of sesquioxides in the clay portion which were found to be very useful in the natural binding process as well as in the presence of a stabiliser (cement). The geotechnical analysis on the lateritic soil revealed a strong compressive strength with a relatively sound dry density which could guarantee a good durability in resulting bricks made from these soil materials.

Further test on the strength and durability of compressed earth bricks (CEBs) made from these lateritic soils revealed a brick with compressive strength ranging between 6.33 and 15.57 MPa which are considered to be of very good strength coupled with its sound durability strength established over a period more than one year under different kinds of severe weather and seasonal conditions.

In conclusion, sesquioxides presence and mineralogy of lateritic soils were found to be largely responsible for their good compressive and durability strength which made them good and sustainable materials for CEBs.

Keywords: lateritic soil, compressed earth bricks, sustainable housing,

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LIST OF ABBREVIATIONS, ACRONYMS AND UNITS

AASHTO	American Association of State Highways and Transportation Officials
ADK	Ado-Ekiti
ARSO	African Regional Standard Organisation
ASTM	American Society of Testing and Materials
BC	Before Christ
CAH	Calcium Aluminate Hydrates
CBR	California bearing ratio
CEB	Compressed earth brick
CFC	Chloro-fluoro-carbons
CRAterre-EAG	International Centre on Earthen Architecture-National Superior School of Architecture in Grenoble
CSH	Calcium Silicate Hydrates
EV	Efficient of variation
FHA	Federal Housing Authority
HRB	Highway Research Board
MDD	Maximum dry density
NBRRI	Nigerian Building and Road Research Institute
NGSA	Nigerian Geological Survey Agency
OIA	Office of Internal Affairs
OMC	Optimum moisture content
OS	Osogbo
REE	Rare Earth Elements
RHA	Rice Husk Ash
UCS	Unconfined Compressive Strength
UN	United Nations
UNCHS	United Nations Community Housing Scheme
UNIDO	United Nations Industrial Development Organization
US	United States
USC	Unified soil classification
USCS	Unified Soil Classification System
WHO	World Health Organization

LIST OF MATHEMATICAL AND CHEMICAL SYMBOLS

Å	Angstrom
Al	Aluminium
Al_2O_3	Aluminium Oxide
C_3A	Tricalcium aluminate
C_4AF	Tetracalcium aluminoferrite
C_2S	Dicalcium silicate
C_3S	Tricalcium silicate
Ca	Cadmium
CA	Calcium Aluminate
$C\underline{S}H_2$	Gypsum
CO	Carbon monoxide
CO_2	Carbon dioxide
Cu	Copper
Fe	Iron
K	Potassium
meq	Milliequivalent
M.A.	Modified AASHTO
Mg	Magnesium
Mn	Manganese
Na	Natrium
Ni	Nickel
NO	Nitrogen monoxide
RaO_3	Sesquioxide ratio
Si	Silicon
SiO_2	Silica/Quartz
σ	Stress
τ	Shear strength or shear stress
φ	Angle of internal friction

LIST OF UNITS OF MEASUREMENT

°C	Degrees Celsius
g	Gram
kg	Kilogram
kg/m^3	Kilogram per cubic metre
kN/m^2	Kilo newton per meter squared
kWh	Kilowatt-hour
m	Metre
Mg/m^3	Milligram per meter cube
mJ	Millijoule
mm	Millimetre
µ	Mu/micron
µm	Micron meter
MN/m^2	Mega newton per meter
MPa	Mega-pascal

Chapter 1

GENERAL OVERVIEW

1.1 BACKGROUND

Countries around the world are facing major challenges relating to shelter and expanding urbanisation; this problem seems more prevalent in West Africa and it is often aggravated by the high cost and scarcity of building materials. This is evident in a country such as Nigeria, with a population of about 174 507 539 in 2013 and a 3.5% population growth rate projected per annum (National Population Commission, 2013). The Nigerian Punch Newspaper (23 March 2015) quoted Mrs Akon Eyakenyi, Nigerian Minister of Lands, Housing and Urban Development, who stated:

> *Government views with concern the fact that the cost of construction projects in Nigeria ranks among the highest in the world. It is a reflection of this concern that Mr. President is determined to find a veritable solution to this challenge.*

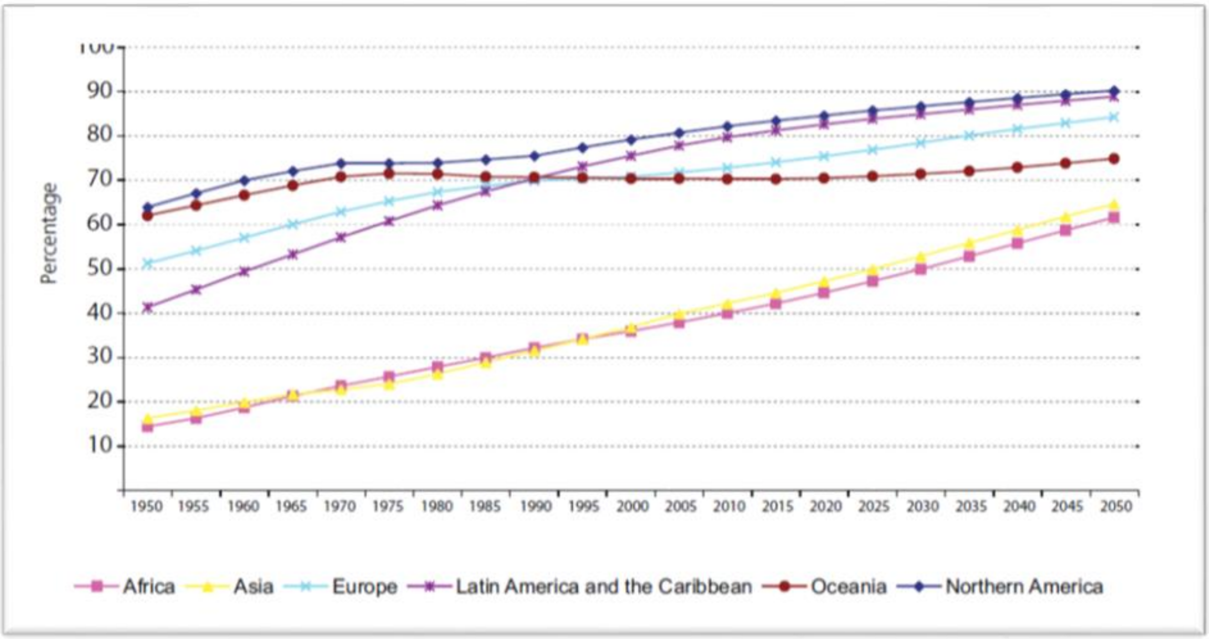
The United Nations (UN) Habitat report on Affordable Land and Housing in Africa (2011) affirms that the high cost of building materials is responsible for unaffordable housing, especially in Africa, and can be attributed to the failure of government to recognise the use of local earth materials in building constructions, but rather focus on conventional building materials stipulated in the national regulations and codes.

Statistics by the UN Habitat (2011) are as follows:

> *Only four out of every ten Africans currently lives in an urban area, the lowest ratio in the world. However, over the coming fifteen years, cities in Africa will, every day, become home to at least another 40,000 people. Growth in African countries is concentrated in cities and recent growth in major African cities was phenomenal: Between 2005 and 2010 Lagos grew by 1.8 million people, Kinshasa by 1.6 million and Luanda by 1.2 million.*

The above statement further confirms the need for sustainable housing development in Africa.

Figure 1.1 shows a 100-year urbanisation projection of different continents based on the earlier stated statistics.



Source: UN Habitat (2011: p 4).

***Figure 1.1:** 100-year urbanisation projection*

Overpopulation in urban cities, coupled with the continuous migration of people to urban areas, will naturally increase the housing demand of the few urban centres, and this challenge together with the fact that building materials are daily diminishing due to demand, scarcity and/or depletion of raw materials used in producing these building materials, are the main factors responsible for the housing deficit. The few available houses are not affordable for an average citizen in African countries.

The demand is not only just for ordinary housing, but also sophisticated habitable infrastructures which require a large volume of building materials, including raw materials and other limited natural resources. If the demand would continue to increase, the whole environment will be totally depleted of vital resources and lead to increased environmental pollution. This concern brings to the fore the need to look elsewhere and find better alternatives for limited conventional building materials. There is an urgent need to explore innovative technologies in building construction which require lesser dependence on imported building materials, promote the use of locally available materials, while maintaining a high standard without compromising on quality and aesthetics.

Nature has a way of providing answers to most of the challenges humans have been facing in recent times. Some soils have proven to be natural building materials from time immemorial. These soils were once used by ancient civilisation with relics found in the Egyptian dynasties, which were built around the fifth millennium Before Christ (BC) (Houben and Guillaud, 1994). Since then it has been used in various other places before the advent of present conventional building materials. An Indian soil scientist, Buchanan (1807), identified one such soil which he referred to as brick earth (laterite). Based on his observations, he described it as indurated clay containing a huge amount of iron in the form of red and yellow ochres.

Laterite, a renewable soil material, is present in many parts of the world, but especially in tropical regions with distinct wet and dry seasons. It could offer a lasting solution to the challenge of cost-effective, sustainable and energy efficient building in Africa.

1.2 PROBLEM STATEMENT

The present state of building construction in most parts of Africa relies solely on imported materials in the form of sandcrete blocks which utilises cement combined with gravelly-sand (sharp-sand). These building materials are getting more expensive, scarce and are non-renewable with the attendant problem of environmental degradation where these materials are sourced.

Although building with earth materials have been introduced in some areas in African countries, but its usages are still largely limited to schools, hospitals and few public places mainly constructed by the government. This is because the people still doubt the efficiency of earth bricks in terms of strength, durability and aesthetics.

Laterite, mentioned earlier, could fill the gap between housing deficit and provision of modest housing for the increasing population in Africa. To achieve this, the suitability of lateritic soil must be studied with an aim of creating awareness of its important use and to build confidence among the populace about its efficacy in brick making and in housing development at large.

Few researchers have worked on the durability of these earth bricks in Nigeria and most of their works were based on the economic aspect and aesthetics. Little attention has been given to the main constituent of earth bricks, which is the soil. Few researchers in Nigeria (Ola, 1978, 1980; Olotuah, 2002) have contributed on the strength and efficiency of compressed earth bricks, their focus were mainly on the engineering properties of lateritic soil with little or no information on the parent materials, namely the rocks from which these lateritic soils are derived and mineralogy of the soils which dictates the binding properties and gives an important clue to durability.

Therefore, this thesis would attempt to look critically at the properties of lateritic soil which makes it suitable as material in brick manufacturing with a focus on the influence of geology on its structure, texture and mineralogy. At the end, a better picture on the various properties of lateritic soils derived from different parent materials and their use in compressed earth blocks is given.

1.3 RESEARCH OBJECTIVES

This research focuses mainly on the usage of different lateritic soils from different parent rock materials with a view to determining their suitability or otherwise in the making of low-cost, durable, energy efficient and aesthetically appealing compressed earth bricks.

The main objectives are to:

- identify the possible influence of geology on the properties of the soil;
- assess the effect of mineralogy on the binding and durability of these soils;
- compare the grading characteristics and engineering properties of different lateritic soils from different parent rocks;
- establish the strength properties of bricks made from these different soil materials; and
- establish the possible mixing ratio by weight, volume and/or percentage of soil to stabiliser that will provide durability and efficiency while maintaining affordable cost.

1.4 SCOPE OF STUDY

This research intends to reach the above-mentioned objectives within the scope listed below:

- Studying the effect of the parent rock materials on the soil. This is grouped into two: those derived from schist bedrock and those derived from the older granites, all within the south-western basement complex of Nigeria.
- Studying the index properties of the lateritic soils, their grading characteristics, Atterberg limits and specific gravity.
- Studying the engineering properties of the soils, in terms of shear strength of the lateritic soils, their compaction characteristics and bearing capacity.
- Determination of the mineralogy of the lateritic soils via X-ray diffraction and X-ray fluorescence analyses.
- Studying the strength properties of the bricks produced from the studied soils.
- Monitoring bricks for at least one year (complete wet and dry season) after curing to establish their natural tendency to withstand harsh climatic conditions.

1.5 OUTLINE OF CHAPTERS

Chapter 1 provides a general overview of the study as well as the research objectives. Chapter 2 presents the earlier literature on the use and suitability of lateritic soils from different parts of the world as materials in earth buildings, as well as documented literatures on compressed earth bricks. Chapter 3 explains the case study area in terms of geography and geology of the area and justifies the reason for the choice of the study area. Chapter 4 of the thesis presents a detailed methodology adopted in the present study, together with minor modifications to established methodology where necessary. The results are presented in Chapter 5 mainly in the form of tables and figures, while the results are discussed in detail in Chapter 6. The conclusions and summary of the main findings in this thesis are presented in Chapter 7 together with the recommendations.

Chapter 2

LITERATURE REVIEW

2.1 INTRODUCTION

> *"Ever since man learnt to build homes and cities around 10,000 years ago, earth has undoubtedly been one of the most widely used construction materials in the world"* (Houben and Guillaud, 1994)

> *"As never before in history, common destiny beckons us to seek a new beginning… Let ours be a time remembered for the awakening of a new reverence for life, the firm resolve to achieve sustainability, the quickening of the struggle for justice and peace, and the joyful celebration of life*" (The Earth Charter).

Earth has proven to be man's best friend, companion and solution to most of his problems. Humans build homes with earth, trample on earth every day; earth equally serves as the solution to some of man's health challenges and this implies that humans cannot do without the earth.

Earth within the above context refers to soil which is unconsolidated mineral grains, usually formed by decomposition/weathering of rocks.

Growing environmental concerns have led to the discovery and appreciation of the unique qualities of earth materials. The use of earth material in construction work is less likely to lead to depletion of resources, increased pollution (water, air, soil), waste generation and biological changes (Bachar *et al.*, 2014).

In order to preserve and sustain the environment we live in, the use of environmental friendly building materials, commonly referred to as green building materials, must be encouraged and promoted in actualising the concept of sustainable building. Compressed earth bricks, as building material, entirely satisfies the requirements of sustainable building.

Sustainable building is defined by Meriani (2008) as structures which are designed, built, renovated or operated in a resource-efficient manner. It is designed generally for the well-being of the environment; well-being of the occupants, improving human productivity and efficient usage of energy, water and other resources; thus, reducing impacts on the environment without compromising standards and aesthetics.

These bricks come with numerous advantages both for man and his immediate environment. With the present global concern about the environment and its sustainability, attention is beginning to shift to energy efficient and environmentally friendly construction materials. Based on this fact, earth construction remains the best and most effective way of addressing the housing deficit, especially in developing countries of the world.

According to the UN Habitat Report (2011), *"…much more can be done in Africa to reduce the cost and increase accessibility of building materials whilst harnessing their ability to contribute to local economies and provide employment opportunities. Increasing affordable housing supply must equally be achieved in a way that is environmentally sustainable and does not affect local, international, and continent's ecosystems and natural resources in adverse manner."*

Earth bricks, especially compressed earth bricks, are naturally available, economical and fire-resistant. They are of high thermal capacity with low thermal conductivity, environmentally friendly, and above all, energy efficient (Bahar et al., 2004; Horpibulsuk et al., 2012; Kasthurba et al., 2008, 2007; Oyelami and Van Rooy, 2016a). It is an ideal material for sustainable building. Despite all these ecological advantages and cost benefits, earth has often been regarded as a building material for the underprivileged and often considered as second class building material for very low-income earners. The poor perception to the use of earth materials in building construction by the elites, and non-acceptance by governments is because of its inappropriate use by the so-called poor people; the use became imperative for them due to poverty, and a desperate urge to meet their housing demands (Oyelami and Van Rooy, 2016b). This is common, especially amongst people living below the poverty line. They use earth materials in its simplest, natural form without any technological or ordinary know-how. This has led to low acceptability amongst most social groups. Based on this, earth materials in building construction have not been widely recognised by government in many countries, which results in the non-availability of common developed building codes and standards (Oyelami and Van Rooy, 2016a, 2016b).

With the recent trend in reviving the use of sustainable materials for building construction, coupled with the extensive research work in this regard and the aggressive promotion of this style of construction by international organisations such as the United Nations (UN), United Nations Industrial Development Organisation (UNIDO), World Health Organisation (WHO), and the International Centre on Earthen Architecture (CRATerre-EAG), earth material has now been used for the realisation of decent housing, especially in Africa. This is with an aim of bridging the housing deficit that exists in the world which has led to a new trend and new faces of beautiful architecture that are aesthetically pleasing. Earth materials are gradually being accepted and more individuals, organisation and corporate bodies are embracing its use in modern housing development. It is now obvious that, the past negative perception is not necessarily about the material, but rather, how it is being used by different categories of people. Figure 2.1 shows a poorly constructed earth building and the new faces of earth construction.

Ebira Farm Settlement in Nigeria

Departments of Economics and Geography (fired bricks), University of Pretoria

Catholic Mission House in Nigeria (CEB)

Source: Author’s own (2016).

Figure 2.1: Different faces of earth bricks in building construction

Earth bricks are made from different types of soils with good cohesion characteristics. Cohesion is usually brought about by the presence of clay which serves as the binder or cementing agent in the soil (Adam and Agib, 2001; Kasthurba et al., 2008; Oyelami and Van Rooy, 2016a).

Houben and Guillaud (1994) identified seven most commonly used techniques in earth building construction. These include:

- **Adobe:** sun-baked earth bricks made with thick malleable mud with straw.
- **Rammed earth:** compacted earth in a framework.
- **Straw clay:** very clayey soil in the form of greasy slip which is added to straw. Earth only serves as binder here.
- **Wattle and daub:** wooden bearing structure filled with a daubed lattice or netting woven from vegetable matter.
- **Direct shaping:** plastic earth from which forms are modelled without any mould or framework.
- **Compressed earth bricks:** refers to manual production of earth blocks by compressing or compacting them in a small wooden or steel mould.
- **Cob:** consists of stacking earth balls together on top of each other and lightly tamping them with the hand or feet to form a monolithic wall.

2.2 BRIEF HISTORY OF CONSTRUCTION WITH EARTH

Construction with earth has been in existence since the early history of humans. Africa as a continent played a major role in human evolution. This role goes beyond evolution; it is on record that Egypt recorded the first form of civilisation with earth constructions which flourished for so many millennia (Houben and Guillaud, 1994).

The history of earth building as compiled by Houben and Guillaud (1994) narrated how homes, constructed from woven reeds and branches, covered with clay or filled with clods of earth, were used in the first human settlements. Evidence of these was preserved at the Merim and Fayum sites in the Nile Delta and dates from the fifth millennium BC. Advanced civilisation made its debut in Africa with the establishment of Egyptian dynasties (2900 BC) in which the Nile provided the prime building material, namely clay, silt mixed with the sand of the bordering desert to which straw from cultivated grains were added (Houben and Guillaud, 1994).

The following stages of the history of earth building can be summarised according to the account given by Houben and Guillaud (1994):

- **1070–1552 BC:** Evidence of the Tel el-Amarna centre in the new kingdoms where homes of nobles, palaces and temples were all built with sun-baked bricks. These buildings lasted for over four hundred years, as seen in their relics.
- **1200 BC:** This era witnesses the building art of brick vaulting as visible in lower Nubwa, between Luxor and Asswam (the Rammesseum granary).
- **Recent civilisation:** The last stage of development of earth building in Africa has to do with the influence by different cultures and civilisation. During this period, numerous shapes and techniques were developed. Northern Africa was influenced by the Mediterranean civilisations, which promoted the use of sun-baked bricks and rammed earth. East Africa was influenced by the people from the Indian ocean (Melanasians), who used daub and direct shaping. The Axum kingdom was influenced by the migrating Kushites, which promotes the use of sun-baked brick. Islam had a greater influence around the eleventh century in Africa. With Islam comes the change in appearance of ancient centres in Africa with the introduction of mosque architecture which serves as models for neighbouring countries.

2.3 LATERITIC SOILS

Laterite or lateritic soil could be one of the best materials to use in compressed earth bricks, because laterite is commonly a well-graded soil that combines both cohesive (silt and clay) and cohesionless (sand and gravel) constituents in a soil (Adeyemi and Wahab, 2008; Oyediran and Okosun, 2013; Oyelami and Van Rooy, 2016b). It contains sesquioxides and clay minerals which are very useful in the natural binding process of lateritic soil or in the presence of any chemical binder. Details of characteristics of laterites and its constituents are discussed in the subsequent sections.

2.3.1 Review of the definition of laterite

There have been many arguments, criticism and learned discussions about the definition of ‘laterites’ and ‘lateritic soils’. Different authors and researchers have tried to give different definitions in terms of its physical nature, chemistry, origin and morphology. Some include the very early arguments by Buchanan (1807), Babington (1821), Benza (1836), Clark (1838), Newbold (1844 and 1846), Prescott and Pendleton (1952), and a host of others. Most of their arguments centred on the definition, the use and general description of laterite, especially the ones around India.

The term 'laterite' appeared in academic literature over a century ago. The use of this term was associated with Buchanan (1807) who first used it to signify a building material in the mountainous region of Malabar, India (Maignien, 1966). The term 'laterite' could mean brick earth in some local dialects, but the name *latérite* got its meaning from a Latin word *later*, meaning 'brick', and therefore relating solely to the use to which these blocks are put (Prescott and Pendleton, 1952).

Walther (1916) described the term 'laterite' in terms of morphology as a red, brick-like colour and meant 'resembling bricks'. He proposed to extend the definition to all red-coloured alluvion and eluvion products and specifically to tropical red earths.

In terms of chemistry, Lacroix (1913) looked at the total content of hydrous oxides in laterites and on this basis, he classified them as:

- true laterites, containing greater than 90% hydroxides;
- silicate laterites, containing between 50 and 90% hydroxides; and
- lateritic clays, containing between 10 and 50% hydroxides.

Along this school of thought the following authors defined laterites based on the SiO_2/Al_2O_3 ratio: Bemmelen (1904), Harrassowitz (1926), and later Martin and Doyne (1927). They were of the opinion that in true laterites, the $Si0_2/Al_20_3$ ratio was less than 1.33, whereas in lateritic soil, the ratio was put between 1.33 and 2.0. Anything above that was considered to be non-lateritic soil. The limiting factor of this definition was that iron was not an essential element. The role of iron was underestimated, whereas iron plays a major role in the hardening and dehydration process in some grades of laterite (Alexander and Cady, 1962). This criteria, together with the $Si0_2/Ra0_3$ ratio, were adopted by Joachim and Kandiah (1941), who proposed the use of SiO_2/Sesquioxide ratio instead of Al_2O_3. The use of the different ratios was generally applauded and widely adopted during this period.

Pendelton and Sharasuvana (1946) (in Gidigasu, 1974), consequently defined laterite as *"soils in which a laterite horizon is found, and laterite soils as those in which there are an* *immature laterite horizon from which a true laterite horizon will develop if appropriate conditions prevail long enough"*. By implication, a lateritic soil could then be described as one in which there is an underdeveloped laterite horizon which will become true laterite given the appropriate conditions with enough time. This definition is closely related to the one given by Harrassowitz (1930), who considered laterites with a characteristic profile developing under tropical savannah and forming the following four distinct levels, from the base to the top:

- a fresh rock zone;
- a zone of primary alteration to kaolinite;
- a proper lateritic bed; and
- a surface zone with ferruginous incrustations and concretions.

Alexander and Cady (1962) defined laterite as *"highly weathered material rich in secondary oxides of iron, aluminium, or both, nearly void of bases and primary silicates, but it may contain large amounts of quartz and kaolinite"*. They submitted that laterites were neither hard nor capable of hardening on exposure to wetting and drying.

In his own contribution to the definition of laterite, Gidigasu (1972) described it as *"all the reddish, tropically weathered residual and non-residual soils including laterite rocks"*. However, he argued that the description includes four of the Great Soft Groups earlier defined by Thorp and Smith (1949) as follows: *"the reddish brown lateritic soils, yellowish brown lateritic soils, lateritic soils and the ground-water lateritic soils."* He concluded that only the initial three were zonal soils of tropical regions that has a well-developed profiles which reflect the dominating influences of climate and vegetation. The groundwater lateritic soil, according to him, was an intra-zonal soil with the profile reflecting the effect of imperfect to poor drainage, with the soil development process occurring over a long period.

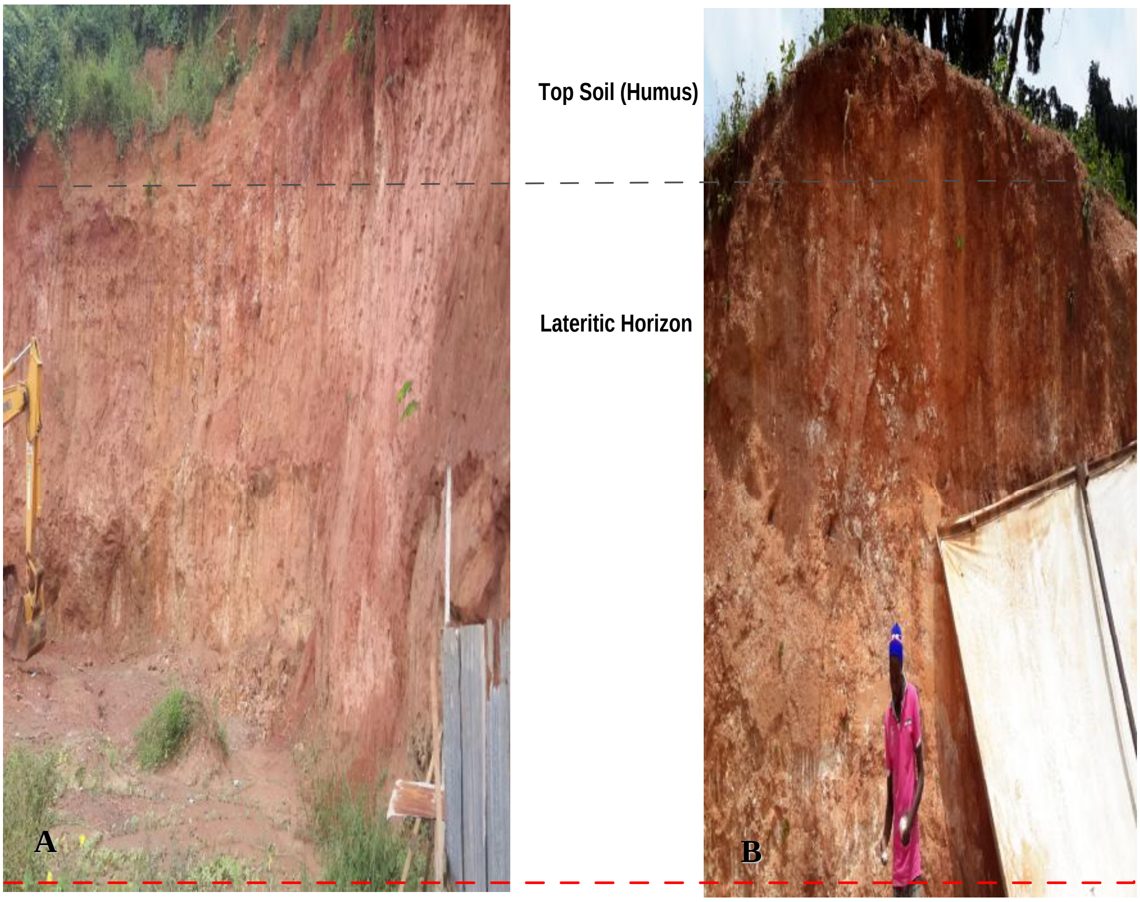
According to Mukerji (in Houben and Guillaud, 1994), *"laterites are highly weathered soils, which contain large, though extremely variable, proportions of iron oxide and aluminium, as well as quartz and other minerals. They are found in large quantities in the tropics and subtropics, usually just below the surfaces of vast open plains and clearings in heavy rainfall regions. The natural state varies from a compact concretion to crumbly soil. It may be of many colours: ochre, red, brown, violet, and black. The material is easy to cut and hardens quickly in air, and becomes quite resistant to meteorological agents."*

In a recent contribution, Giorgis *et al.* (2014) referred to another attempt to redefine laterite years ago by Schellmann (1983, 1986), who proposed a new definition and classification based on rock chemistry; basically on the Si/(Al + Fe) ratio in comparison with the chemical composition of the underlying parent rock. This again was widely accepted, but later criticised strongly by several authors such as Bourman and Ollier (2002, 2003), reply in Schellmann (cited in Giorgis *et al.*, 2014), where they proposed that the term 'laterite' should have been abandoned.

Maignien (1966) noted earlier in his review as follows: *"the words 'laterite' and 'lateritic' had come to be used in a variety of meanings, for the definitions were based sometimes on morphological, sometimes on physical and sometimes on chemical characteristics. The many attempts to standardize usage had met with varying degree of success."* He further observed that it was increasingly difficult to give an entire morphological, physical or purely chemical definition, because the word 'laterite' covers a wide variety of aspects of tropical soil formation and is too general.

Considering the difficulty and varying opinions as to the term 'laterite' and 'lateritic soil', within the context of this research work, laterite will be defined strictly based on field observations and the purpose of this work. Therefore, the word 'lateritic soil' is used to describe the highly weathered tropical or sub-tropical residual soils with varying proportions of particle size, ranging from clay size to gravel, usually coated with sesquioxide rich concretions (Oyelami and Van Rooy, 2016a, 2016b). The colour varies from liver brown to a rusty red colour. In the same vein, a 'laterite' could therefore be regarded as fitting exactly this definition, but in the case of laterites, non-residual (transported) soils are often included. Akpokodje (1986), referred to two types of soils occurring around his study area in the south-eastern part of Nigeria, namely: *"Lateritic soils that are essentially residual and non-lateritic tropical soils that are transported in origin"*. This implies that there is a distinction between lateritic soils, laterite and non-lateritic soils.

Lateritic soils come with varying degrees of laterisation which, given adequate time and right conditions, later becomes matured and true laterites. Figure 2.2 shows two exposed lateritic soils at varying degrees of laterisation. Profile B contains a large amount of kaolinite with white colouration still visible. It follows that, given the right conditions and time; it could still reach a stage of complete maturity like every other laterite.



Source: Author’s own (2016)

Figure 2.2: Two exposed lateritic soil profiles at different stages of laterisation

This is in line with the proposal by Harrassowitz (1930) and Pendelton and Sharasuvana (1946) in their definition of laterites and lateritic soils. This process is highly possible in residual soils, but seldom seen in non-residual soils. Giorgis *et al.* (2014) identified a compositional trend in agreement with an in situ laterisation process, where the lateritic profile develops with progressively increasing weathering and associated selective leaching upwards. Giorgis *et al.* (2014) therefore postulated a strong decrease in SiO_2 until kaolinite forms as a stable phase as the result of an Al_2O_3/SiO_2 ratio typical of this clay mineral.

Therefore, for the purpose of this research, the term 'lateritic soil' will be adopted because not all soils involved could fall into the category of true laterites.

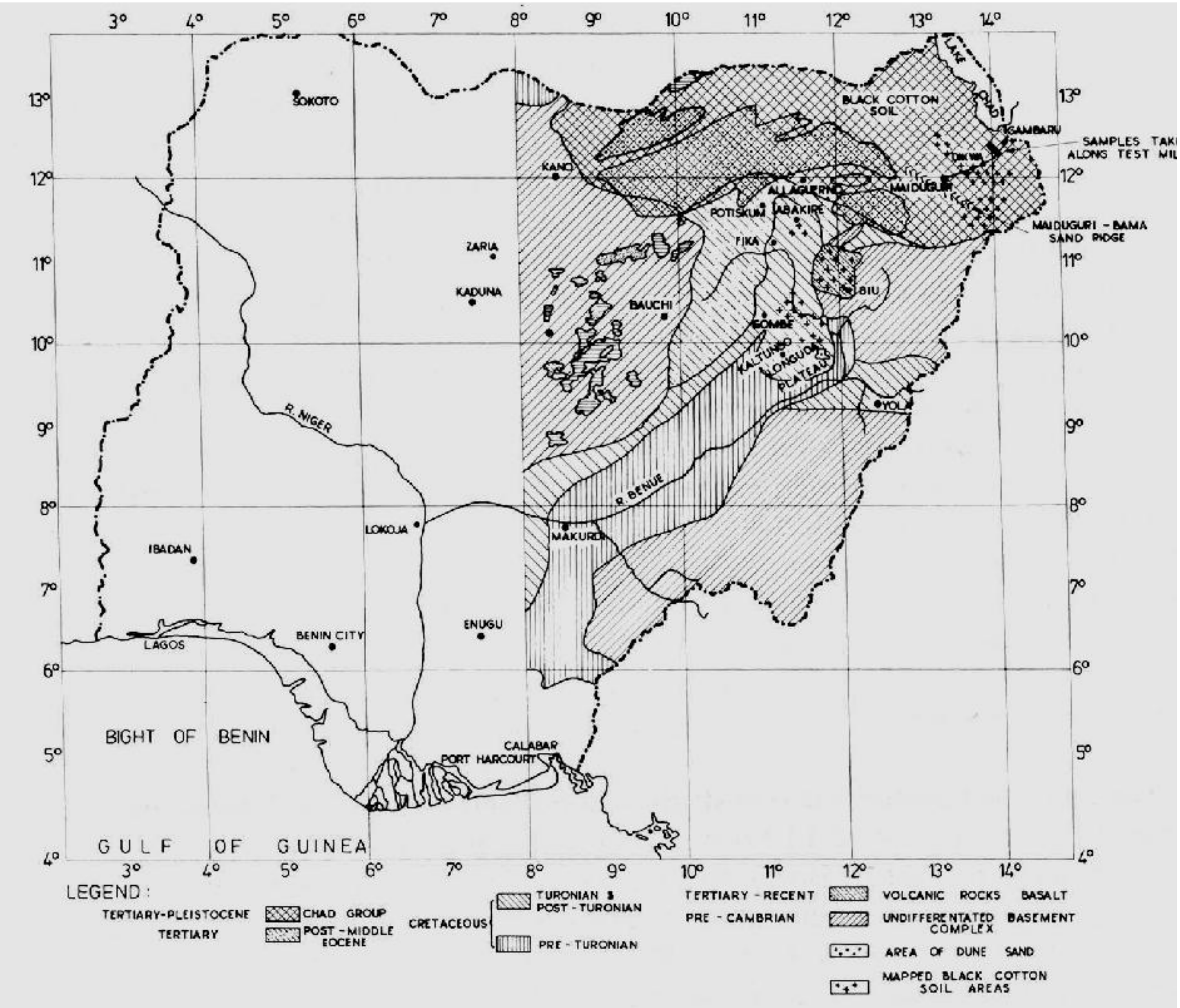
2.3.2 Formation of laterites/lateritic soils

In the tropics, the rainfall intensity (mean annual rainfall between 750 mm to 3 000 mm), coupled with a high temperature and a good drainage system, creates a distinct seasonal wet and dry climate which favours the formation of laterites. Laterites are weathering products formed on a variety of rocks containing a high amount of Fe content which may be igneous, sedimentary or metamorphic in origin. Levinson (1974) observed that the optimum conditions important in the formation of laterites may include high seasonal rainfall, high temperature, intense leaching, strong oxidising environment, subdued topography, long duration of weathering and a chemically unstable parent rock. He further noted that adequate supply of sesquioxide of iron which may originate from the underlying parent material or from an adjacent area of higher topography is essential in the process of formation.

High water content and high temperature cause intense chemical weathering that produces well-developed residual soils (De Vallejo and Ferrer, 2011). The geotechnical behaviour of these soils is controlled by mineralogical composition, micro-fabric and geochemical environmental conditions. When the iron (Fe) and aluminium (Al) content are high, laterites are formed. When the drainage is poor, black cotton soils may form, which is rich in smectite clays (Mitchell and Soga, 2005).

During the raining season, leaching of the parent soil takes place, while during the dry season capillary action transports solutions of leached ions to the surface from where it evaporates, with the salts left behind to be washed down the following wet season. Thus, the whole zone is progressively depleted of the more mobile elements such as sodium (Na), potassium (K) and cadmium (Cd). Olanipekun (2000) observed that the high proportion of Fe^{3+} oxides in laterites signifies a left-behind accumulation as a result of removal of silica and alkalis.

Ola (1978) identified some conditions responsible for the formation of black cotton clays, although they are considered as problematic soils, but their chemistry and formation classify them as lateritic soils. According to him, this type of soil is formed from basic igneous rocks with calcium rich feldspars (plagioclase group – Anorthite) and mafic minerals which are highly unstable. They are formed in conditions where evaporation exceeds precipitation, poor leaching occurs, alkaline conditions and retention of magnesium (Mg) and Ca in the soil. All these conditions prevail in the black cotton soil areas of Nigeria. The parent rock materials or geology greatly influences the formation of black cotton soils as is evident on Figure 2.3.



Source: Ola (1978).

Figure 2.3: ***Map showing distribution of black cotton soils in Nigeria***

MacLean (1990) explained the influence of relief on weathering processes. When there is a steep slope, the bulk of rainwater occurs as surface runoff with little infiltrating into the ground. The development of a deep weathering profile is prevented and the early products of weathering are removed by surface waters. As the slope becomes gentler, a larger proportion of rainwater infiltrates and leaches the soil. As the weathering process advances, the land surface becomes flat. Topography plays a significant role in the development of laterite profiles. It controls the drainage conditions and hence the proportion of rainwater that infiltrates and leaches the weathered zone.

Bourman and Ollier (2003) engaged Schellman (1981 and 1983) in a strong debate on the origin, chemistry and mode of formation of laterites. On the mode of formation, Bourman and Ollier stressed the significance of lateral movement of materials associated with the formation of ferricretes and other materials which were preferentially deposited in valleys, then followed by inversion of relief. By this, they noted that the ferricretes were now found commonly on the plateaus. Pain and Ollier (1995) and Ollier (1995) went further to argue, based on the evidence of the origin of ferricretes of northern India found on the basalts of the Deccan plateau, that the so-called 'laterites' are not formed by vertical chemical evolution, but by lateral concentration of iron. In as much as their hypotheses could be reasonably based on the environment they studied, the common process of formation of laterites in most parts of the world has to do with vertical chemical evolution in the form of leaching through percolating water and effect of intense tropical climate. Residual laterites of the south-western part of Nigeria are as a result of vertical chemical evolution due to weathering and intense tropical climate. This clearly differentiate them from ferricretes formed by lateral concentration of iron, as seen in other parts of the world.

2.3.3 Processes and factors of laterisation

Laterisation could be referred to as a weathering process that leads to the formation of lateritic soils (Gidigasu, 1974). This begins with the formation of cracks in rocks as a result of prolonged heating and cooling by rain and sun. The main stages in the process of laterisation according to Maignien (1966), Gidigasu (1972), Ola (1983) and Lecomte *et al.*, (2009) are:

- **Decomposition / Primary weathering**: This is a physiochemical breakdown of primary minerals and the release of constituent elements. This stage has to do with complete or partial breakdown (both physical and chemical) of primary minerals and subsequent discharge of small primary elements and sesquioxide gels. This could be referred to as the primary weathering stage as mentioned in Gidigasu (1972). In temperate conditions of low chemical and soil-forming activities this process does not proceed beyond the clay-forming stage, where such clay minerals tend to decompose into various forms of aluminous and iron oxides in relation to the nature of the weathering conditions under tropical climates with high temperature and rainfall.
- Gidigasu (1973) identified the next stage as **secondary weathering** or **laterisation**. This stage, according to him, involves partial or complete leaching of bases and combined silica; together with other chemical processes which includes dissociation, mobilisation, migration, partial regrouping among minerals and/or elements, and finally, immobilisation of the weathering elements according to the prevailing drainage

conditions coupled with the quality of percolating water. The level to which the second stage develops depends on the nature and extent of the chemical weathering of the primary minerals. In semi-tropical regions this process stops at the formation of clay minerals, while warm, humid climates, with dense vegetation cover the weathering, may be so intense and may carry on for so long that even the clay minerals, which are primary hydrous-aluminium silicates, are destroyed in the continued weathering. The silica is leached and the remainder consists merely of aluminium oxide such as gibbsite, or of hydrous iron oxides such as limonite or goethite derived from the iron.

- **Dehydration or dessication** is the third stage according to Gidigasu (1973), which involves a partial or complete dehydration of hydrated colloidal oxides of iron and aluminium. Laterite materials cover a large succession of reddish tropically weathered materials, varying from earthy to rocky materials which genetically form a chain of materials starting with fresh rock and ending with sesquioxide rock.
- **Leaching** is the final removal of combined silica and bases under appropriate drainage conditions and the relative accumulation or enrichment from outside sources of oxides and hydroxides of sesquioxides.

Gidigasu (1976) reviewed the geotechnical significance of the degree of leaching in tropically weathered residual profiles, noting that if the release of primary chemical constituents is not removed through leaching, the lateritic soils would not form because the removal of bases, and especially combined silica, causes further decomposition which finally leads to the breakdown of the aluminous silicate clay minerals and the formation of hydroxides of sesquioxides.

The pH of the groundwater and the drainage conditions equally determine the soil conditions under which the various elements are rendered soluble and removed through leaching. The extent of the chemical weathering of the primary minerals determines the level to which the leaching process develops.

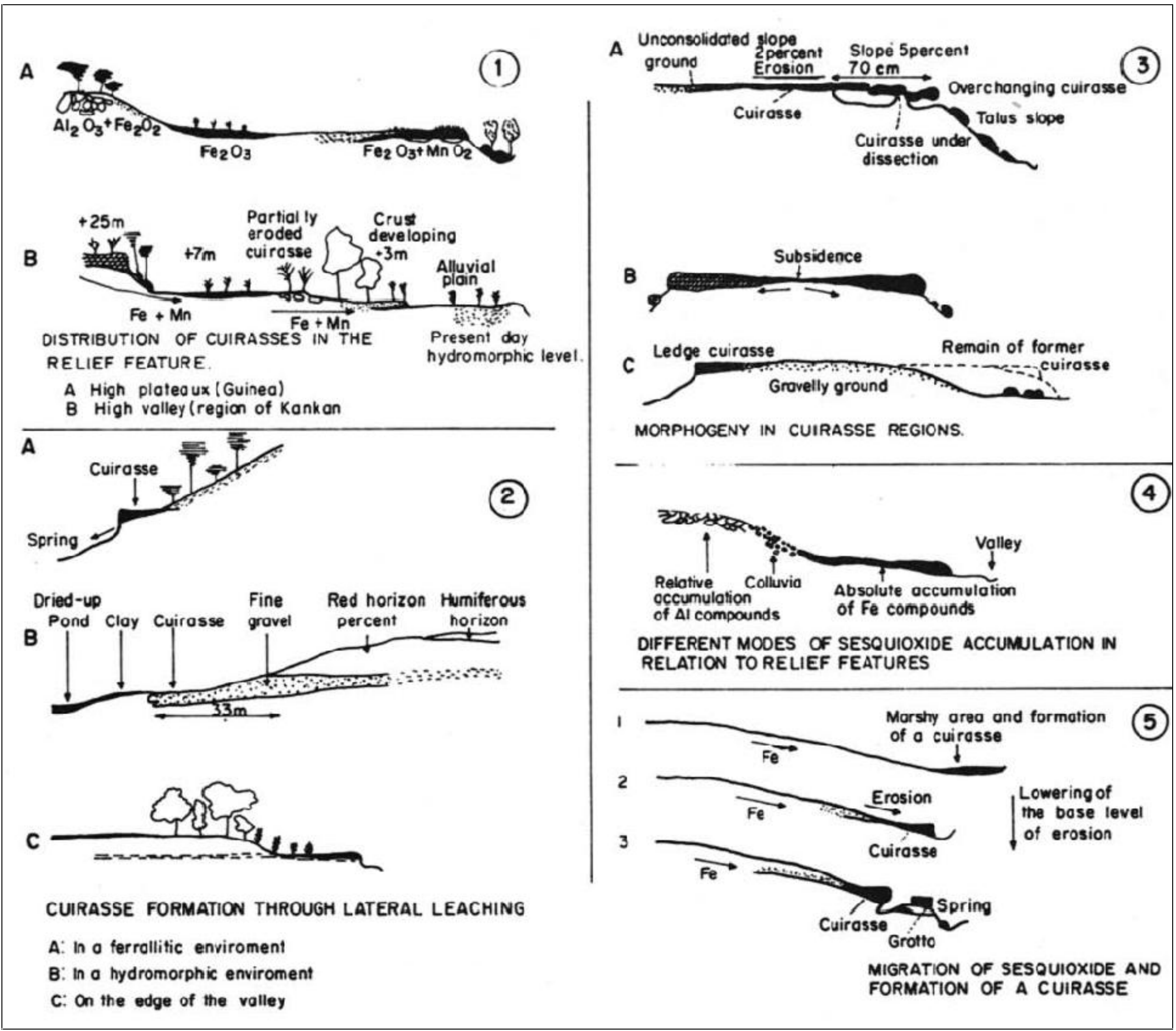
The ratio of the silica-sesquioxide of iron and aluminium in soils has, however, been used to assess the degree of laterisation of tropical soils. It follows that the lower the ratio the higher the degree of laterisation. Thus the higher the amount of kaolinite in the soils the higher would be the degree of laterisation (Winterkorn *et al.* cited in Gidigasu, 1976).

A further study by Bhatia *et al.* (1970) (cited in Gidigasu, 1973) on soil profiles and laterisation as illustrated in Figure 2.4, suggests that the formation of laterite profiles is either from removal in solution through leaching of combined silica and bases with the relative accumulation of sesquioxides, or the upward movement of sesquioxides with underground water to form high-level laterites and the lateral movement of sesquioxides to form low-level laterites. This, according to the research, depends on the sources of sesquioxides, local topography and drainage conditions. It was, however, shown that the extreme variations in the chemical and mineralogical composition of the parent rock lead to variation in their derivative soil type to tropical weathering. Gidigasu (1973), therefore, concluded that the development of such materials is favoured by some or all of the following conditions: availability of sesquioxide-rich parent material, average annual rainfall of 400 mm or more; existence of neutral or acidic soil conditions; good internal drainage and soil permeability; and alternate wet and dry seasons.

The rate at which laterisation proceeds is controlled by many factors, part of which are mentioned below:

- Climate.
- Topography.
- Vegetation.
- Time.
- Parent rock material.

A short discussion on each factor is given in the following sections.



Source: Gidigasu, 1972).

Figure 2.4: ***Typical topographical sites of indurated laterite profiles, including crusts***

2.3.3.1 Climate

Climate determines the amount of water present in a particular region. It determines the temperature and the character of vegetative cover. All these in turn affects weathering and leaching of elements. Some general influences of climate, as observed by Mitchell and Soga (2005) are listed below:

- The amount of rainfall controls the rate of chemical weathering. Chemical weathering proceeds more rapidly in a warm compared to a cool climate. Reaction rates at normal temperature, doubles by approximately 10 $^{\circ}$C rise in temperature.

- Provided there is good drainage, laterisation proceeds more rapidly in a wet climate than in a dry climate. This is responsible for better laterisation in tropics than in the arid regions. This is clearly seen in the formation of laterites in the south-western region of Nigeria, compared to the black cotton soils of the north-eastern part of the country.
- Depth to water table equally influences weathering by determining the depth to which air is available as a gas or in solution, this affects the prevailing biotic activity in the region.
- The intensity and type of rainfall affects the degree of laterisation and/or weathering. For instance, short intense rainfall usually erodes as surface runoff and probably ends up in the ocean, but light intensity coupled with a long duration of soaking in the soil, aid leaching of chemical constituents.

Fookes *et al.* (2000) (cited in Mitchell and Soga, 2005) gave detailed morphoclimatic zones and their associated geomorphologic processes. For instance, in a hot, wet, humid and tropical region with mean annual temperature of between 20-30 °C and a mean annual rainfall of greater than 1 500 mm, this kind of region is expected to have a high potential rate of chemical weathering, mechanical weathering limited, moderate to low rates of stream corrosion, but locally a high rate of dissolved and suspended load transport.

2.3.3.2 Topography and drainage

Adeyemi (2003) investigated the effect of topography on engineering and geological characteristics of sandstone derived lateritic soils from Ishara-Remo, south-western Nigeria. He concluded that it is better to win soil required for construction work from gently sloping terrains, since they are likely to possess better geotechnical properties than those from flat sites. This conclusion was reached because of higher amounts of kaolinite and ferric oxide in samples from gentle slopes than those from flat terrains, which is an indication of better drainage and a higher degree of laterisation in the former than the latter.

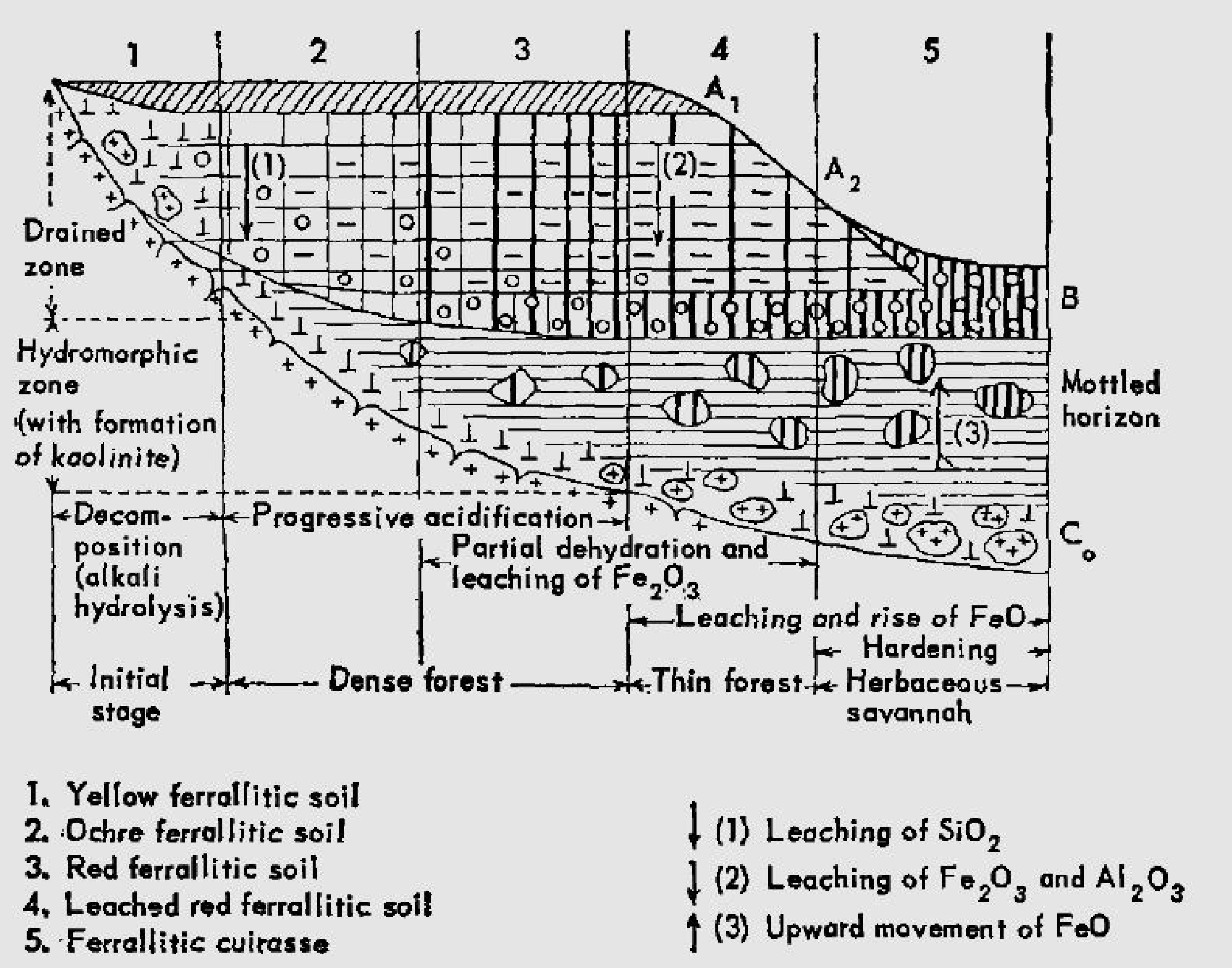
Topography influences climate and it controls the rate of erosion, which in turn affects the depth of soil accumulation. The depth of soil accumulation is greatly responsible for laterisation or weathering; this has a great effect on the quality of laterite or lateritic soil derived in a particular region. Topography equally controls drainage; good drainage conditions favour leaching and laterisation, while poor drainage conditions lead to formation of clay soil usually rich in smectite and swells when it comes in contact with water.

2.3.3.3 Vegetation

As already noted, laterites develop under conditions of high precipitation and forest vegetation. These factors affect the formation of a high or hilly relief and especially of a series of convex slopes which facilitates run-off. In a densely populated forest, supply of organic matter to the surface, according to Maignien (1966) is usually very large with a very important consequence on plant nutrition. These plants provide an excellent protection against heat and erosion. A dense vegetation reduces total radiation reaching the soil.

Aubert (1959) (cited in Maignien, 1966) is of the opinion that vegetation plays a protective role in thermal amplitude, that is, it reduces thermal amplitude.

All these conditions listed above aid in the formation of laterites. The response and reaction of each zone of vegetation to the formation of laterites and lateritic soils have been studied by Mohr and Van Baren, later modified by Duchaufour in 1960 (cited in Maignien, 1966). This is summarised in Figure 2.5.



Source: After Mohr and Van Baren, complemented and modified by Duchaufour (1960) (cited in Maignien, 1966).

Figure 2.5: Development of ferrallitic soils

2.3.3.4 Parent rock material

Description and detailed study of different soil profiles developed on different types of rocks have been presented by Maignien as far back as 1966. These profiles include the lateritic soils from different parts of the world, especially Africa and India (Aubert, 1954; Dommergues, Poulain and Moureax, 1962 and Fauck 1962). These were reported in Maigniens' review of laterites (1966:28-33).

Quartz-rich parent rocks produce gravelly materials containing coarser quartz particles than the concretionary one. (Gidigasu, 1974).

As earlier mentioned, the influence of parent material in the form of basic igneous rocks (for example basalt) was responsible for the formation of black cotton soils in the north-eastern part of Nigeria (Ola, 1978). Textural and mineralogical characteristics of parent material could be responsible for behavioural differences in the engineering properties of soils derived from genetically different parent rock material (Adeyemi, 1995).

The parent rock factor may cause a remarkable difference between the index properties and engineering properties, including strength characteristics of soils.

Adeyemi (1995) investigated soils derived from sandstone, migmatic-gneiss and quartz-schist around the Lagos–Ibadan expressway in south-western Nigeria. This was done with the aim of establishing the effect of parent rock factor on some basic engineering index properties of the derived soils. The study showed a significant influence of parent rock factors on the derived soils.

The influence of parent rock material on the formation of laterite was studied by Thorne *et al.* (2012) who found a significant impact of the parent rock material on the soil material. More recently, Giorgis *et al.* (2014) ascertained the influence of parent rock (bedrock) using a strong evidence to support the hypothesis of being the precursor rock of laterite – they reported that the Rare Earth Elements (REE) patterns measured in a set of laterite horizons closely matched that of the underlying basement. It was further reported that the shape of the chondrite normalised REE patterns of all lateritic horizons is virtually the same, thus implying a direct origin of the sequence from the underlying bedrock.

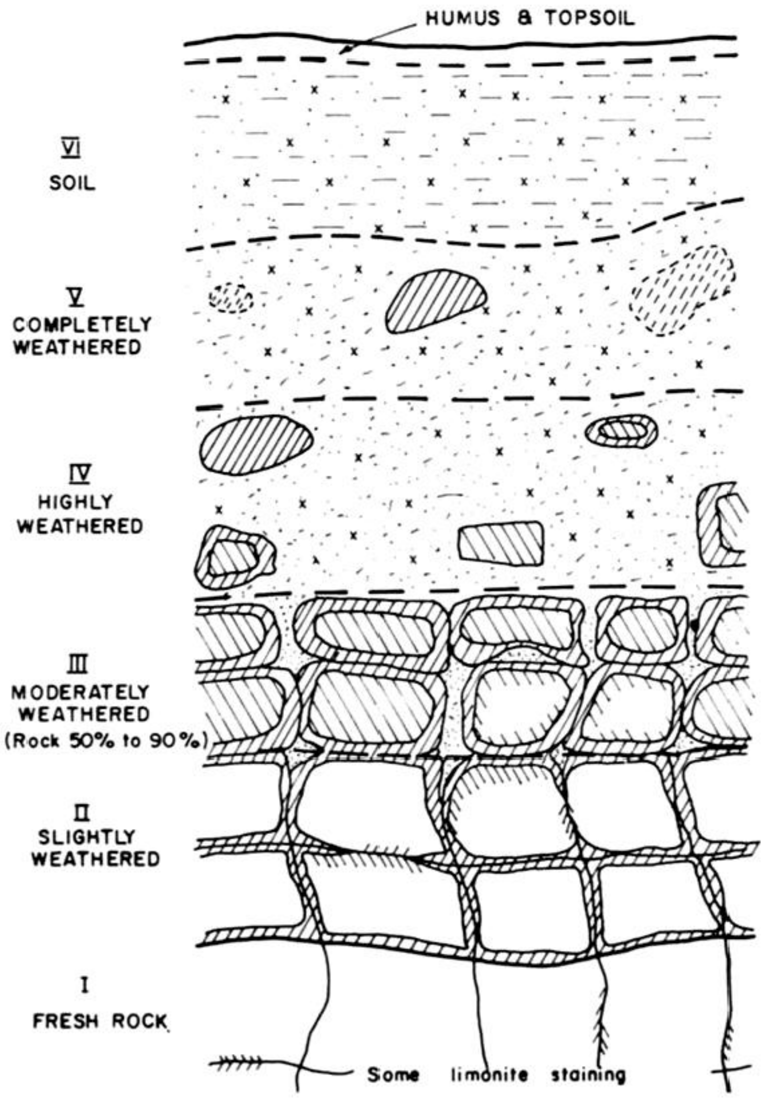
2.3.4 Lateritic profile

Generally, laterite profiles have been classified into three (Gidigasu, 1974), namely:

- Lateritic soils in which the crust is derived from the overlying soils (*i.e.* downward transport/leaching).
- Lateritic soils from underlying weathered rock (*i.e.* upward transport)
- Lateritic soils in which the crust-forming material is detrital (*i.e.* transport and deposited and or formed by precipitation)

The laterite developed *in situ* has a number of horizons. This varies in thickness, hardness, and colour. Hard crust results if the laterite soil profiles are dried out as a result of lowering of the water table.

Little (1969) (cited in Gidigasu, 1974) gave a simple model illustrated in Figure 2.6 of the degree of decomposition of rocks. This model perfectly suits field conditions encountered in the field. It shows a progressive weathering upwards with different horizons having different degrees of weathering.



Source: Little (1969) (cited in Gidigasu, 1974).

Figure 2.6: ***Morphological definition of degree of decomposition of rocks***

Kasthurba *et al.* (2007) reported a downward softening of material due to a decrease in sesquioxide cementation, and an increase in the proportion of clay filled vesicles. This brings about laterite profiles which are characterised by accumulation of sesquioxides at upper levels and kaolinitisation at the lower level.

2.3.5 Composition of laterite

Laterites are referred to as c–ϕ (C-Phi) soils. The cohesionless portion consists of gravel, sand and silt, while the cohesive portion includes fine particles usually in silt and clay size. The matrix is illustrated in Figure 2.7, based on a Unified Soil Classification system:

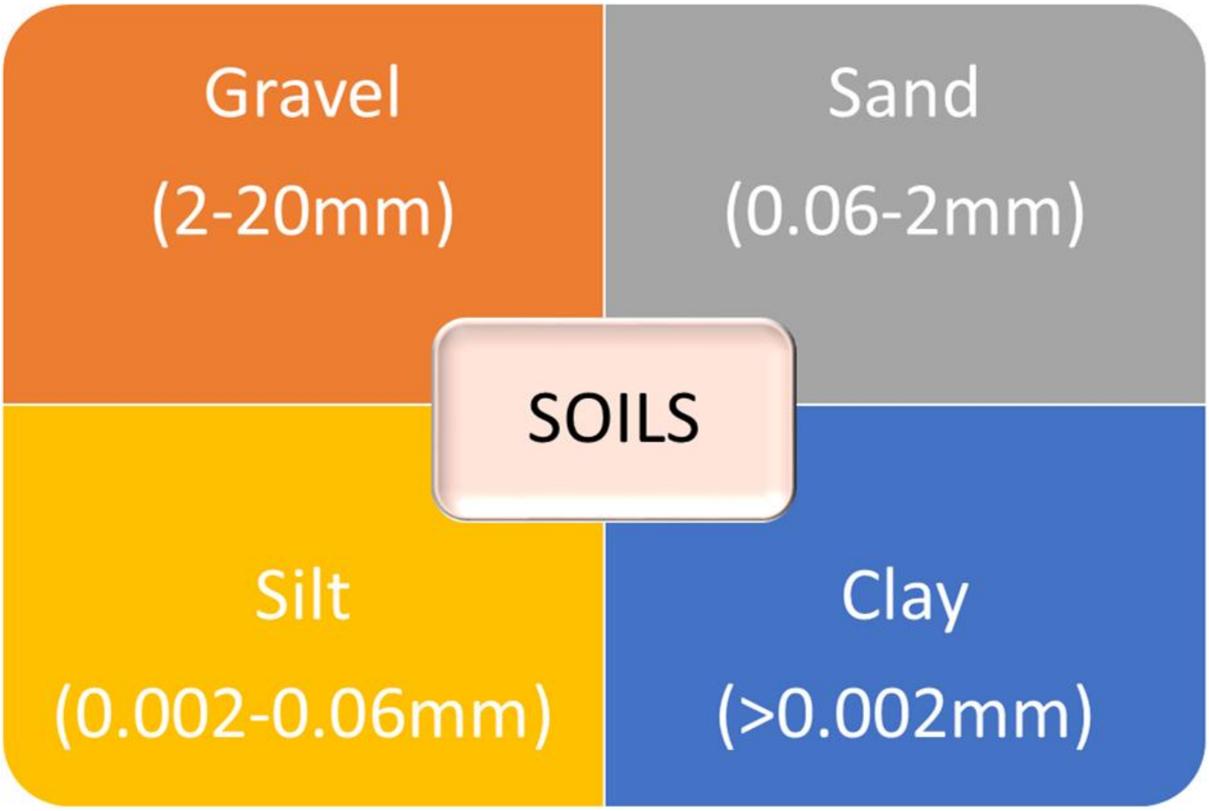


Figure 2.7: A typical lateritic soil matrix showing constituent particle sizes

Lateritic soils behave in a characteristic way: thus, for example, when exposed to variations in humidity, some will change in volume, others will not. Hence, some components of it are referred to as stable, *i.e* gravel and sand, while silt and clay are referred to as an unstable portion. Stability in this sense is based on their ability to withstand alternate wetting and drying without its properties changing, which is of course fundamental in materials for building construction.

Rigassi (1995) described the properties of each of these components of lateritic soil as follows:

- Gravels are composed of pieces of rock of varying hardness, whose size ranges between approximately 2 and 20 mm. These form the stable constituent of the soil with stable mechanical properties in the presence of water. It remains unchanged due to the presence of water.

- Sands are usually composed of mineral particles, whose size ranges between approximately 0.06 and 2 mm. Sand are equally stable constituents of the soil; though it lacks cohesion when dry, it has a high degree of internal friction, i.e. very great mechanical resistance to intra-particle movement. When wet, they display apparent cohesion as a result of the surface tension of the water present in their void spaces.
- Silts are made up of particles the size of which range between approximately 0.002 and 0.06 mm; their cohesion is little when dry and resistance to movement lower than that of sands. They display cohesion when wet. When exposed to different levels of moisture, they are sensitive to wetting and change perceptibly in volume. In a dry state, they have very poor cohesion and therefore cannot be used independently as main building material.
- Clays are the finest of the particle sizes of lateritic soil, usually less than 0.002 mm. Their characteristics differ completely from those of other particle sizes. They consist mainly of microscopic clay minerals which include: kaolinites, illites and smectites. Clay particles are coated in a film of absorbed water and because they are so minute, they are very light in weight compared to the surface tension forces occurring in the film of absorbed water.

Clays, unlike gravel and sand, are unstable and very sensitive to variations in moisture. Most are sharply attracted to water and as their moisture content rises, the films of absorbed water become thicker and the total apparent volume of the clay increases. On the other hand, during shrinkage, that occurs as they dry out, cracks can appear in the clay mass, thus reducing its strength. The cracks formed turns out to be pathways for water when next they are exposed to water. This is a major setback for clay being used independently as building material. Thus, a combination of the stable constituents, *i.e.* gravel and sand, with silt and clay forms good soil material for construction purposes. Lateritic soils thus appear to be suitable in this regard, because it is made up of all these different particle sizes in varying proportions.

2.3.6 Clays and clay mineralogy

Mitchell and Soga (2005) stated that clay could refer to both size and class of minerals. As a size, it refers to all constituents of soil smaller than a particle size of 0.002 mm (i.e 2 µm) in the engineering soil classification. While as a mineral, it refers to specific clay minerals distinguished by a small particle size, net negative electrical charge with plasticity when mixed with water and high weathering resistance.

Clay minerals are primarily hydrated alumino-silicate with a layered flaky structure. They usually occur as a platy elongated shape. As mentioned earlier, they are susceptible to shrinkage and swelling.

Clays are formed usually as a result of chemical weathering of silicate in rocks (silicates like feldspar, mica, amphiboles and pyroxenes) (Houben and Guillaud, 1994).

In terms of structure, large clay molecules. often referred to as micelles, are fine crystals of an irregular or hexagonal shape. Clay with hexagonal shapes are the most common, but other forms like cylindrical, thick tablets, pseudo-hexagonal, equally exists. Each micelle is made up of several sheets up to hundreds of sheets. The structure of these sheets determines the structure of the mineral. The sheets have a chemical makeup that varies according to the type of clay, degree of hydration, as well as thickness and spacing, usually from 7 to 20 Angstrom (Å). Size ranges between 0.01 and 1 µ.

Clay sheets are made of silica, *i.e.* atoms of silicon surrounded by oxygen atoms, and some made of alumina, *i.e.* aluminium atoms surrounded by oxygen atoms, and hydroxyl groups.

Clay structure is determined by the way in which the oxygen and the hydroxyl groups are arranged. It may be tetrahedral or octahedral.

There are three most frequently encountered clay minerals. These three are common clay minerals in lateritic soils, namely:

- Kaolinite.
- Illite.
- Montmorillonites.

2.3.6.1 Kaolinite

Kaolinite sheets are made up of a layer of oxygen tetrahedron with a silicon centre and a layer of oxygen (or hydroxide) octahedrons with an aluminium centre. Both are negatively charged at the edge of the sheet and has a low ion fixing capacity.

Kaolinite possesses a net negative charge with the particles charged positively on their edges when in low pH (*i.e.* acidic environments), but negatively charged in high pH (*i.e.* basic environment). Invariably, a low exchange capacity is measured in an acidic environment and high exchange capacity in basic environments (Mitchell and Soga, 2005).

2.3.6.2 Illite

This is commonly referred to as hydrous mica because of its structure which is similar to that of muscovite. This group has a three-layer structure, one mainly composed of aluminous octahedral layer, between two mainly silica tetrahedral layers. Mg^+ or Fe^+ may partially replace the Al ion in the aluminous layer, while Al^+ may substitute equally for Si in the silica layer.

A unit cell of illite is electrically neutral. Values greater than 10 to 15 meq/100 g may be indicative of an expanding layer (Weaver and Pollard, 1973, cited in Mitchell and Soga, 2005).

2.3.6.3 Montmorillonite

This is the most common mineral of the smectite group. It has a structure similar to that of illite, except that substitution takes place in the octahedral alumina layer. The aluminium ion may be replaced by any of following: Mg, Fe, Ni, Mn. The sheets are weakly linked and the ions between them are exchangeable cations (*e.g. Na*, *Ca*) and water molecules. Montmorillonite is not stable in contact with water, it swells appreciably more than illite. It has low volume forces compared to surface forces. Thus any film of absorbed water which sticks strongly to the clay layers, links the micro-particles of the soil together; this process gives clay its cohesion and most of its mechanical strength. This can only be eliminated by very advanced desiccation. Clay acts as a natural binding agent between the coarser particles which form its skeleton.

2.3.7 Geotechnical characteristics of lateritic soils

2.3.7.1 Particle size and distribution

In order to understand the properties of any soil and lateritic soil in particular, it should be classified according to particle sizes. Sieve analysis method is used for particles greater than 0.075 mm in size, while for smaller sizes sedimentation or a hydrometer test is employed (Houben and Guillaud, 1994).

Meanwhile, laterite has a good representation of both a cohesive and cohesionless portion of its particle sizes. Based on this, engineering soils could be divided into the following:

- Fine-grained soils (amount of fines) composed of silt and clay size particle. Predominantly, with varying degrees of plasticity as measured by their plasticity (Atterberg) limit, rather than by sieving and setting velocity methods.
- Coarse-grained soils (amount of coarse) composed of sand size and larger particles. They are separated into size ranges by sieving of materials up to cobble size and except for micro-fractions of plastic fines, they are characteristically non-plastic.

Well-graded soils are expected to compact to a lower porosity and permeability; thus resulting in better engineering properties than uniformly graded soils (Oyediran and Adeyemi, 2011, cited in Oyediran and Okusun, 2013). The grain size classification results are important due to the fact that particle size and measure of plasticity are widely used as basis for engineering soil classification. It is a most versatile and universally accepted soil classification (Gidigasu, 1974; Oyelami and Van Rooy, 2016b). Details of soil classification is given in the subsequent section.

Some of the very important parameters that could be deduced through particle size analysis and distribution include (González de Vallejo and Ferrer, 2011; Mitchell and Soga, 2005):

- **Coefficient of uniformity (Cu):** This implies the slope of a grain size distribution curve.

$$Cu = \frac{d60}{d10}$$ *Equation 2.1*

Where d_{60} and d_{10} correspond to the sieve sizes that passes through 60 and 10% particle weights, respectively. Generally speaking, a soil with C_u between 5 and 10 is considered well-graded.

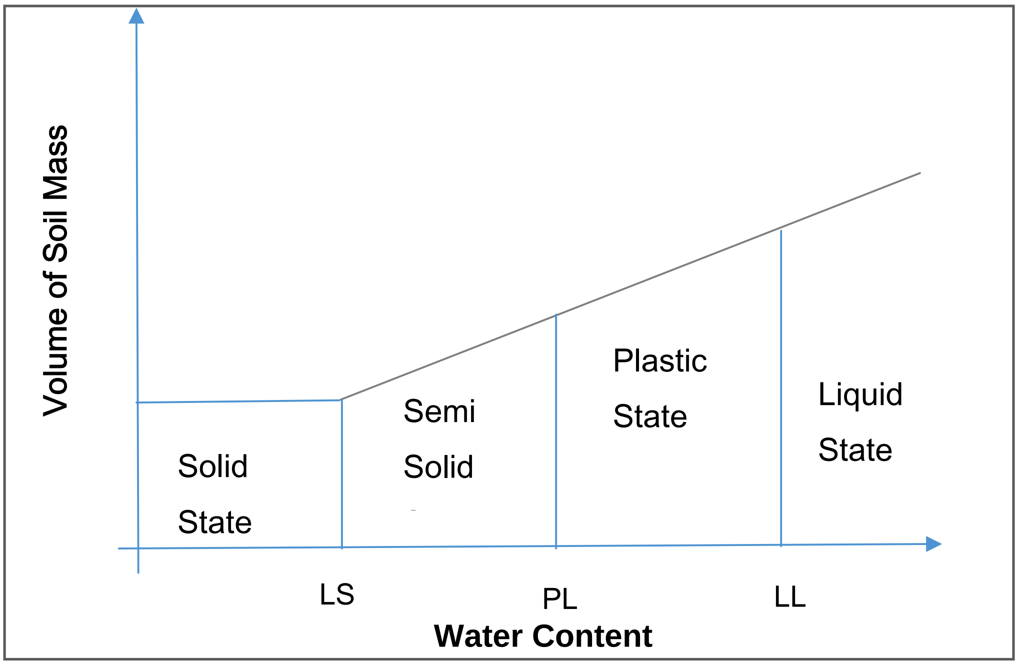
- **Relative Density, D_R**: Relative density measures the *in-situ* void ratio in relation to the maximum and minimum void ratios:

$$D_R = \frac{emax - e}{emax - emin}$$ *Equation 2.2*

2.3.7.2 Plasticity/Compressibility characteristics

The ability of a soil to undergo deformation without failing due to elasticity is referred to as plasticity. It refers to the ease at which a soil could be deformed without being cracked or disintegrated. It is otherwise known as the Consistency Limit or Atterberg Limit.

Ramamurthy and Sitharam (2014) explained soil consistency as follows: A soil in solid state experiences no change in volume of soil mass due to change or addition of water. In the remaining three conditions as illustrated in Figure 2.8, addition of water will lead to increase in volume of soil mass and removal of water leads to reduction in soil mass as well. Soils in liquid state behave like liquid with a very poor shear strength, in plastic state it could be deformed without being cracked, and in semi-solid state it cannot be deformed without being cracked.



Source: Modified after Ramamurthy and Sitharam (2014)

Figure 2.8: States of consistency

Terzaghi (1925) (cited in Mitchell and Soga, 2005) noted as follows on Atterberg limits: *"the result of the simplified soil tests (Atterberg limit) depend precisely on the same physical factors which determine the resistance and the permeability of soils (i.e. shape of particles, effective size, uniformity) only in far more complex manner."*

According to Houben and Guillaud, (1994) a liquid limit of 25 - 51 and plasticity index of between 2 - 31 was given as a range for soil in earth building. African Regional Standard (ARS) recommended a liquid limit range of about 25 - 50 and plasticity index range of about

2.5 – 29 for soils to be used for CEBs. Adam and Agib (2001) noted that soils with high plasticity could be very difficult to handle when mixed with water.

The greater the plastic limit, the greater the swell when the soil comes in contact with water (Houben and Guillaud, 1994). Millogo et al. (2008) attributed a decrease in compressive strength of soil to the presence of clay because of their plasticity, hence he recommended a careful study of swelling potential of soils according to their plasticity index, liquid limit and the percentage of clay fraction. Their different swelling potentials are given in Table 2.1.

TABLE 2.1: SWELLING POTENTIAL OF SOILS BASED ON THEIR ATTERBERG LIMITS AND CLAY FRACTIONS

PI (%)	<2 µm (%)	<74 µm (%)	LL	Swelling Potential
>35	>95	>95	>60	Very high
22-35	60-95	60-95	40-60	High
18-22	30-60	30-60	30-40	Moderate
<18	<30	<30	<30	Low

Source: After Millogo *et al*. (2008a).

The importance of compressibility was stressed in Rigassi (1995) as a parameter that defines the maximum capacity of a soil to be compressed at a particular compaction energy and moisture content. This clearly indicates the importance of a significant number of fine particles (clays and silts) in any particular soil to be used as material in the production of CEBs.

Casagrande worked extensively on the soil classification system following which in 1932, he developed a standard device for determination of liquid limits. He observed that non-clay minerals, like quartz and feldspar, do not develop a plastic mixture with water, even at the least of particle size. In 1948, he developed a soil classification system based on the Atterberg limit particularly for the identification of cohesive soils. His work was adopted with a little modification as part of the Unified Classification System (Casagrande, 1932; 1948, cited in Mitchell and Soga, 2005).

2.3.7.3 Specific gravity

De Graft Johnson (1969) noted that specific gravity is a measure of the degree of laterisation in tropical soils, He concluded that specific gravity of lateritic soils falls within a wide range of about 2.60 to 3.40. Coarser particles having a higher value than the finer particles. Specific gravity depends significantly on the factors which are size of samples, position of samples in soil profiles, grading characteristics, mineralogical composition of parent rock materials, iron content and accumulation of titaniferrous materials.

Winterkorn et al. cited in Gidigasu (1976) and Adeyemi, (1994), established a negative correlation between silica–sesquioxide ratios and the amount of kaolinite in assessing the degree of laterisation of tropical soils. It was concluded that the lower the ratio, the higher the degree of laterisation. It follows invariably that the higher the amount of kaolinite in the soils, the higher would be the degree of laterisation. The specific gravity has been found as a useful identification and evaluation index of lateritic soils, especially in construction since it correlates well with mechanical strength of soil (Adeyemi and Wahab, 2008). This is related to evaluation of soils as materials in earth building, especially as it relates to determination of the degree of laterisation.

2.3.7.4 Shear strength characteristics

Shear strength is the main structural property of soils in providing support, ability or bearing capacity of rock and soils, it also permits a slope to be stable (Oyelami and Alimi, 2016). Shear strength is dependent on factors such as the nature of the soil, the soil structure, bonds and the degree of deformation, particularly in terms of stress and fluid pressures in the pore spaces (De Vallejo and Ferrer, 2011). Shear strength could be expressed mathematically as shown in Eqs. 2.3 and 2.4. Some of the laboratory shear strength tests include direct shear tests, unconfined compressive tests and triaxial tests. Shear strength characteristics of engineering soils can be divided into two, which are cohesive soils (clays and clayey soils) and cohesionless soils (sands and gravels).

Shear strength could be expressed mathematically as:

$$\tau = c + P \tan \phi \qquad \textit{Equation 2.3}$$

or

$$\tau = c + (\sigma_n - u) \tan \phi \qquad \textit{Equation 2.4}$$

Equation 2.4 above is referred to as Mohr Coulomb’s equation.

where

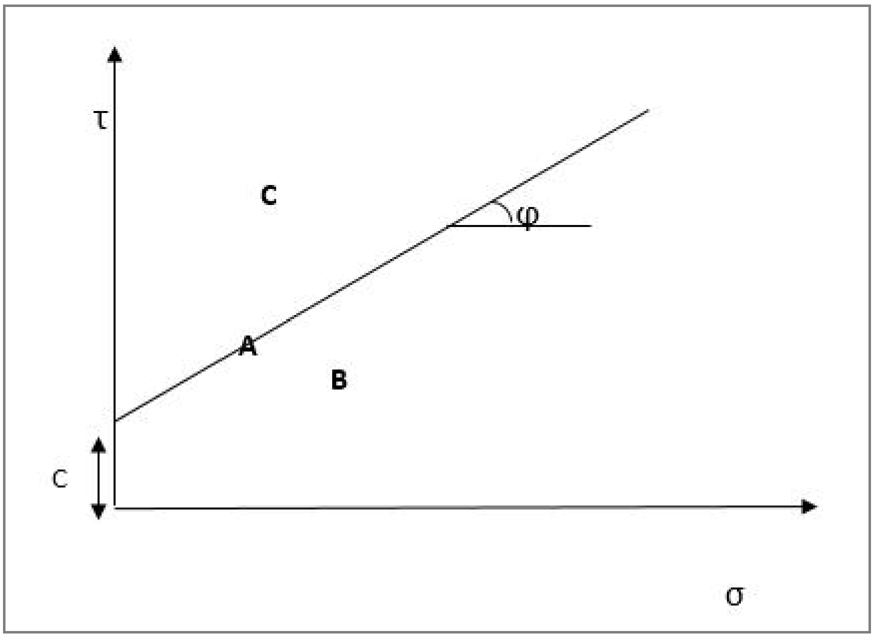
c is cohesion,

ϕ is angle of internal friction.

P is effective stress given as total normal stress (σ_n) acting on a plane *minus* pore water pressure *(u)*.

$$P = (\sigma_n - u) \qquad \textit{Equation 2.5}$$

Equation 2.4 is represented by a straight line called the failure envelop of a soil as shown in Figure 2.9.



Source: Modified after De Vallejo and Ferrer (2011).

Figure 2.9: A failure envelope

Point A on the graph represents a state of failure, while point B represents possible states with a certain factor of safety. Point C is an impossible state; it lies above the envelop which implies that it has exceeded the combination of failure criterion.

Millogo *et al.* (2008b) on the geotechnical properties of laterites in Burkina Faso, gave the following result: an optimum moisture content (OMC), the maximum dry density (MDD) and the CBR at 95% of MDD (6.6%, 21.7 kN/m^3 and 43%, respectively). He went further to describe the strain–stress curves of the soils, which according to him, shows a ductile type of rupture mechanism with a well-marked plastic phase. The samples were described as nearly flexible with its compressive and tensile strengths given as 1.26 and 0.09 MPa, respectively. In general, compressive strength falls within the range 0.5–1.5 MPa. This kind of soil possesses an excellent geotechnical property and it could be suitable for different engineering construction works.

It has been established in previous literature that clays are always detrimental to geotechnical properties of soils (Adeyemi and Wahab, 2008; Oyediran and Okosun, 2013; Oyelami and Alimi, 2016; Oyelami and Van Rooy, 2016a, 2016b). This invariably means that the lower the clay content, the better the soil for engineering construction. But this statement should be understood carefully, because soil as construction material depends not only on the geotechnical properties, but at times a careful evaluation must be made based on the purpose of the project and specific function of the clay material in the soil (Oyelami and Van Rooy, 2016b). For instance, in the building construction industry in some countries (e.g. South Africa), clay material is preferred in brick making, this is possible because the bricks are subjected to firing after the whole process of moulding. Equally, clays especially the montmorillonite group, may be useful in landfill and dumpsite construction, where their swelling properties will be useful in containing the leachates and preventing rapid percolation and interaction of leachates with the environment (Oyelami and Van Rooy, 2016b)

Oyediran and Okosun (2013) established a positive influence on the geotechnical properties of lateritic soils from south-western Nigeria, with the application of lime as stabiliser. It was discovered that the Unconfined Compressive Strength (UCS) of the soils increased significantly with addition of about 6% lime, which brings about more than 100% increase in the UCS of the soils. Other index properties of the soils were appreciably improved by the addition of lime as well.

It is then certain that geotechnical properties of soils for construction materials could be improved significantly by stabilisation. This could be achieved either by physical stabilisation in the form of addition of more suitable soil to a less suitable one, mechanical stabilisation in the form of compaction, compression or consolidation and chemical stabilisation in the form of addition of lime or cement.

Adeyemi and Wahab (2008) investigated the variation of geotechnical properties of residual lateritic soils in South-western Nigeria. The soil samples were taken from a restricted area of about 39 m^2. The soils were found to exhibit medium plasticity with low linear shrinkage; it possesses high strength parameters in terms of CBR and UCS. It was equally found that the geotechnical properties of these soils vary even within the shortest distance possible. A summary of the geotechnical parameters is listed in Table 2.2 (Adeyemi and Wahab, 2008).

TABLE 2.2: SUMMARY OF SOME GEOTECHNICAL PARAMETERS OF LATERITIC SOILS IN SOUTH-WESTERN NIGERIA

Test no.	OMC at M.A. level (%)	MDD at M.A. level (Mg/m^3)	Unsoaked CBR (%)	Soaked CBR (%)	Uncured unconfined compressive strength (kN/m^2)	Cured unconfined compressive strength (kN/m^2)
1	17.0	1.94	11.02	12.33	200.00	807.69
2	17.3	2.09	7.52	9.92	191.67	815.38
3	9.7	2.03	98.20	24.75	562.00	1006.67
4	10.3	2.05	95.89	23.34	534.00	1013.34
5	14.2	1.83	38.33	38.52	622.00	1023.08
6	12.9	1.81	41.83	45.32	646.15	1038.46
7	12.1	1.93	58.16	39.83	630.77	976.92
8	12.6	1.91	60.05	43.43	630.77	992.31
Average	13.26	1.95	51.38	29.74	502.17	959.23

Source: After Adeyemi and Wahab (2008).

2.3.7.5 Compaction characteristics

Compaction is a process during which the volume of soils is reduced due to expulsion of air present in voids and rearrangement of particles resulting in closer packing. In compaction, densification of cohesionless soils occur. Compaction is achieved as a result of dynamic load, *i.e.* mechanical energy which often results in soil compression. It increases the dry density of a soil, thereby leading to increase in soil strength and bearing capacity, while it reduces compressibility, volume change, permeability and susceptibility to frost action. It also modifies elasticity of the soil.

The amount and percentage of fine particles plays a positive role in compaction. When they are sufficiently present in a soil, they tend to fill the void spaces between the coarse particles and maximum bulk density can be achieved. This dry density is dependent on water content, amount and type of compaction. The amount and type of compactive effort are determined by the energy level at which the soil is compacted. For a given energy level, soil mass attains a MDD at a specific moisture content. The specific moisture content is referred to as OMC.

The American Association of State Highways and Transportation Officials (AASHTO) recommended two energy levels of compaction based on the compactive effort, standard and modified AASHTO, but based on the findings of Gidigasu (1976) (cited in Adeyemi, 2000), it was observed that these input energy levels may cause over-compaction and hence excessive degradation of the grains in tropical soils. Hence, researchers around south-western

Africa, of which Adeyemi (2000) is one, adopted a third energy level known as the West African level of compaction. Therefore, in terms of their input energy levels, we have the following:

- Standard AASHTO (Proctor) Least energy level
- West African Level (1970) Intermediate energy level
- Modified AASHTO (World War II) Highest level

Some researchers within the West African region prefer the modified AASHTO. For instance, Owolabi (1991), while studying lateritic sub-base soils along the Ile-Ife/Sekona Oshogbo road in Nigeria, reported better geotechnical properties of soils compacted at modified AASHTO than those compacted at West African levels. This could be as a result of a difference in particle sizes, and due to the fact that these study areas are on different geological bedrocks, their properties are expected to vary considerably as well. Equally, the presence of mica has the effect of reducing the MDD and increasing the OMC for a given compactive effort.

When a soil has a fixed moisture content, an increasing amount of compaction results in closer packing (densification) of the soil particles, and an increase in dry density under further compaction efforts does not produce any further change. Moisture density is influenced by size of mould, nature of soil, temperature, type of compaction efforts (Villamizar *et al*., 2012).

Gidigasu (1976), due to the spring action of the flaky minerals, identified factors affecting compaction. He classified them into two:

- **Process of laterite formation:** These are generic and compositional factors such as nature of the soils, particle size, mineralogical composition, plasticity and amount of clay size content in the fines.

- **Pre-test preparations and testing procedures:** Air-dried samples seem to take particles under the impact of the rammer. Recompacted soils tend to give a consistently higher MDD thus making workability of soils easier.

In terms of brick making, Rigassi (1985) noted that compaction could be related to compression. When a force is applied to a quantity of soil, the material is compressed and the proportion of voids decreases. The more the density of a soil, the lower the porosity. This property results from the tighter overlapping of the particles, which lowers the risk of the structure being modified in the presence of water. Schroeder (2012) suggested that OMC with a defined compaction energy helps in identifying the maximum weight of a soil per volumetric unit, whereas, Delgado and Guerrero (2007) recommended a water content verification test by *'drop test'*, which is about dropping a handful of moist soil from a height of about one metre from the ground and interpreting the level of moisture by the shape it forms as it breaks on the ground.

A maximum density of 2 300 kg/m^3 was recommended for CEBs (Delgado and Guerrero, 2007).

Finally, it is noteworthy that compaction is different from consolidation, in that compaction is achieved by a dynamic load, while consolidation is achieved under a static or sustained load (Mitchell and Soga, 2005). Consolidation gradually reduces volume of soil through expulsion of pore water; compaction reduces volume of soil mass suddenly due to expulsion of air and water in pore spaces.

2.3.8 Classification of Lateritic soils

Soil classification is very important in the arrangement of soils into specific groups based on their physical properties and engineering behaviour with the aim of finding their suitability for different engineering purposes and applications (Ramamurthy and Sitharam, 2014).

Original texture classifications were based on the relative percentages of sand, silt and clay-size fraction. In identification, Atterberg (1911) proposed the use of plasticity tests as well as a dry strength concept, which emphasised pertinent physical characteristics whose differentiation is not possible by using particle size distribution alone. The plasticity tests were standardised by Casagrande (1948) for the identification and classification of fine-grained engineering soils. The extreme value of the plasticity concept has been emphasised by the position of the various soils on the chart. For instance, kaolinitic clay soils and bentonite may plot at the same point on the textural triangular chart, whereas their positions may be widely different on the plasticity chart, thus necessitating a combination of the two concepts in engineering soil classifications.

In terms of the use of lateritic soils in earth building, research and building standards agreed on the need for a minimum clay content in soil material for building constructions. Prominent among the standards are the African Regional Standard Organisation (ARSO) (Centre for Development of Industry, ARSO, 1998) the United States Office of Internal Affairs (OIA)

Handbook for Building Homes of Earth (Wolfskill et.al, 1970), and the *Australian Earth Building Handbook*, HB 195, (Australia Standards and Walker, 2002). Houben and Guillaud (1994) recommended a maximum clay content of about 23% for CEB. Rigassi, (1995) recommended a range of 6–30% clay, 10–25% silt, 25–80% sand and 0–40% gravel. Other recommendations cited in Delgado and Guerrero (2007) for CEBs fall within the range of 5–40% clay, 10–30% silt, 25–80% sand and fine gravel.

2.3.8.1 Textural-plasticity soil classification

This classification type is as a result of particle size and measure of plasticity being widely used as engineering soil classification. It is most versatile and a universally accepted soil classification. Based on this, engineering soils could be divided based on particle size as mentioned in section 2.3.7.1.

They are separated into size ranges by sieving of materials up to cobble size and except for micro-fractions of plastic fines, they are characteristically non-plastic. (See section 4.3.3). Mitchell and Soga (2005) noted that well-graded cohesionless soils are relatively easier to compact to a high density by vibration, while the loss of fine fraction by internal erosion could lead to large changes in the engineering properties of the soil.

2.3.8.2 AASHTO soil classification

AASHTO, formally known as the Highway Research Board (HRB) classification system (Ramamurthy and Sitharam, 2010), is used to determine the suitability of soils as subgrade materials in pavement construction, but not limited to that, it could still be used in establishing suitability of soils as materials in other engineering constructions, such as building, dam construction, etc. It employs particle size ranges and Atterberg limits for assigning soils into seven major groups (A-1 to A-7) and several other sub-groups. (See table 2.3 below.) This classification is more specific than the unified soil classification (USC) system in the limits placed on size ranges, amount and ranges of liquid limits and plasticity indexes for fines.

AASHTO utilises what is known as the group index to describe the performance of a soil as a subgrade material. The higher the value of the group index, the poorer the quality of material.

Group index of soils according to Ramamurthy and Sitharam (2014) depends on:

- the amount of material passing 75µ sieve;

- liquid limits; and
- plastic limits.

TABLE 2.3: AASHTO SOIL CLASSIFICATION SYSTEM

General classification	Silt-clay materials (more than 35% of total sample passing No. 200)			
Group classification	*A-4*	*A-5*	*A-6*	*A-7* *A-7-5** *A-7-6*†
Sieve analysis (percent passing)				
No. 10				
No. 40				
No. 200	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 40				
Liquid limit	40 max.	41 min.	40 max.	41 min.
Plasticity index	10 max.	10 max.	11 min.	11 min.
Usual types of significant constituent materials	Silty soils		Clayey soils	
General subgrade rating	Fair to poor			

*For A-7-5, $PI \leqslant LL - 30$
†For A-7-6, $PI > LL - 30$

Source: Ramamurthy and Sitharam (2014)

2.3.8.3 Unified soil classification system

This classification was invented by Casagrande. This was later adopted jointly by the United States Corps of Engineers and the United States Bureau of Soils during the1950s. In this system, soils are broadly divided into the following:

- Coarse-grained soils – if more than half by weight is retained on the No. 200 American Society of Testing and Materials (ASTM) sieve.
- Fine-grained soils – if more than half by weight passes through the No. 200 ASTM sieve.
- Organic soil.

The soil components are assigned group symbols as indicated below:

Coarse-grained soils:	Gravel:	G
	Sand:	S

Fine-grained soils:	Silt:	M
	Clay:	C
	Organic soil:	O

Further division of silts and clay are primarily by liquid limit values and degree of plasticity.

2.4 COMPRESSED EARTH BRICKS

2.4.1 Definition of compressed earth blocks/bricks

Compressed earth blocks (CEBs) is defined as *“masonry elements, which are small in size and which have regular and verified characteristics obtained by the static or dynamic compression of earth in a humid state followed by immediate demoulding”* (Stulz and Mukerji 1993).

The nomenclature 'Compressed Earth Blocks' in Earth Science may portray another meaning, this is because ‘Earth Blocks' refer to something else in Plate Tectonics. Based on this fact, for the purpose of this research, which is in line with earth sciences, the term '**Compressed Earth Bricks**' would mean the same thing as the commonly used term '**Compressed Earth Blocks**'.

Villamizar *et al.* (2012) referred to Compressed Earth Bricks as: *“one of the oldest identifiable man-made building materials due to their simplicity and low cost, good thermal and acoustic properties, which at the end of a building’s life, could easily be reused by grinding, wetting or returned to the ground without any interference with the environment*”. The use of CEBs was described as an accomplishment and fulfilling the necessities of eco-development.

CEBs generally come in a rectangular parallel piped shaped and are either solid, perforated or with vertical and/or horizontal indentations (Figure 2.10 and 2.11).

Adam and Agib (2001) considered compressed earth bricks with a size of 290 mm long, 140 mm thick and a height of 90 to 100 mm as an acceptable size. In as much as this is true, to a large extent CEBs come in different shapes and sizes based on purpose and the kind of press. The bricks shown in Figure 2.10 are small CEBs, while Figure 2.11 shows a big interlocked bricks that requires no mortar for wall setting.

Source: Author's own (2016).

Figure 2.10: Small-sized compressed earth bricks

Source: Author's own (2016).

Figure 2.11: Big-sized interlocking compressed earth bricks

CEB's strength characteristics and cohesion can be enhanced by the addition of additives. The final properties of CEBs are dependent on the following factors: kind/quality of raw materials utilised (for example the kind of stabiliser, soil) and the steps and expertise in executing various stages of manufacturing, i.e. the preparation of materials, addition and mixing of stabilisers, compaction or compression up to the curing stage.

2.4.2 Strength characteristics of compressed earth bricks

The compressive strength of CEBs depends on the soil type, type and amount of stabiliser, and the compaction energy used to form the block (Adam and Agib, 2001). In addition to this, their strength and durability equally depends on parent rock material, their grain size characteristics, soil mineralogy as well as method and duration of curing.

Maximum strengths (usually measured in MN/m^2 or MPa) are obtained by proper mixing of suitable materials and proper compacting and curing. In practice, typical wet compressive strengths for compressed earth bricks may be less than 4 MN/m^2 (Adam and Agib, 2001).

Calcium lime stabilised Sudanese black cotton soils have been reported by Adam and Agib (2001) to give a wet compressive strength of about 6–8 MN/m^2. This strength is suitable for many building purposes. It equally satisfies the minimum British Standard requirements of 2.8 MN/m^2 for precast concrete masonry units and load-bearing fired clay blocks, and of 5.2 MN/m^2 for bricks. He further recommended a compressive strength of about 1–4 MN/m^2 for building with small loads, for instance, a single-storey building.

Arumala and Gondal (2007) on the suitability of on-site soil for compressed earth bricks carried out on cohesive and non-cohesive soils from different locations in the United States, recommended a soil mixture of 8% sand, 87% clay and 5% ordinary Portland cement for making CEBs. This recommendation is highly controversial because of the very high clay content, which could have detrimental effects on the strength and durability properties of the bricks. The effects were apparent on the bricks as shown in their response to climatic conditions after three months (Figure 2.12).

Source: After Arumala and Gondal (2007)

Figure 2.12: Response of compressive earth bricks to climatic conditions after three months

In the work of Aguy (2011) on the engineering characteristics of CEBs, he found that the performance of clay bricks in terms of their compressive and flexural strength were greatly improved by mixing clay with sand, which became better by adding lime and Rice Husk Ash (RHA). He observed that clay–sand mixed specimens absorb less water compared to the stabilised clay specimens. Results of his finding is summarised in Table 2.4. His work confirms that the stabilised clay–sand mix is more resistant and impermeable to moisture/water than the stabilised clay specimens. Invariably, this goes further to prove that well-graded soil such as lateritic soils, are more durable than cohesive soils.

TABLE 2.4: COMPRESSIVE STRENGTH AND LOSS OF STRENGTH OBSERVED IN CEBS

Compressive strength (MPa) and strength loss (%).

Lime:RHA ratio	Dry		Wet		Strength loss	
	Clay	Clay–sand	Clay	Clay–sand	Clay	Clay–sand
0	13.3	11.2	–	–	–	–
1:3	16.7	14.9	10.4	12.0	38	20
1:2	18.1	17.7	11.8	15.1	35	15
1:1	20.7	18.6	15.5	16.1	25	13
2:1	17.3	16.6	14.8	15.2	15	9
3:1	15.4	13.0	13.8	12.4	11	5

Source: After Aguy (2011).

From Table 2.4, clay compressive strength seems higher than that of clay–sand mixtures in dry state, but then, at wet conditions, clay–sand has the best strength parameters. The implication is that compressive strength has little to do with durability, a brick may possess a very high compressive strength, but it may not stand the test of time in terms of climatic/weather conditions and moisture absorption. This is the case with the bricks tested by Arumala (2007).

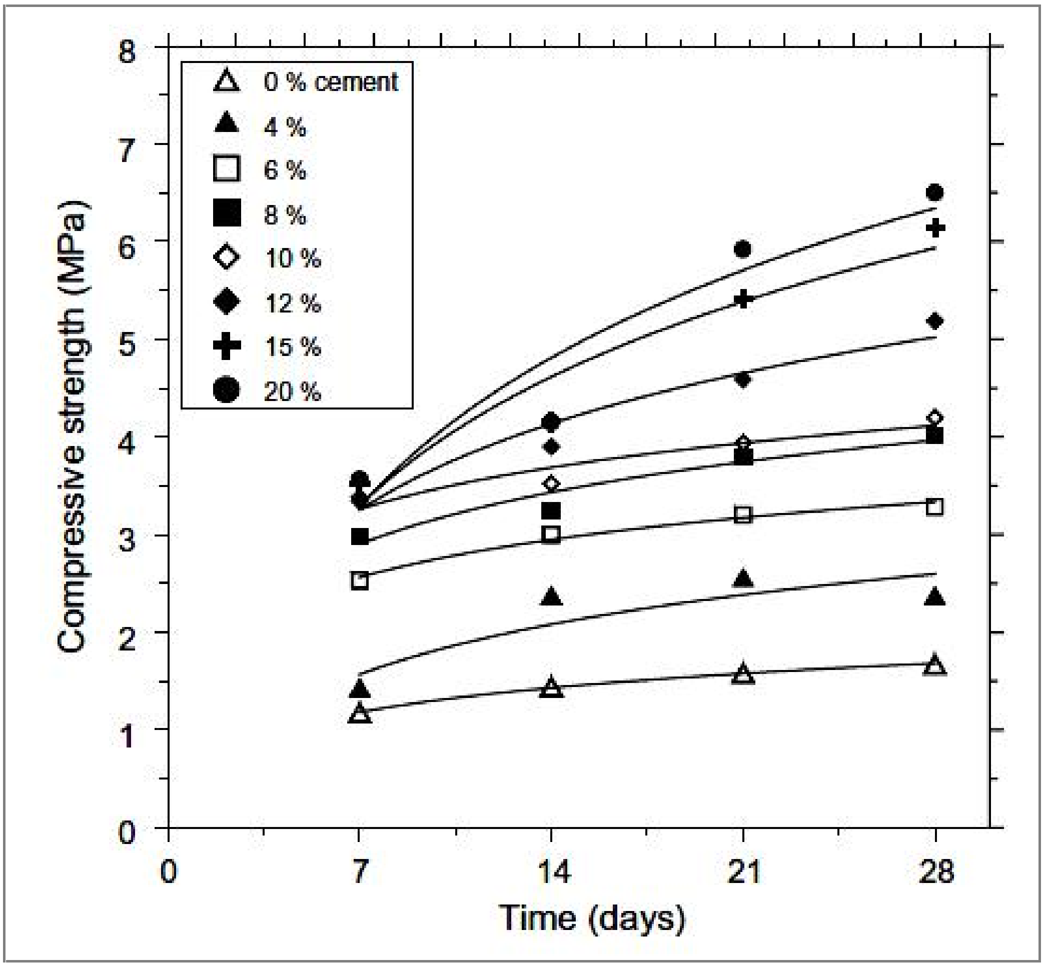
Compressive strength of CEBs as stated earlier could depend on the duration of curing as well. This is evident in the study on the performance of cement-stabilised soil by Bahar *et al.* (2004) who discovered a maximum improvement on compressive strength of the cement-stabilised soil after curing for about 28 days. The study revealed that the relative compressive strength obtained after seven days of curing was about 70% of that obtained after 21 or 28 days of curing for up to 10% of cement content (Figure 2.13). A range of 21 to 28 days of curing was therefore recommended to achieve a maximum strength in cement-stabilised soils.

Compressive strength and specific gravity of laterite blocks have equally been found to decrease with depth, as concluded by Kasthurba *et al.* (2007) in their investigation of laterite stones for building purposes within different quarries in south-west India. They discovered there was an increase in water absorption as the depth increases; this was attributed to the increase in clay content (kaolinite).

It is therefore pertinent to know that in selecting soils for building construction, durability is very important and this should be carefully studied without compromising the minimum standard for compressive strength as well. This can be achieved by properly selecting soil materials, preferably well-graded with an adequate particle size mix.

In as much as clay is very important in soil materials for brick making, especially because of its cohesion properties that serves as binder for the soil, extreme care must be taken to avoid excess clay fractions because of their swelling in contact with water, shrinkage and subsequent cracking due to removal of water.

Compressive strength of CEBs can be improved by stabilisation which comes in different types and quantities. The next section explains more on improving soil by stabilisation.



Source: After Bahar *et al.* (2004)

Figure 2.13: Effect of curing on compressive strength of CEBs

2.4.3 Stabilisation of compressed earth bricks and stabilisers

Building with earth bricks involves making a choice between using the available soil as it is, and adapting the project to the quality of the soil (building standard requirements may be compromised but saving cost) or importing well-suitable soils for the project, which implies extra cost, or modifying the available local soil to suit the requirements of the project. Modifying the available local soil seems the best option and a link between saving cost and meeting required standards. Hence, stabilisation comes in.

Stabilisation has been described as a means of modifying the properties of a soil, water and air system with the aim of obtaining lasting properties (Houben and Guillaud, 1994).

Different materials ranging from natural to synthetic fibres have been used in stabilising soil for building construction. Most often, these materials are used in combination with either cement or lime to improve the properties of construction materials. Some of these fibre materials include: Rice husk ash, peat, marl, coal ash, cassava peels and a host of others.

Many researchers have studied the response and effect of the addition of these materials in improving the strength characteristics and durability of CEBs. These include: Ola (1989); Moore (1987) (in Attoh-Okine, 1995); Osula (1991; 1996); Bells (1996); Basha *et al.* (2005); Venkatarama and Gupta (2008); Billiong *et al.* (2009); Al-Amoudi *et al.* (2010), Villamizar *et al.* (2012); Al-Jabri and Shoukry (2014) and Cong *et al.* (2014). All their findings were positive in improvement of strength properties and durability of compressed earth bricks. This implies that the strength and durability of compressed earth bricks depend largely on the method and type of stabilisation. With stabilisation, one could influence the compressive strength and durability of compressed earth bricks to suit any required purpose, although this could as well depend on cost and time frame.

Soil materials used in building may expand slightly in wet and dry conditions. These expansions may lead to cracking and other defects to the building. Adam and Agib (2001) opined that the use of a stabiliser will reduce this expansion in soil. It was, however, noticed that structures built with compressed stabilised earth bricks suffers more expansion than those built using alternative construction materials. This is reflected in Table 2.5 below. They recommended proper block manufacturing and construction methods in order to reduce such movement.

Durability requirements of Saudi calcareous marl soils were studied by Al-Amoudi *et al.* (2010). They found out that the durability of the soils was satisfactorily improved by marl stabilised with both 5% and 7% cement and lime. It was, however, concluded that cement-stabilisation provided superior results, with material losses less than that for lime-stabilised marl.

TABLE 2.5: COMPARATIVE PROPERTIES OF DIFFERENT CONSTRUCTION MATERIALS USED IN BUILDING CONSTRUCTION

Property	Compressed stabilised earth blocks	Fired clay bricks	Calcium silicate bricks	Dense concrete blocks	Aerated concrete blocks	Lightweight concrete blocks
Wet compressive strength (MN/m^2)	1 - 40	5 - 60	10 - 55	7 - 50	2 - 6	2 - 20
Moisture movement (%)	0.02 - 0.2	0.00 - 0.02	0.01 - 0.035	0.02 - 0.05	0.05 - 0.10	0.04 - 0.08
Density (kg/m^3)	1700 - 2200	1400 - 2400	1600 - 2100	1700 - 2200	400 - 950	600 - 1600
Thermal conductivity W/m°C	0.81 - 1.04	0.70 - 1.30	1.10 - 1.60	1.00 - 1.70	0.10 - 0.20	0.15 - 0.70
Durability against rain	good to very poor	excellent to very poor	good to moderate	good to poor	good to moderate	good to poor

Source: After Adam and Agib (2001).

2.4.3.1 Types of stabilisation

Three methods of stabilisation have been identified in Engineering Geology. These three are equally relevant to building technology, they are:

- **Mechanical stabilisation:** This, according to Lemougna *et al.* (2011), involves compacting the soil to affect its resistance, compressibility, permeability and porosity. This altogether leads to changes in density. The processes of mechanical stabilisation are compaction and consolidation.
- **Physical stabilisation**: This process acts on soil texture. It is achieved by mixing different type of soils, introduction of natural or synthetic fibres from cereal plants, animals and minerals into the soil. Physical stabilisation equally utilises other forms of curing, which include air cure, heat treatment, moisture curing and freezing.
- **Chemical stabilisation:** This involves acting on the physicochemical properties of the soil by the addition of chemicals. This is the most widely used method in compressed earth bricks, as well as building construction generally. The two most commonly used additives in soil stabilisation are cement and lime. These two are explained in detail below.

2.4.3.2 Cement

Ordinary Portland cement hydrates with an addition of water. The process is called hydration process.

Bachar *et al.* (2014) noted that the hardening of any soil is generally affected as the cement hydrates in the presence of water to form complex carbohydrates.

An ordinary Portland cement consists of the following major compounds:

- Tricalcium silicate (C_3S).
- Dicalcium silicate (C_2S).
- Tricalcium aluminate (C_3A).
- Tetracalcium aluminoferrite (C_4AF).
- Gypsum ($C\underline{S}H_2$).

Chemical reactions during hydration: Mamlouk and Zaniewski (1999) described details of cement hydration as follows:

Tri-calcium aluminate reacts with gypsum in the presence of water to produce ettringite and heat:

Tricalcium aluminate + gypsum + water → ettringite + heat
$C_3A + 3C\underline{S}H_2 + 26H \rightarrow C_6AS_3H_{32}, \Delta H = 207 \text{ cal/g}$ *Equation 2.6*

The tricalcium silicate (alite) is hydrated to produce calcium silicate hydrates, lime and heat:

Tricalcium silicate + water → calcium silicate hydrate + lime + heat
$2C_3S + 6H \rightarrow C_3\underline{S}_2H_3 + 3CH, \Delta H = 120 \text{ cal/g}$ *Equation 2.7*

CSH has a short-networked fibre structure, which contributes greatly to the initial strength of the cement glue.

Once gypsum is used up, the ettringite becomes unstable and reacts with any remaining tricalcium aluminate to form monosulfate aluminate hydrate crystals:

Tricalcium aluminate + ettringite + water → monosulfate aluminate hydrate $2C_3A + 3\ C_6A\underline{S}_3H_{32} + 22H \rightarrow 3C_4ASH_{18},$ *Equation 2.8*

Monosulfate crystals are only stable in a sulfate-deficient solution. In the presence of sulfates, the crystals resort back into ettringite, whose crystals are two-and-a-half times the size of the monosulfate. This increase in size is responsible for cracking when cement is subjected to sulfate attack.

The belite (dicalcium silicate) also hydrates to form calcium silicate hydrates and heat:

Dicalcium silicates + water ì *calcium silicate hydrate + lime* *Equation 2.9*
$C_2S + 4H$ ì $C_3\underline{S}_2H_3 + CH, \Delta H = 62\ cal/g$

Here calcium silicate hydrates, like Equation 2.7, contribute to the strength of the cement paste. This reaction generates less heat and proceeds at a slower rate, meaning that the contribution of C_2S to the strength of the cement paste will be slow initially. This compound is, however, responsible for the long-term strength of Portland cement concrete. This is the reason why bricks made with cement should be moisture cured for a certain number of days.

The ferrite undergoes two progressive reactions with the gypsum as follows:

In the first of the reactions, the ettringite reacts with the gypsum and water to form ettringite, lime and alumina hydroxides, namely:

Ferrite + gypsum + water ì *ettringite + ferric aluminium hydroxide + lime* C_4AF *Equation 2.10*
$+ 3C\underline{S}H_2 + 3H$ ì $C_6(A,F)\underline{S}_3H_{32} + (A,F)H_3 + CH$

The ferrite further reacts with the ettringite formed above to produce garnets, i.e.:

Ferrite + ettringite + lime + water ì *garnets* *Equation 2.11*
$C_4AF + C_6(A,F)\underline{S}_3H_{32} + 2CH + 23H \quad 3C_4(A,F)\underline{S}H_{18} + (A,F)H_3$

The garnets only take up space and do not in any way contribute to the strength of the cement paste.

In summary, the cement–water hydration reaction could be simply expressed as:

$\text{Cement} + H_2O \rightarrow CSH + Ca(OH)_2$ *Equation 2.12*

Based on the reactions above, one could conclude that cement stabilisation is well suitable for lateritic soils, because of the presence of iron oxides that react with cement as a result of pozzolanic reactions. However, as much as possible sulphates should be avoided because of the presence of monosulfates as demonstrated in Equation 2.8.

Research by the Nigerian Building and Road Research Institute (NBRRI) has shown that the addition of 4% cement to earth improves the cohesive nature of the soil resulting in good quality bricks (Olotuah, 2002).

Osula (1996), commenting on the results of the aspect of the work of Herzog and Mitchell (1963) on free lime content in clay–cement mixtures for kaolinite clay, confirmed an earlier hypothesis that calcium produced during cement hydration undergoes a pozzolanic reaction with the clay; this is in addition to certain interactions taking place between calcium and the clay minerals.

Findings according to Adam and Agib (2001) show that a relationship exists between linear shrinkage of a soil and the cement content needed for stabilisation. Table 2.6 below shows a cement to soil ratio which ranges between 5.56% and 8.33% for measured shrinkage variations of between 15 mm to 60 mm.

TABLE 2.6: RELATIONSHIP BETWEEN LINEAR SHRINKAGE AND CEMENT CONTENT

Measured Shrinkage (mm)	Cement to Soil Ratio
Under 15	1:18 parts (5.56%)
15-30	1:16 parts (6.25%)
30-45	1:14 parts (7.14%)
45-60	1:12 parts (8.33%)

Source: After Adam and Agib (2001).

Trials on stabilisation of soil using cement date back to 1917; since then, it has gained a wide acceptance and usage in different engineering constructions all over the world. Recently, various workers have studied its use in building construction, especially using earth materials. These include: Walker (1995), Bahar *et al.* (2004), Horpibulsuk *et al.* (2010), Goodary *et al.* (2012) and Cong et al. (2014). All their findings pointed to the fact that cement stabilisation improves the properties of a soil and decreases the rate moisture absorption.

2.4.3.3 Lime

Five basic reactions involved in lime stabilisation as highlighted by Houben and Guillaud (1994) as presented below:

- **Water absorption:** Quicklime undergoes an exothermic hydration reaction in the presence of water or moist soil. Findings have shown that about 300 kcal of energy is released for every kilogram of quicklime.

- **Cation exchange:** Addition of lime to a moistened soil brings about influx of calcium ions to the soil. This leads to cation exchange whereby calcium ion replaces exchangeable cations (sodium, magnesium, potassium) in the soil compounds.
- **Flocculation and aggregation:** As a result of cation exchange and increase in the quantity of electrolytes in the pore water, flocculation and accretion occurs. This occurs especially with the clay particles.
- **Carbonation:** Lime reacts with carbon dioxide from the air to form carbonated cements.
- **Pozzolanic reaction:** The dissolution of clay minerals in an alkaline environment produced by lime, and recombination of silica and alumina in clay together with calcium to form complex aluminium and calcium silicates, is highly responsible for the strength of the material, by cementing the grains together. This is the most important reaction involved in lime stabilisation.

Attoh-Okine (1995) gave the following reactions between lime and soil:

i. $Ca(OH)_2 \rightarrow Ca^{2+} + 2(OH)^-$ *Equation 2.13*

ii. $Ca^{2+} + 2(OH)^- + SiO_2 \rightarrow CSH$ *Equation 2.14*

iii. $Ca^{2+} + 2(OH)^- + Al_2O_3 \rightarrow CAH$ *Equation 2.15*

Where SiO_2 is Silica Clay, Al_2O_3 is Alumina Clay, C is CaO, S is SiO_2 and A is Al_2O_3

A significant use of the RHA has been shown in addition with lime and/or cement to improve the geotechnical properties and soil-bearing capacity of soils. The study gave a recommendation based on an optimum quantity of lime and RHA which brings about a maximum strength; this is obtained at a ratio of 1:1 of lime and RHA. By implication, about 5% lime should be added to 5% RHA to give the highest strength in soil (Muntohar, 2011). The compositional properties of RHA and lime are given in Table 2.7

TABLE 2.7: CHEMICAL COMPOSITION OF ADDITIVES

Chemical composition of the additives.

Constituents	RHA (%)	Lime (%)
SiO_2	89.08	0.00
Al_2O_3	1.75	0.13
Fe_2O_3	0.78	0.08
CaO	1.29	59.03
MgO	0.64	0.25
Na_2O	0.85	0.05
K_2O	1.38	0.03
MnO	0.14	0.04
SO_3	0.01	0.02
P_2O_5	0.61	0.00
H_2O	1.33	0.04
Loss on ignition	2.05	40.33

Source: After Muntohar (2011).

Bell (1996) observed that many of the important engineering properties of clay soils are appreciably improved by the addition of lime. However, the properties of such soil–lime mixtures vary and they are dependent on the character of the clay, the type as well as length of curing and method/quality of construction. Lime was equally reported by Osula (1996) as a more effective form of stabilisation than cement in particle-size enhancement at all elapsed times considered. With respect to immediate improvement in plasticity, lime proves twice as effective as cement at 1% modifier content. According to him, the relatively better effect of cement modification was noticed with time; this suggests that the reactivity of the lime product of cement hydration increases with time. An increase in the percentage of lime in the binder was found by Billong *et al.* (2009) to increase the compressive strength of the final products and slightly decreases water absorption and apparent density.

2.4.4 Economic aspects of stabilisation

Before deciding on the method of stabilisation to adopt, the following should be carefully considered:

- **Properties of the soil involved**: This goes a long way in determining the quantity and quality (type) of stabilisation to be adopted. For cost-efficiency, soils should be carefully chosen with an adequate particle mix. For instance, clayey soil material will require more cement as a stabiliser than a sandy soil. It has equally been recommended that lime is used for any material with >35% clay. Lime, as stated earlier, is more expensive than cement, judging from the situation in Nigeria.

- **Purpose of the project**: In choosing a method of stabilisation, purpose and type of the project is very important. The stabilisation method would dictate the strength of the bricks to be produced. This strength is dependent on the type of project at hand, for instance, for a single-storey building, a compressive strength of about 1–4 MN/m^2 may be good, but for a multiple-storey building, double of that may be necessary. All these depend on the type and method of stabilisation.
- **Time and cost economy**: The time it takes to stabilise and the cost of project is a critical factor in the economy of any project. A decision has to be taken either to minimise cost by spending more time on the production (for curing and stabilisation) or to save time by spending a little more on stabilisation. Findings have shown that it takes different times for different stabilisers to attain a maximum strength (Osula, 1996).
- **Maintenance of the project**: In order to minimise the cost of maintaining a project, the choice of material, choice of stabilisation and durability should be of paramount importance. These factors help in minimising the future expenses on maintenance.

2.4.5 Curing of compressed earth bricks

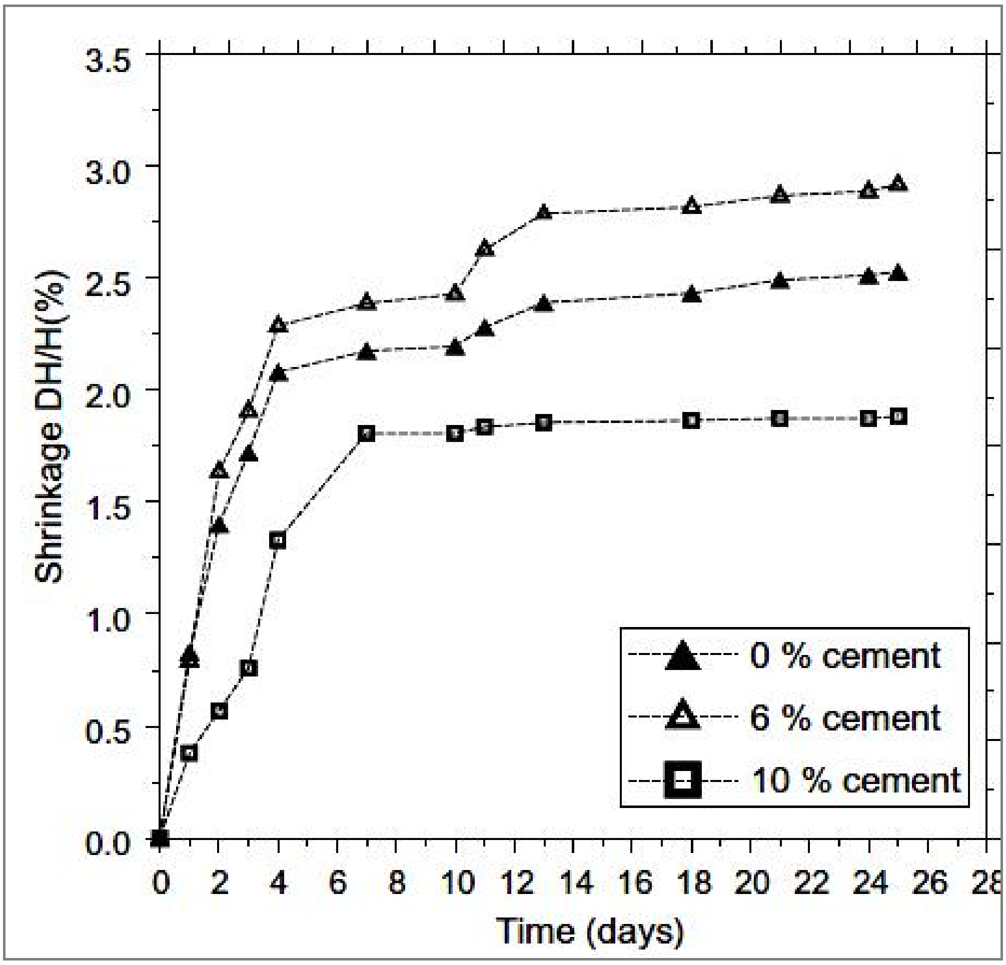
Curing is a form of mechanical stabilisation necessary for bricks, especially when any form of chemical stabilisation is involved. For any cementitious material to achieve a maximum strength, a period of time is required when this material is moisture-cured, meaning that water/moisture is retained in such material for a minimum of three to seven days for cement materials, after which they are exposed to be air-cured for another period of time. Lime stabilisation may require more days of damp curing. Sudden withdrawal of moisture from soil materials could lead to cracking and shrinkage.

Different methods according to Adam and Agib (2001) are used in ensuring a proper curing procedure; these include the use of plastic bags (polythene bags), grass and leaves (see Figure 2.14). They posited that various soils require different durations of curing, which varies alongside with type of stabiliser used. In cement stabilisation, for instance a minimum of three weeks curing have been recommended, while lime stabilisation requires a minimum of four weeks to attain a maximum strength.

Source: After Adam and Agib (2011).

Figure 2.14: A curing process of compressed earth bricks

Bahar *et al.* (2004) studied the performance of compacted cement-stabilised soil. They noticed the following effect: shrinkage was found to increase rapidly during the first four days after moulding for both cement-stabilised soil and unstabilised soil specimen. The shrinkage decreases as the day proceeds. Hence, they recommended curing for the first four days after moulding, as this would reduce drying, shrinkage and cracking. They reported shrinkage of cement-stabilised soil at 25 days as compared to that of unstabilised soil which was reduced by about 20% and 44%. Figure 2.15 below shows the variation of shrinkage with time for cement-stabilised soil. It was equally noticed that the addition of sand reduces the shrinkage as sand particles oppose the shrinkage movement. The reduction in shrinkage was given as 29% and 64% for 10% and 15% of sand content, respectively.



Source: After Bahar *et al.* (2004).

Figure 2.15: The effect of cement content shrinkage

It was concluded in Deboucher and Hashim (2011) on the water absorption capacity of lime and cement, that increasing curing periods improved their compressive strength and decreased their water absorption capacity.

2.4.6 Advantages of Compressed Earth Bricks

Published literature which discussed the advantages of CEBs, are Houben and Guillaud (1994), Riggasi (1995), Adam and Agib (2001), Kasthurba *et al.* (2007, 2008), Meriani (2008), Lemougna *et al.* (2011), Al-Jabri and Shoukry (2014) and many more. All emphasised the numerous benefits of compressed earth bricks to the end user and to the environment at large. They described CEBs as material suitable for sustainable building construction which brings about a sustainable environment. Building in a 'sustainable' way offers an opportunity to use resources efficiently, while creating buildings that improve human health and well-being and preserving a better environment, keep economic costs affordable.

A closer look at the highlight of the benefits is given below.

2.4.6.1 Cost-efficiency

Madeador (1994) (cited by Olotuah, 2002, reported the findings of an experiment by the NBRRI in collaboration with the Federal Housing Authority (FHA), that there was at least a 40% cost-saving in the use of local building materials compared to conventional ones of identical construction. He noted that the use of cement in Nigeria is more economical than the use of lime. This, according to him, was because lime is more expensive than cement.

Affordable housing can be achieved by the use of compressed earth bricks. This is in line with the definition of affordable housing according to UN Habitat Report (2011). Affordable housing was defined as that which is adequate in quality, location and does not cost so much that it prevents its occupants or owners from meeting other basic living costs or threatens their enjoyment of basic human rights. Table 2.8 shows the findings of the UN habitat on the cost-efficiency of compressed earth bricks. The work was carried out in Sudan; hence, the cost implication was given in Sudan currency (SGD).

TABLE 2.8: A COMPARATIVE COST ANALYSIS BETWEEN BURNT BRICKS AND COMPRESSED EARTH BRICKS

Cost component	Burned Brick	Stabilized Soil Blocks	Cost Difference in SGD
Materials	22,155	17,020	5,135
Labour	12,267	11,162	1,105
Total Cost of Construction	**34,422**	**28,182**	**6,240**

Estimated materials and labour cost inputs in SGD.

Source: UN-Habitat (2001).

Compressed earth bricks are cost-efficient in so many regards. First, the raw materials are readily available in large quantities in most regions, particularly tropical regions. Winnowing distance is greatly reduced or eliminated totally with the availability of these materials. Compressed earth bricks measure up to present-day building material requirements; it presents a technological alternative to imported materials. A wide range of cheap presses and production units presently available in the market has made it more accessible to different classes of people. These presses range from manually operated presses to mechanical presses which are easily moved from one place to another, making it very flexible to use with brick production ranging from small-scale to medium- and large-scale or industrial.

In addition, compressed earth bricks come in natural bright colours (purplish red to red) which are resistant to weathering, making CEBs aesthetically pleasing and as well as saving cost of painting and mortar rendering.

Construction with compressive earth bricks eliminates the cost of heating and cooling the building. CEB possesses low thermal conductivity and a high thermal capacity. This makes it possible to conduct heat slowly in the afternoon and release it slowly as well at night.

2.4.6.2 Material efficiency

Compressed earth bricks are material efficient. In the production of CEBs, it has been found that water is conserved by about 30% more than what is used in other conventional building materials (Rigassi, 1995). CEBs made of earth material (lateritic soil) are sustainable construction materials. These earth materials are recyclable, that is, they can be reused, they possess very low harmful off-gassing air emissions, zero or low toxicity, durable and readily available. Generally, these materials are green building materials; *i.e.* they are environmentally friendly. CEBs are well suitable as construction material for most parts of the building; from the foundation, walls, arches up to the roof of any building.

2.4.6.3 Energy-efficiency

Meriani (2008) opined that energy-efficient buildings should be properly sized with energy-efficient heating/cooling systems in conjunction with a thermally efficient building interior. In an energy-efficient building, electric loads and usage from lighting, equipment and appliances should be minimised. CEBs have proven to be excellent housing material in cold climatic regions due to its high thermal capacity and low thermal conductivity, which makes it possible to moderate extreme outdoor temperatures and a satisfactory indoor temperature, leading to temperature balance; warm especially at night and cool during the day.

Bachar *et al.* (2014), on thermal conductivity of stabilised earth concrete, examined at theories capable of predicting the effective thermal conductivity of a dry blend considered like a two-phase system. Their finding shows that the linear model and some theoretical models gave the best concordance with experimental results. These theoretical models estimate the best values of the thermal conductivity of these materials at porosity of less than 80%. By implication, CEBs which are densely packed and compacted, has a higher thermal conductivity than other conventional materials in building construction.

CEBs require low energy input in terms of manpower; it requires only about 1% of the energy required to manufacture and process the same volume of cement concrete. Report has it that the energy needed to manufacture and process one cubic meter of earth (soil) is about 36 MJ (10 kWh), while that required for the manufacture of the same volume of concrete is about 3 000 MJ (833 kWh) (UN Habitat [UNCHS], Technical Note No. 12 on comparison of adobe and fired clay bricks, cited in Adam and Agib, 2001).

Compressed earth bricks are fire resistant; firing makes earth material stronger in terms of compressive strength. They are soundproofed, hence, privacy is guaranteed.

2.4.6.4 Environmental friendliness

As stated earlier in the introduction, the world is tending towards a global green environment and all construction material is required to promote a sustainable environment. There have been different calls by world leaders, organisations and bodies on the promotion of a sustainable environment, an environment where there will be a lesser use of fossil fuels, zero emission of toxic gasses (CO_2, NO, CFCs, etc.), an environment that will promote sustainable materials that are renewable and can be recycled, an environment conducive for all.

Recently, the Catholic Pontiff, Pope Francis, joined his voice with those of other world leaders in catering for the environment. He noted in his Encyclical as follows:

> *"Investments have also been made in means of production and transportation which consume less energy and require fewer raw materials, as well as in methods of construction and renovating buildings which improve their energy efficiency. But these good practices are still far from widespread"* (e-spiritusanto 'Laudato Si' 2015).

The use of earth material, which is almost of unlimited resource in its natural state, involves no pollution and negligible energy consumption, thus further benefiting the environment by saving bio-mass fuel. Numerous ecological and environmental benefits are accruable from the use of earth materials in building industry. Lemougna *et al*. (2011) highlighted some of the salient benefits of building with earth materials. The amount of cement is reduced drastically, which in turn reduces the amount of CO_2 emitted and the total energy used for construction. Rahdi (2009), on the impact of global warming in the residential building of United Arab Emirate, noted that the largest contribution to carbon emissions emanates from power generation, transport, industries and building operations. In producing one ton of cement, 0.55 tons of CO_2 is generated and needs a combustion of carbon-fuel into 0.40 tons of CO_2 (Davidovit, 1991 cited in Lemougna *et al*., 2011). This reflects a huge contribution of the production and consumption of cement to global warming.

It is, therefore, of utmost importance to reduce the consumption of excessive cement through promotion and use of environmentally friendly earth materials that utilises minimal quantity of cement. Research is on-going on how to further reduce or eliminate the use of cement totally.

2.4.6.5 Recycling

Compressed earth bricks can be recycled easily at the end of its usable life. CEBs can easily crushed and put into another use because of the little content of cement present.

2.5 CONCLUSION

In conclusion, previous literature mostly presented in this chapter have established the critical role soil materials could play in achieving a sustainable housing, with the focus on lateritic soils and compressed earth bricks. However, while the importance and unique properties of lateritic soils have been elaborated, more insight is required into its suitability as material in CEBs, especially those mineralogical properties that could render it either useful or otherwise in CEB making.

Chapter 3

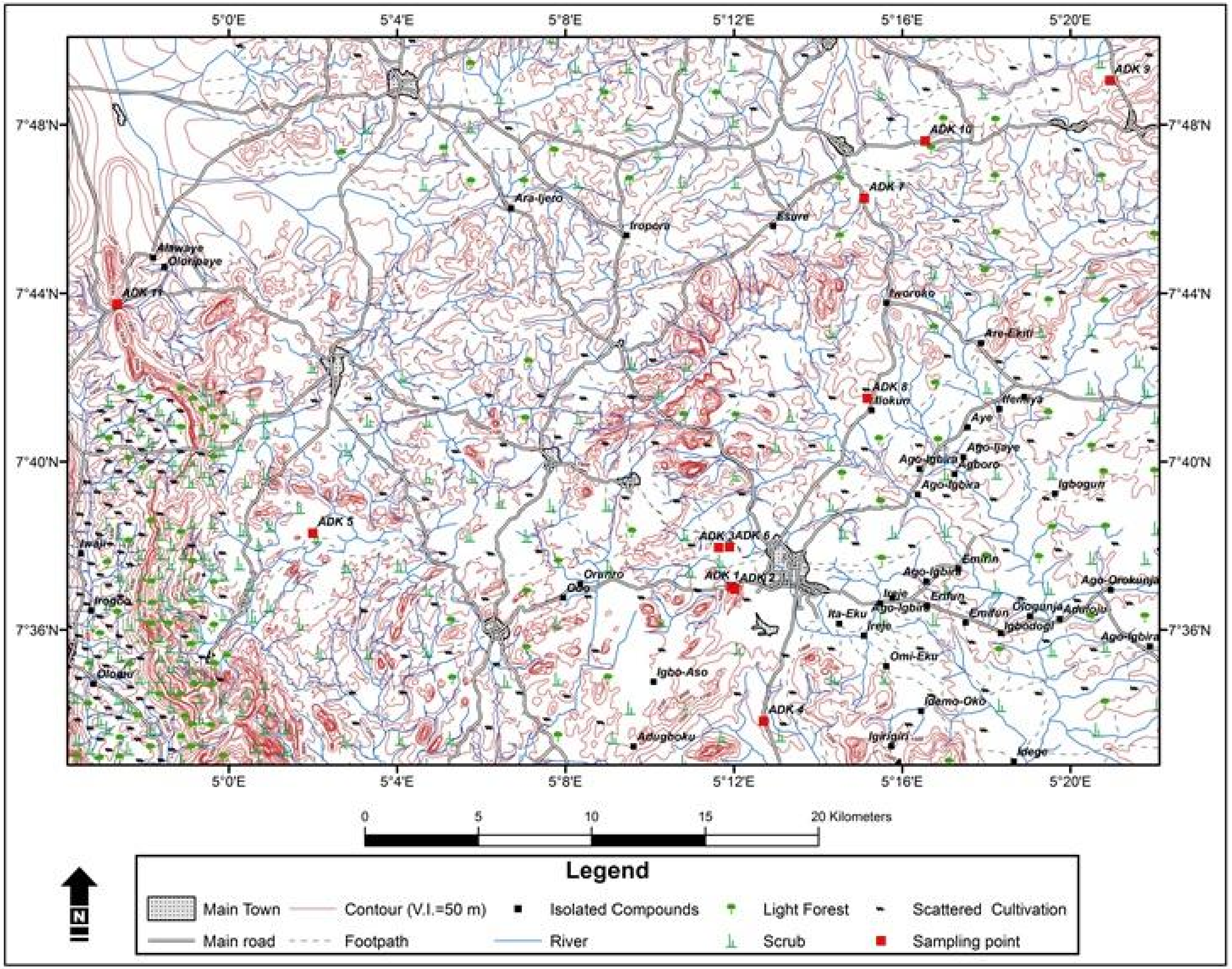
PHYSICAL AND GEOLOGICAL SETTINGS

3.1 INTRODUCTION

Based on different opinions about the origin, formation and processes of laterisation as stated in the previous chapter, it is pertinent to study and understand the physical and geological settings of the study areas with a view to understanding the type and quality of lateritic soils found in the study areas. It is equally important to understand the geomorphology of the study area, that is, the relationship between the geology and geography of the area. In literature it is established that topography and landform contribute largely to the formation and characteristics of soils, especially lateritic soils (Adeyemi, 2003, Bourman and Ollier, 2003, Mcfarlane, 1990 and Ola 1978). The physical features discussed here were identified as factors of laterisation by Fookes *et al.* (2000) (cited in Mitchell and Soga, 2005). These physical features determine to a large extent the soil type and ultimately the engineering characteristics of the soil.

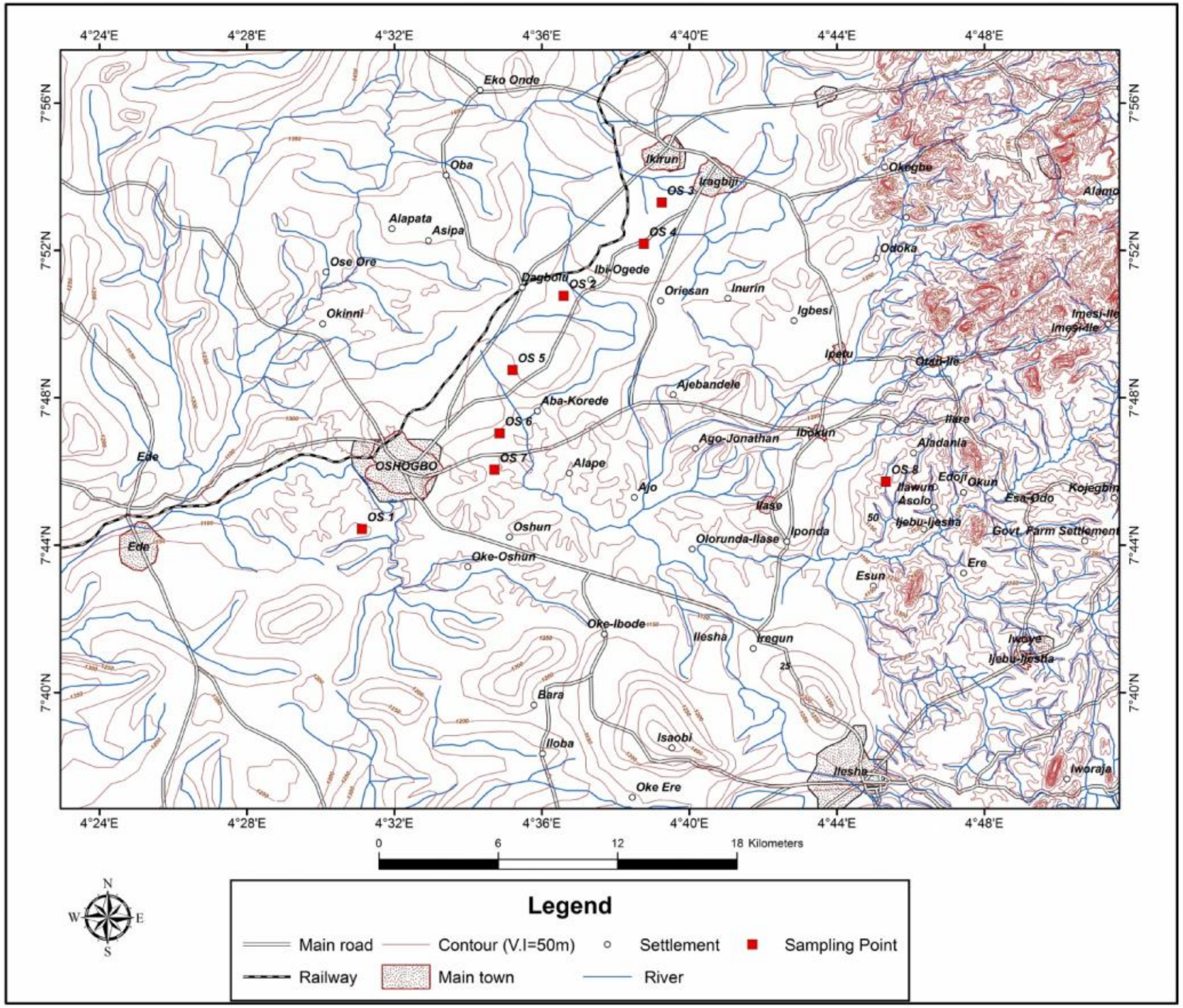
3.2 LOCATION AND ACCESSIBILITY

Osogbo and Ado-Ekiti are towns in the Osun and Ekiti States, respectively, in Nigeria. The study area covers part of the two states, limited mainly to the capital cities and their suburbs in both states. These are Ado-Ekiti and its environs in the Ekiti State, labelled as ADK, and Osogbo and its environs in the Osun State, labelled as OS. The study areas based on UTM coordinate system of Minna datum, fall within N7°40'–7°56', E4°24–4°48' and N7°36'–7°48', E5°0'–5°20' latitude and longitude for Osogbo and Ado-Ekiti, respectively. All the sampling locations and towns are accessible by roads and in most cases with the sampling points not farther than 50 m from the main roads. This is common in both areas because it is general practice to convert old road construction borrow pits into laterite mines, especially for local brick making. Figures 3.1 and 3.2 show details of the surrounding towns and other topographic features in the two study areas.



Source: Modified after NGSA (2006).

Figure 3.1: Topographical map of Ado-Ekiti with sampling location

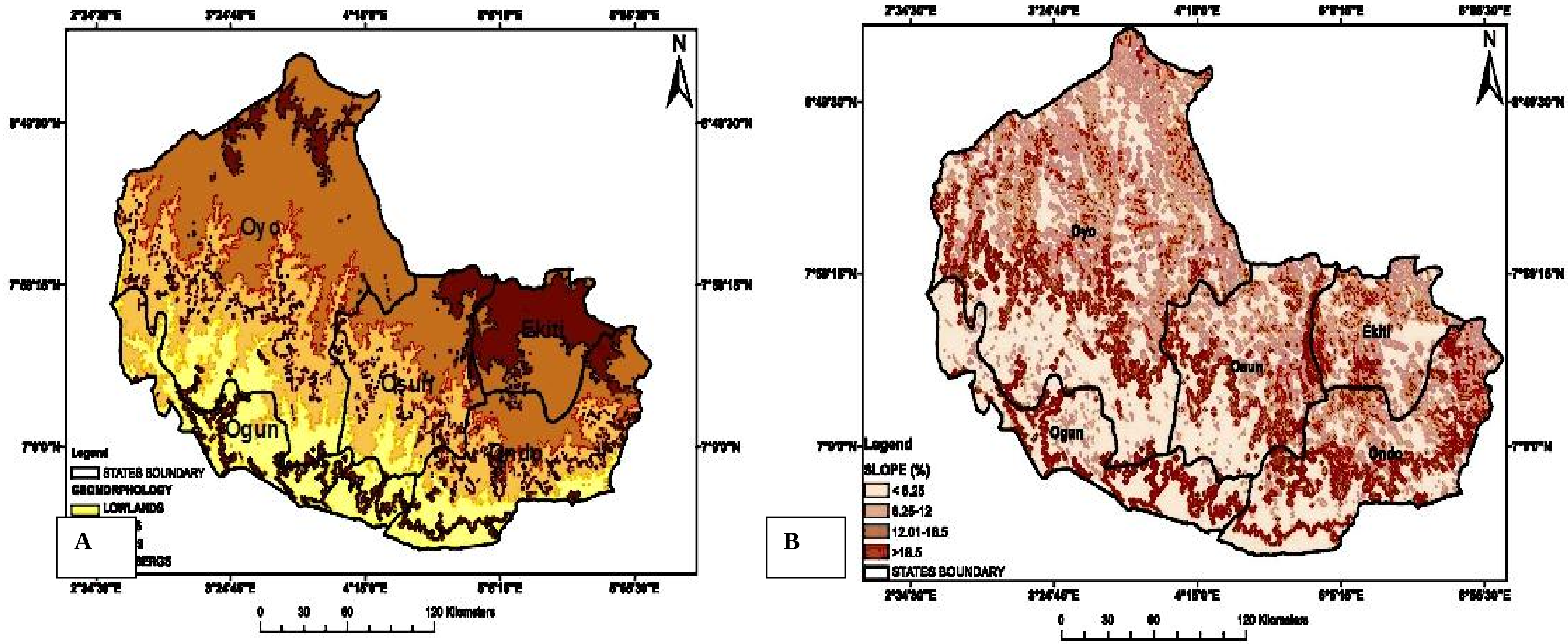


Source: Modified after NGSA (2008).

Figure 3.2: Topographical map of Osogbo with sampling points

3.3 RELIEF AND TOPOGRAPHY

The topography of the general area can be described as undulating with several high and low elevations. Both locations are characterised by hills and valleys with the elevated platforms formed by the high-rise outcrops. The altitude of sampling points within ADK ranges between 406 and 582 m, while those at OS are between 342 and 395 m. Ekiti State (ADK) has the highest elevation points due to the whole state falling on a very high altitude above sea level. The name Ekiti translates to hill, meaning a mountainous region. The study areas in general are characterised by many ridges and inselbergs which are mostly of gneiss bedrock alongside valleys and ridges of quartzite that forms a very large envelope that covers almost the whole study area. These ridges form part of the Effon Psammite formation. The areas are further characterised by high and low landforms that range from steep-sided hills to gently sloping features. Figures 3.1 and 3.2 show the topography (geomorphology and slope) of the study areas. The study areas, according to the geomorphological map (Figure 3.3) are characterised based on slope percentage as suggested by Fashae *et al.* (2014). The slopes in both the Osun and Ekiti States falls within a range of 6.25% to >18.5%. It is noticed that the boundary between the Osun and Ekiti State from Figure 3.3 that is south-west up to the north-western part of Ekiti, which corresponds to the north-eastern part of the Osun State. These are the areas with dense ridges, inselbergs and slopes, affecting the drainage pattern, hydrogeology and geology of the particular areas. According to Talabi (2013), Ekiti State is blessed with well-defined undulating topography with a series of prominent hills usually characterised by steep slopes. These hilly features include inselbergs, whalebacks and other categories of residual hills commonly associated with massive granite bodies. The inselbergs are characteristic features of the Pan-African granites.



Source: After Fashae *et al.* (2014)

***Figure 3.3:** A) Geomorphological map of south-western part of Nigeria; B) Slope map of south-western part of Nigeria*

3.4 DRAINAGE

A salient feature of the basement in the study area is the structurally controlled drainage systems evident in the drainage patterns of rivers and streams. The dendritic drainage pattern in the study areas is mainly controlled by fractures. Part of the study area, belong to the Ilesha schist belt and corresponds to the south-western/north-western part of Ekiti and north-eastern part of Osun State. Anifowose and Borode (2007), reported that one of the post-folding fractures in the area serves as the source of the Ikogosi warm spring in the Ekiti State. Prominent as well are the minor rivers shown in the topographical maps of the study areas (Figures 3.1 and 3.2).

3.5 CLIMATE

The study area falls within the tropical climatic zone with well-defined rainy (wet) and dry seasons. These two distinct variations in seasons occur between March/April to September/October for the rainy season, and November/December to February for the Harmattan/dry season, respectively. These two seasons are caused by the moist maritime south-western monsoon winds that blow inland from the Atlantic Ocean for the rainy season and dry dust-laden winds that blow from the Sahara Desert for the dry season (Talabi, 2013). During the dry season, usually marked with bright sunshine, the north-east trade winds, known as the Harmattan, keep the humidity extremely low, the grasses are dry, most of the rivers and streams in the areas are dry. Due to the current global climatic changes, the duration of the wet and dry season could vary, with the wet season sometimes stretching up to November/December, while the dry season extends until May, and at times lasts for a very short period from December to February. The total annual rainfall ranges between 1 500 and 2 500 mm, with peaks in June and October. The mean annual temperature is about 28 °C with 3 °C annual variability and relative humidity of 90% average throughout the year (Gbadegesin and Nwagwu, 1990). According to Fookes climatic classification (in Mitchell and Soga, 2005), the study area falls within the morphoclimate zone category.

3.6 VEGETATION OF THE STUDY AREA

Vegetation density reflects the magnitude and pattern of rainfall, soil type and variation in altitude (Mitchell and Soga, 2005). The study areas are covered by typical vegetation occurring in the south-western part of Nigeria and is characterised by evergreen trees and dense vegetation typical of tropical rainforest and moist woodland. The primary vegetation encourages a great deal of farming activities, and due to the densely populated trees around this area, lumbering activities are very common apart from agriculture. Moreover, human activities have greatly affected the vegetation of the area, especially deforestation, cultivation of land, urbanisation and activities associated with lumbering.

3.7 OVERVIEW OF THE GEOLOGY OF SOUTH-WESTERN NIGERIA

3.7.1 Basement complex

The basement complex of Nigeria has been reported as part of the Neoproterozoic (750–500 Ma) Trans-Saharan mobile belt. It forms the southern part of this mobile belt, which is bound to the west by the West African craton, south-east by the Congo craton and to the north-east by the Eastern Saharan block (Caby, 1989; Ferré *et al*., 2002). Rocks of the basement complex have been grouped into the migmatite–gneiss complex, the schist belts and the Pan-African granites (Elueze, 2000).

Detailed lithology of the basement complex, especially the south-western part of Nigeria, comprises of migmatitic and granitic gneisses; slightly migmatised to unmigmatised metasedimentary schists and meta-igneous rocks; quartzites, charnockitic, gabbroic and dioritic (mainly) granites, granodiorites and syenite. The supracrustal sequences in this basement complex was studied and found to have yielded relicts of Archaean, Palaeoproterozoic, as well as Neoproterozoic ages (Oversby, 1975; Rahaman, 1976; Rahaman *et al*., 1983; Dada, 1998; Ekwueme and Kröner, 1998; Ferré *et al*., 1998; Okonkwo and Ganev, 2015).

3.7.1.1 Gneiss–migmatite complex

The gneiss-migmatite complex has been studied as being part of the basement complex by Okonkwo and Ganev (2015). It underlies the major parts of the south-western basement complex, comprising gneisses, quartzites, calc-silicate, biotite-hornblende schists and amphibolites. Based on various earlier studies by different authors (Oversby, 1975; Rahaman, 1976; Okonkwo and Folorunso, 2012), the migmatite-gneiss complex is composed of three main components:

- **Early gneiss:** These are grey foliated biotite and/or hornblende quartzo-feldspathic rocks of granodioritic to quartz dioritic composition.
- **Mafic:** They are mainly amphibolites, amphibole schists, biotite and biotite hornblende schists or rarely meta-gabbros. Usually foliated and as far as can be observed, the foliation is often parallel to that in the enclosing rocks.
- **Granitic or Felsic component:** This is usually of granitic composition and varies in texture from aplitic to granitic to pegmatitic. It occurs as (i) concordant to discordant veins and dykes of pegmatite; (ii) swarms of augen-shaped porphyroblast of white or pink microcline aligned along the pre-existing foliation in the host rock; and (iii) indefinite impregnations commonly resulting in destruction of the structure of the host rock.

Different grades of gneisses, ranging from othogneiss, paragneiss and metabasic rocks within the migmatite-gneiss complex, have reached the amphibolite facies of metamorphism, while few others have attained granulite facies metamorphism (Rahaman, 1976; Dada, 1998; Adepelumi *et al.*, 2008; Oyinloye, 1998; Okonkwo and Ganev, 2015).

3.7.1.2 Quartzite

Quartzites usually form positive/prominent topographic features which rise up to about 100 m above the surrounding terrain. Generally speaking, they outcrop poorly, but a couple of good outcrops occur around the Osogbo and Ekiti States. In terms of mineralogy, quartz is the dominant mineral present, usually more than 90% with minor amounts of muscovite, sillimanite, staurolite, garnet, haematite, graphite, tourmaline and zircon. Clinopyroxene, tremolite, actinolite, epidolite, calcite and sphene have been described in quartzite from Iseyin (Rahaman, 1976). Okonkwo and Ganev (2015) reported north–south trending thickly-bedded quartzite around the Bode-Baadu area of the Ogun State with the presence of sillimanite fibres which indicate metamorphism of upper-amphibolite facies.

3.7.1.3 Older granites

The 'older granite' was first identified and distinguished from the younger or the plateau tin-bearing alkaline granite of Jos, by Falconer (1911). Falconer used the term to differentiate these porphyritic granites of the Pan-African age (600 ± 150 Ma) from the 'younger granites' of Jurassic age (Ekwueme and Kröner, 1998). Ajibade (in Ekwueme and Kröner, 1998) reported that the south-western basement complex is composed of about 20% older granite, and it increases eastward in volume up to about 70% found in the eastern part of the country. Circular to elliptical bodies occur in schist environments, while more elongated bodies are common in migmatite-gneiss terrains. The older granites include rocks of a wide range of compositions, which include granites, granodiorites, adamellites, quartzites, monzontites, syenites and pegmatites. Granitic-granodioritic compostions are the most common types (Rahaman, 1976).

3.7.1.4 Granitic-granodioritic rock

Three main groups of granites were recognised by Jones and Hockey (1964). These are: the early phase, comprising granodiorites and quartz diorites; the main phase, comprising coarse porphyritic hornblende-granite, syenite together with coarse porphyritic biotite granite and the late phase, comprising homogenous granites with dykes and pegmatites with aplites.

Rocks of the early and late phases, although widely distributed, are usually of small areal extent with a few exceptions. The contact relationship of the older granite with the surrounding country rock may be abrupt, but most are gradational over very short distances.

3.7.1.5 The schist belts

In the Basement, the north–south trending, elongated linear zones of low to medium grained metamorphosed super-crustal rocks are known as 'schist belts' and mostly occur within the migmatites and gneiss. Common lithologies within the schist belts are mica-schist, meta-greywackes, quartzites, marble and meta-volcanic rocks that have been metamorphosed from greenschist to amphibolite facies grades (Okonkwo and Ganev, 2015). Schists are derived from ancient (probably Paleoproterozoic, 2000 Ma old) sedimentary and igneous series, which were deposited and intruded on the pre-existing gneiss-migmatite quartzite basement, hence they are described as supracrustal or cover rocks. These rocks were subsequently metamorphosed and deformed together with the basement during several episodes. Caby and Boessé (2001) observed the existence of two generations of metasediments in terms of their age, the older Palaeoproterozoic sequence comprising quartzites, as well as aluminous metapelites and a younger sequence comprising polymictic metaconglomerates, metagreywackes and schists.

The bulk of the schist belt is located within the western half of the country, namely: Katsina, Kaduna, Niger, Zamfara, Kebbi, Kwara, Kogi, Edo, Ekiti, Ondo, Osun, Oyo and Ogun States.

Schist belts in the south-western part include Iseyin, Igarra, Egbe-isanlu, Ife-Ilesha, Lokoja-Jakura and Bode Saadu (Okonkwo and Ganev, 2015; Ige *et al*., 1998; Caby and Boessé, 2001; Rahaman, 1976).

3.7.2 Age and evolution of south-western basement rocks

Geochronological data available in the south-western part of the Nigerian basement complex reveal Late Archean to Palaeoproterozoic ages (Okonkwo and Ganev, 2015; Caby and Boessé, 2001; Dada, 1998; Ferré *et al*., 1998; Ige *et al*., 1998; Oversby, 1975 and Grant, 1970).

In most recent findings, Okonkwo and Ganev (2015) confirmed that Paleoproterozoic was an important period in the crustal growth of south-western Nigeria. Metasedimentary rocks were linked to the Paleoproterozoic age, while studies on the granitoids, mainly granites, pegmatites and granodiorites revealed a Neoproterozoic age emplaced during the Pan-African orogeny (600 ± 150 Ma). The study equally concluded that Eburnean orogeny affected some sectors of the Nigerian basement complex.

Earlier literature (Caby and Boessé, 2001; Dada, 1998; Ekwueme and Kröner, 2006, 1998; Ferré *et al*., 1998; Grant, 1970; Ige *et al*., 1998; Kalsbeek *et al*., 2013; Okonkwo and Ganev, 2015; Oversby, 1975) on different sections of the Nigerian basement complex, proposed different evolution models of the basement rocks based on their studies. However, most authors agree on the following:

- Rocks of the basement complex comprise largely of a metasedimentary series with associated minor meta-igneous rocks which have been variably altered to migmatitic gneisses, and the rocks of the older granites suite of both an intrusive and replacement origin.
- That migmatite–gneiss complex is the most common rock type in the basement and it happens to be the oldest rock type of about 2.0–3.0 Ga (Rahaman et al., 1988; Dada et al., 1998) and a product of several tectonothermal events which brought rocks of various origins together.
- The un-metamorphosed cross-cutting dolorite and syenite dykes are the youngest rock in the basement complex.

However, both Dada (1998) and Ferré *et al*. (1998) gave a detailed account of the ages of different lithologies found in the basement complex of Nigeria.

In conclusion, the south-western part of Nigeria, judging from the earlier literature, shares a common age with the rest of the Precambrian rocks in Nigeria which lies within the zone of Pan-African reactivation, 600 ± 150 Ma.

3.8 LOCAL GEOLOGY

The study areas occur in the south-western part of Nigeria, therefore the geology is of the basement complex. Both study areas, Ado in the Ekiti State and Osogbo in the Osun State, are underlain by the crystalline basement rocks of Ado-Ekiti and the Ilesha/Iwo schist belt of the Osun State, respectively.

3.8.1 Ado Ekiti and environs

Ado-Ekiti and environs is underlain mostly of Precambrian crystalline rocks of igneous and metamorphic origin. Prominent lithologies around the Ado-Ekiti area include charnockites, porphyritic granite, fine to medium grained granite, migmatite, banded gneiss, quartzite and schist. Migmatite and gneiss exists in different forms and types in and around Ado-Ekiti, namely migmatite-gneiss, granite gneiss and banded gneiss. The age of migmatite gneiss complex in the basement terrain of the south-west was found to be about 2.0–3.0 Ga (Dada, 1998; Rahaman, 1988; Talabi, 2013). Distinct mineralogical banding was observed in the banded gneiss with the mafic portion consisting of biotite, hornblende and some other opaque minerals, while the felsic portion consists of quartz and feldspars. Quartzite and quartz schist vary in colour from off-white to grey, depending mostly on their iron oxide content. They mostly form ridges, especially around Ado town rising up to about 100 m above the surrounding terrain. This could be linked to their resistance to chemical weathering, hence their massive, non-foliated texture. Granites within this area occur as later intrusions within the migmatite–gneiss–quartzite complex, that is Pan-African (600 ± 150 Ma) in age (Okonkwo and Folorunso, 2012; Oyinloye, 1998; Talabi, 2013). Granites within the area mostly outcrop as dome-shaped hills with a fine-medium grain texture with no preferred orientation of minerals. Coarse-grained textured granites equally exist as porphyritic granites in some other places within the study area. It was, however, noticed that weathering in porphyritic granite seems more than in fine-medium grained granite. Charnockites occur within the study area as intrusive rocks emplaced during the Pan-African orogeny. They are massive greenish to dark-grey with a medium-coarse grain texture. Charnockites around Ado-Ekiti exists along the margin of the older granite bodies (Afolagboye *et al*., 2015; Rahaman, 1976; Talabi, 2013).

In terms of mineralogy, a recent finding on rocks around Ado-Ekiti and environs, by Afolagboye *et al.* (2015) revealed that quartzite within this area consists of quartz as its main mineral constituent. Other minerals present are presented in 3.1 below:

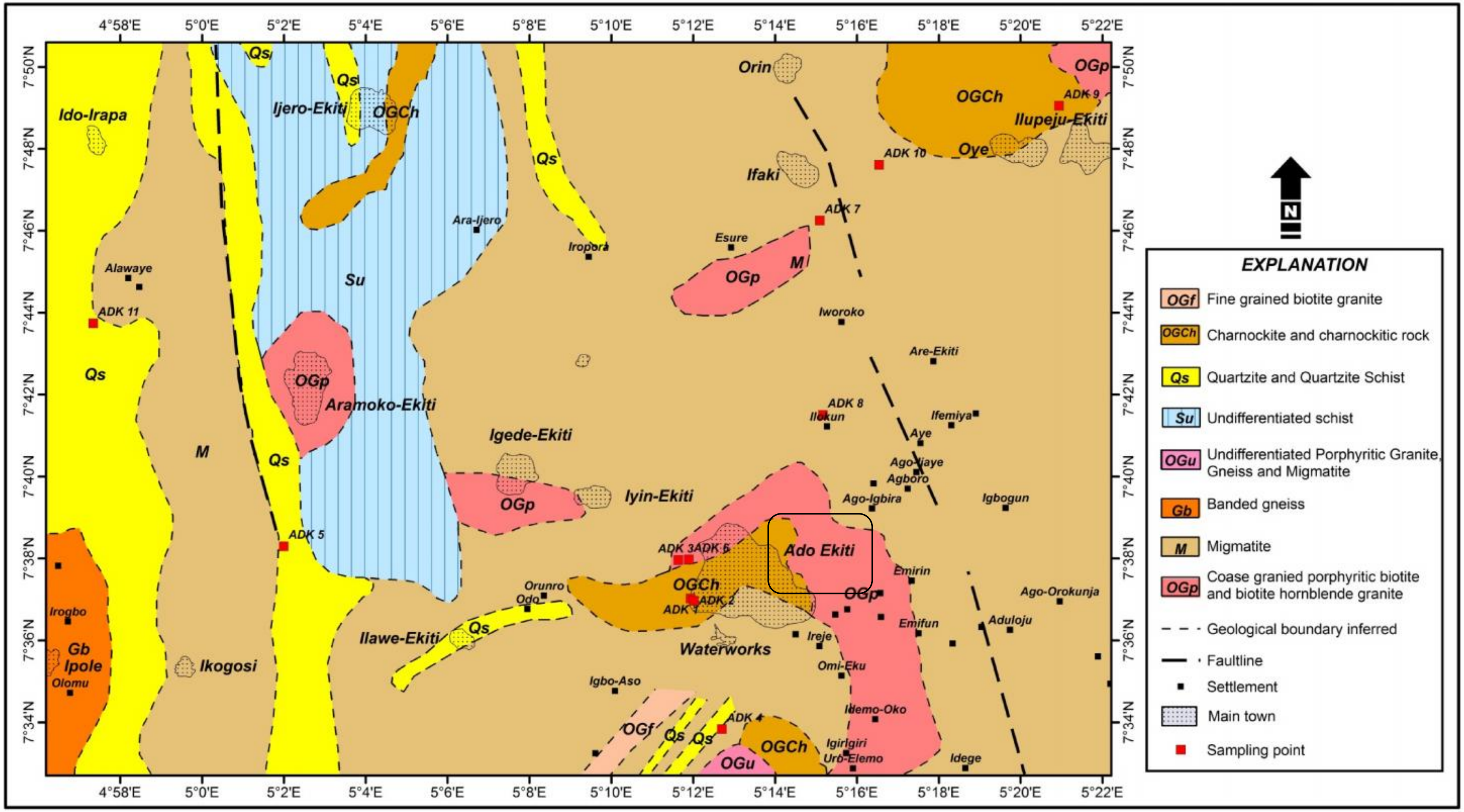
TABLE 3.1: MODAL COMPOSITION OF ROCKS AROUND ADO-EKITI

Minerals	Q	M	GG	MGG	FGG	PG1	PG2	PG	CGC	MGC	FGC
Quartz	98.8	38.7	22.3	36.2	30.2	18.2	33.3	24.4	19.3	21.3	14.5
Microcline	1.2	13.6	14.5	14.2	17.6	62.5	22.2	36.1	31.9	16.7	42.6
Plagioclase		24.6	12.3	24.6	25.3	6.5	18.9	18.3	18.6	36.8	18.5
Hornblende		6.4	9.6	6.4	8.2	2.1	9.7	5.9	10.1	5.0	6.8
Biotite		12.5	16.6	16.4	17.8	10.1	11.3	10.7	18.9	16.5	15.3
Muscovite		2.4	23.5	1.0			3.5	3.5		2.5	
Opaque		1.8	1.2	1.2	0.9	0.6	1.1	1.1	1.2	1.2	2.3
Total	100	100	100	100	100	100	100	100	100	100	100

Q quartzite, *M* migmatite gneiss, *GG* granite gneiss, *MGG* medium grained granite, *FGG* fine grained granite, *PG* porphyritic granite, *CGC* coarse grained charnockite, *MGC* medium grained charnockite, *FGC* fine grained charnockite

Source: Afolagboye (2015).

Rocks mineralogy within this area provides a clue as to what the strength and texture of resultant soil will be. For instance, it is expected that quartzite will produce soils that are medium- to coarse-grained in texture with good strength properties, while rocks with a high amount of mica and feldspar are expected to breakdown into fine-grained soils with a sizable amount of clay. Figure 3.4 gives details of the geology corresponding to each sampling location.



Source: Modified after NGSA (2006).

Figure 3.4: Geological map of Ado-Ekiti and its environs with sampling points

3.8.2 Osogbo and environs

The Ilesha succession comprises metasedimentary rocks with closely associated meta-basic/meta-ultramafic assemblages. On the basis of field relationships, petrological and petrochemical characters, the rock may be subdivided into three principal groups, namely the Ilesha amphibolite complex, the Ilesha metaclastics and the Effon quartzitic sequence.

The metasedimentary complex comprises the metaclastics and the Effon rocks which generally exhibit different structural metamorphic and petrochemical features (Elueze, 1982).

3.8.2.1 The Effon quartzitic sequence

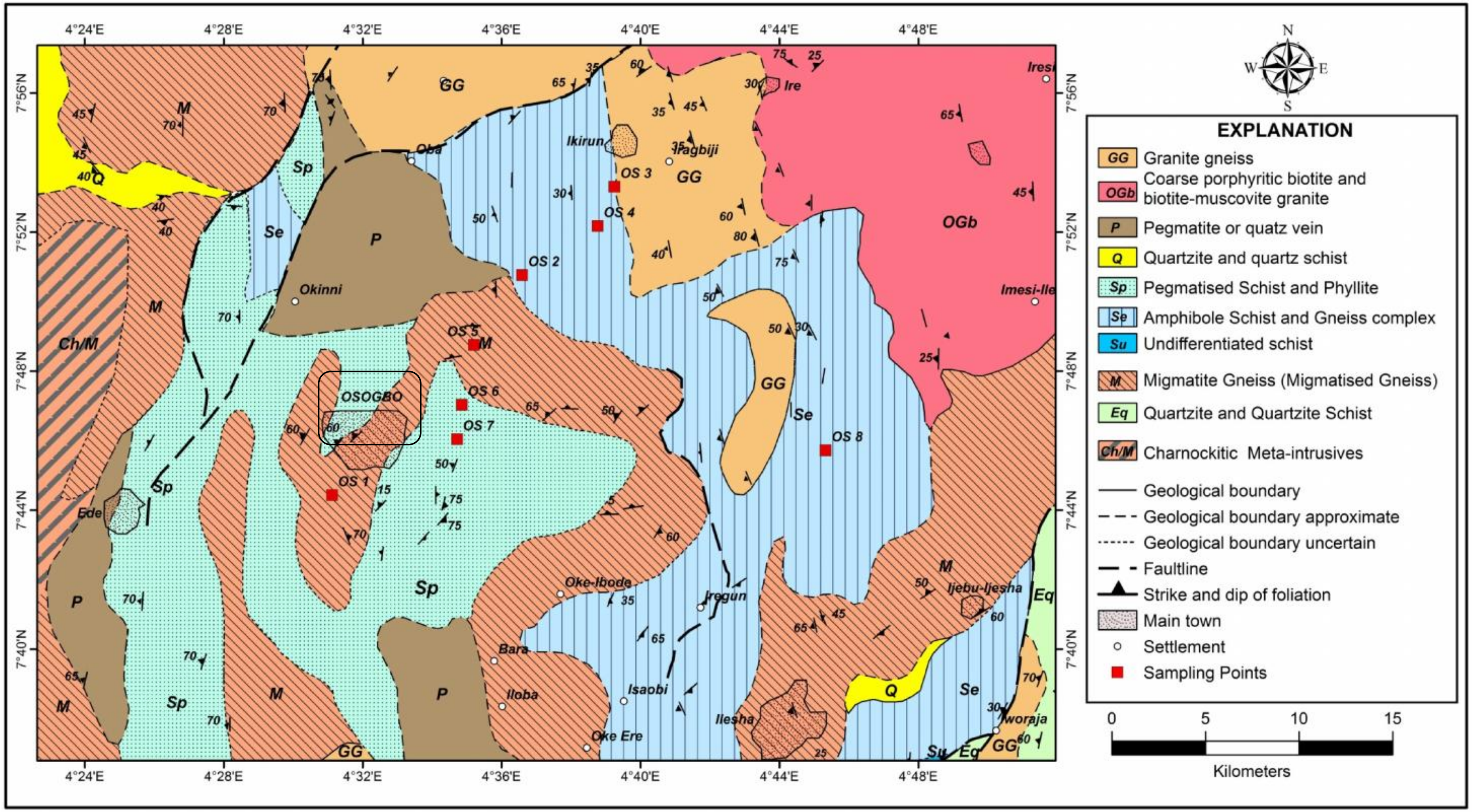
These are found in the eastern parts of the Ilesha area, occurring mainly as quartzite/quartz schist that form north-south strike ridges standing out in bold relief. They occur prominently in the adjacent district farther east and north of the area, particularly around Okemesi. The rocks overlie the strongly deformed Ilesha metaclastics and commonly between both sequences may be observed along major road cuttings and valleys in the north-east of Ilesha. Hubbard (1975) suggested a thrusted interface between them. Tight to isoclinals slightly overturned? folding with northerly axes and right angle fractured system, define the structural pattern of the Effon rocks. Concordant veins and lenses are found in outcrops, in addition to pegmatite veins composed mainly of quartz, microcline, muscovite and tourmaline developed with cross joints. In some places, quartz rich lenses contain disseminated metallic crystals, notably galena. In the south, the Ipetu granite intrudes the sequence. Quartz is dominant in most samples, while muscovites, chlorites and sericite occur in different proportions. Biotite is seldom present. Chemical data shows that the rocks are largely metamorphosed sandstones containing minor arkosic interrelation. The petrographic and structural features of the Effon quartzitic rock are similar to those of the psammite and metasediments of the Anka schist belt (north-west Nigeria), which also overlie pellitic and semi-pellitic rocks (Holt *et al*., 1978). However, small flakes of sericites that are developed at plains are considered to be late to post-deformation and may be related to the metasomatic/thermal metamorphic effect of the Pan-African event. The Effon quartzitic sequence is considered to represent a renewed face of sedimentation in the later Proterozoic.

3.8.2.2 Ilesha amphibolites and Ilesh metaclastics

Two structural units with contrasting lithology have been identified within this study area. These units are differentiated by the North North East-trending Ifewara fault zone. Rock units like amphibolites, amphibole-schists and pellitic-schists, granite, gneiss and pegmatite occurring together, were found to the west of the fault zone, while to the east of the fault zone units like quartzite, quartz-schist, quartzo-feldspathic gneiss and iron-rich schists were found (Adepelumi *et al*., 2008, Elueze and Kehinde-Philips, 1993, Kehinde-Phillips and Tietz, 1995).

Caby and Boesse (2001) (cited in Adepelumi *et al.*, 2008) identified two prominent structural styles which characterise the rocks of this area: (1) a low-plunging lineation that suggests a general transpressive regime; and (2) a recumbent.

Intrusive suites around Ilesha as identified by Elueze and Kehinde-Phillips (1993) consist of Pan-African granitic units of 600 Ma. Other minor rock units include garnet-quartz-chlorite bodies, biotite-garnet rocks, syenite bodies and dolerites. Ajayi (1980) (cited in Elueze and Kehinde-Phillips, 1993) identified the source of amphibolite in the area as the product of an altered derivative of an ultramafic rock band related to basic volcanics. A closer look at the field relations revealed that talc-schist, tremolite and anthophyllite within this region were exclusively underlain by amphibolites. Low temperature regional metamorphism, coupled with hydrothermal alteration, were responsible for the development of varieties of schist in the area, while chemical weathering of the parent rock material resulted in lateritic soil development over the rock units. Figure 3.5 shows each sampling point imposed on the geology of the area.



Source: Modified after NGSA (2008).

Figure 3.5: Geological map of Osogbo (OS) with sampling points

The processes of laterisation within the present study areas was investigated by Emofurieta and Salami (1993) while looking at the geochemical dispersion patterns associated with the laterisation process at Ile-Ife. It is reported that the soils derived from the melanocratic bands are SiO_2 rich, compared to soils derived from the leucocratic bands. Based on their average $SiO_2/Al_2O_3 + Fe_2O_3$ ratios, the soils derived from the melanocratic bands are lateritic, whilst the leucocratic derivatives were described as non-laterite. In the author's opinion, based on the earlier definition of lateritic soil and on the field experience around that region, those leucocratic derivatives could still be described as lateritic soils, knowing that they could as well reach matured stage of laterisation (become mature laterites) with time. See Figure 3.6 below.

Source: Author’s own (2016).

Figure 3.6: ***An exposed lateritic profile with a varying degree of laterisation due to mineralogy***

Abundant leucocratic bands are found with varying degrees of laterisation within this study area. This implies that the degree of laterisation varies as well, based on the terrain or topography, mineralogy of the parent rock material and time factor.

Figures 3.7 to 3.9 show some of the sampling locations with their descriptions.

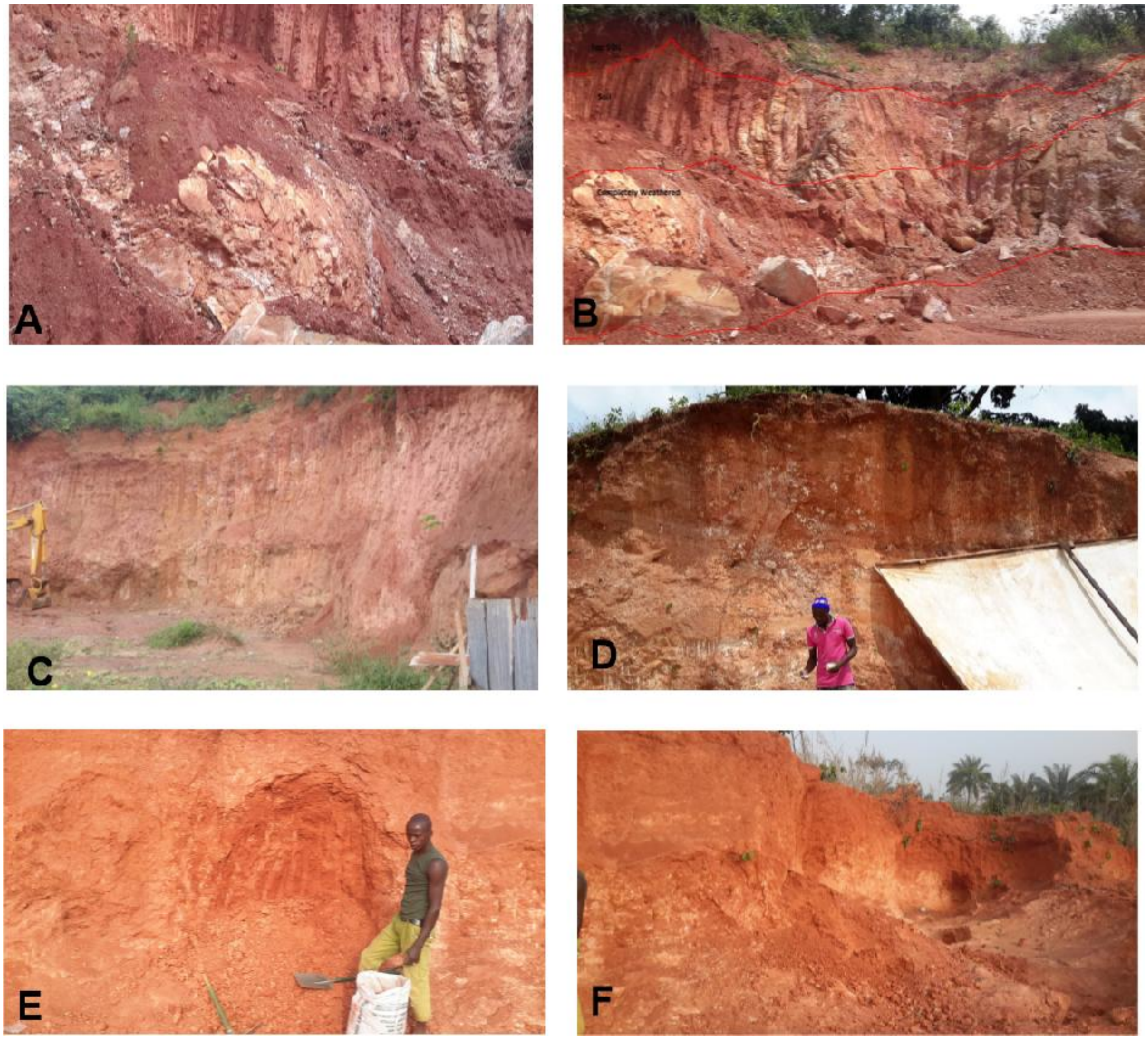


Figure 7.7: Sampling locations in Osogbo (OS); (a)and (b) shows the gravelly stone layer within the lateritic profile, (c) shows a profile formed over pegmatite, (d) profile rich in kaolin, (e)and (f) sample collection.

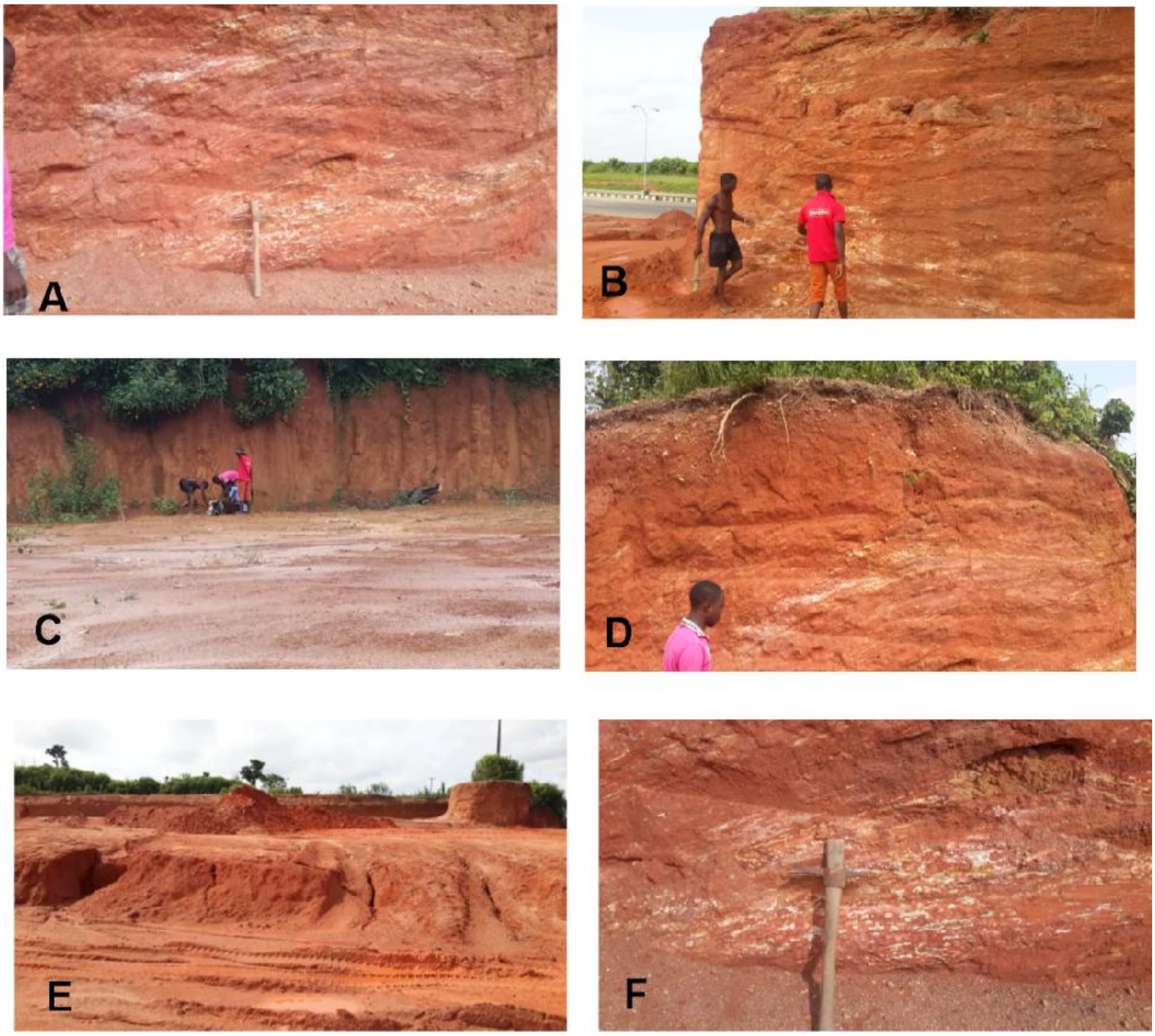


Figure 8: Sampling locations in Osogbo (OS) and Ado (ADK); (a), (b), (c) and (d) are locations within OS while (e)and (f) are locations within ADK, note the relics of foliation within the (f) profile.

Figure 9.9: Sampling locations in Ado (ADK); (a), (b), (c) and (d) shows variation in colour according to their parent rock materials.

3.9 CONCLUSION

In conclusion, the specific geology of the study areas has been described in this chapter to provide an understanding of the bedrock geology and therefore the expected residual soils and properties of these soils.

Chapter 4

METHODOLOGY

4.1 INTRODUCTION

The method of study for this research involves three main aspects, namely: field investigation, laboratory analyses and Compressed Earth Brick making.

4.2 FIELD INVESTIGATION

4.2.1 Literature study

This involved a detailed review of existing reports on the study areas, and a study of topographical and geological maps of each location in order to gain adequate knowledge of the areas. Historic construction of buildings and uses of construction materials were equally studied.

4.2.2 Preliminary site visit

A preliminary site visit of the whole terrain (two study areas) was carried out with a view to understanding the general outlook of the whole study areas, identifying their similarities and differences in terms of geography and landforms. At this stage, probable sampling sites were identified based on lithology and proximity to residential areas.

4.2.3 In situ classification and soil sampling

Based on the information gathered during the preliminary field visit, a second visit involved the description of soils including soil texture (grading), colour and moisture of soil. At this stage, in situ testing of the soils were done. This involved a **visual description** which estimates the relative proportions of the grain sizes and differentiate lateritic soil from top soil through colour differences; **a smell test** to differentiate organic soil from other types of soil (organic soils smells musty); and a **touch** test which categorises the soils according to texture. Soil sampling was done by taking soils from an average of 2 m depth from the ground surface. Generally, based on recommendations from the preliminary visit, soil sampling was done mainly at borrow pits where lateritic soils were mined for building and road constructions. (See Figure 4.1 showing some of the sites.)

Source: Author’s own (2016).

Figure 4.1: ***Some borrow pits where soil samples were taken***

The shallowest sample depth was about 2 m below the ground surface and the deepest up to about 10 m depth at some very deep borrow pits. In all, 25 disturbed samples were taken from 19 locations with 13 samples around Osogbo and 12 samples from Ado-Ekiti. Soil samples within schist belts (Osogbo) tend to have different properties within short proximity and necessitated the collection of two different soil samples based on change in colour within a single sampling locality (see Figure 3.6 in Chapter 3). Soil samples from the various sampling localities comprised of 50 kg samples to cater for the different laboratory analyses (about 20 kg) and to produce a minimum of four (4) standard size compressed earth bricks (29.5 cm × 14 cm × 9 cm).

Soil samples were bagged and labelled ADK 1 to ADK 11 for samples from the Ado-Ekiti site and those from Osogbo were labelled OS 1 to OS 8. Samples from the same locality were differentiated by adding the letters A and B for each individual sample.

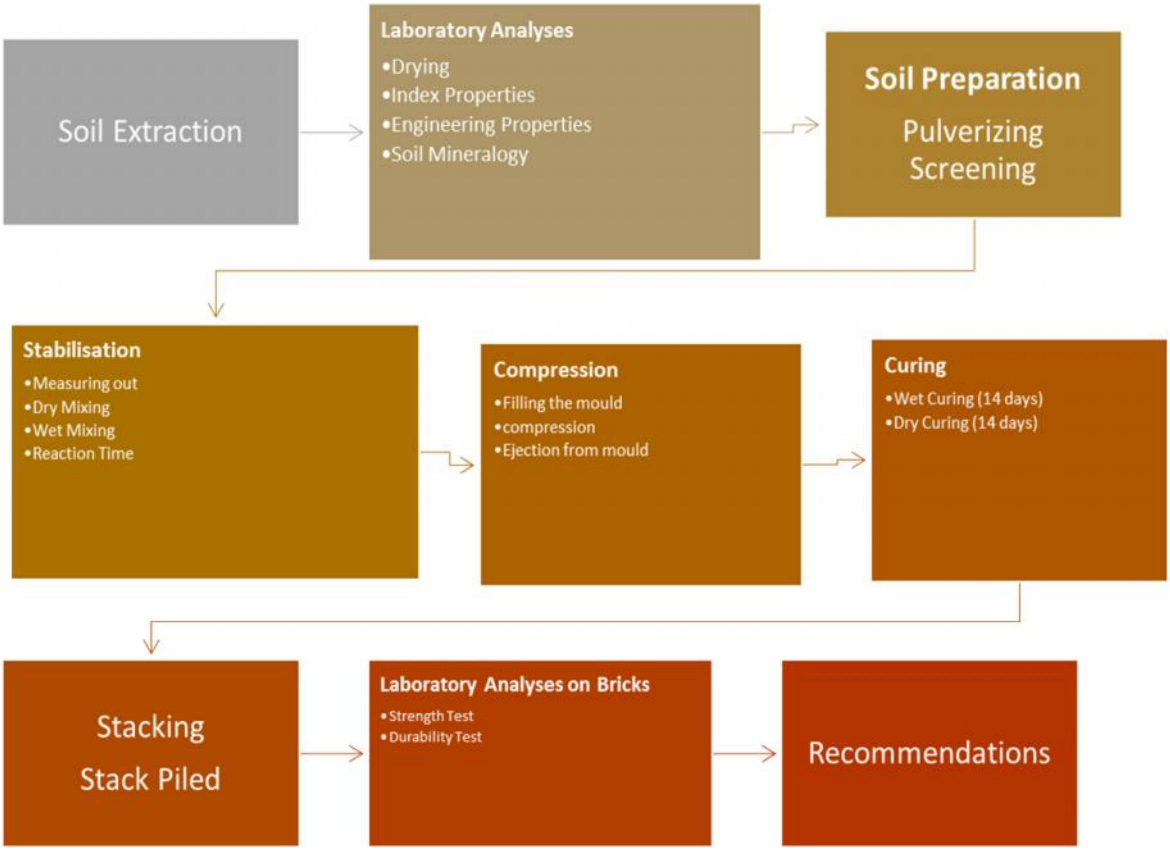
4.2.4 Ground truthing

The field geology of the study areas was correlated with the published geological maps by the Nigerian Geological Survey Agency (NGSA, 2006 and 2009). Geological ground truthing was conducted with the aim of matching the published map with the reality on the ground and establishing the various geological features of the area based on the published maps. Most of the information on the map were verified and minor details were added in order to produce large-scaled maps of the two study areas.

4.3 LABORATORY ANALYSES

The soil samples were tested for index and mechanical properties, mineralogy and chemical compositions were determined and the strength of the produced bricks were also determined. The index and engineering properties as well as the brick compressive strengths were determined at the soil laboratory in the Department of Civil Engineering, Obafemi Awolowo University, Ile-Ife, Nigeria and the soil mineralogy and chemical analyses (XRD and XRF) were carried out at the Department of Geology, University of Pretoria, South Africa.

A summary chart of the main sections of research methods is shown in Figure 4.2.



Source: Author's own (2016).

Figure 4.2: Summary chart of the research methodology

4.3.1 Sample preparation

This involves the pre-test preparation by air-drying the disturbed soil samples for about three weeks prior to further laboratory tests. During the period of air drying the samples were continuously turned over to ensure effective air drying. In addition to this, some of the soil samples were pulverised to increase the surface area and to allow for proper grain size sorting of the samples. The following tests were conducted on the samples: classifications based on tests, including specific gravity; grain size distribution and Atterberg limits. Geotechnical tests include compaction tests, direct shear tests, and California bearing ratio (CBR) tests. All tests were carried out according to the British Standard (BS), BS 1377 (1990) procedure, with little modifications where necessary. The following section deals with test detail procedure of the various tests.

4.3.2 Specific gravity

By definition, specific gravity is the ratio of the weight of a given volume of material to the weight of an equal volume in water (Liu and Evett, 2009). In engineering geology, specific gravity of soils refers to the specific gravity of solid matter of a given soil sample to the weight of an equal volume of water.

Specific gravity: $$Gs = \frac{Mass\ of\ soil\ solids}{Mass\ of\ equivalent\ volume\ of\ water}$$ *Equation 4.1*

$$Gs = \frac{Ws}{yw}$$ *Equation 4.2*

Standard procedure was followed according the stipulation of BS 1377 (1990).

4.3.3 Grain size distribution analyses

The analysis was carried out in order to determine the particle size distribution of the soils. It is widely used in the engineering classification of soils. Particle size properties influence engineering behaviour of soils. The test usually involves both mechanical and hydrometer analyses. The test procedures are discussed in the following sections.

4.3.3.1 Wet-sieving

It involved soaking about 500 g of the air-dried soil sample in a Calgon solution (sodium hexametaphosphate) as a dispersing agent, stirred and left overnight. The sample was then washed after a period of 24 hours. The washing involved separating the silt and clay fractions from the coarse fractions. The separation was effected by using a sieve with aperture openings of 75 µm. The separated coarse fractions were oven-dried for 24 hours. These were later dry-sieved using a set of sieves and amechanical shaker.

4.3.3.2 Mechanical/dry-sieving:

Fractions retained on sieve number 200 after wet-sieving was oven-dried and then subjected to mechanical sieving using a shaker. This analysis involved the separation of soils into different portions with the use of a stack of sieves with different mesh sizes. The apparatus used include a set of sieves with standard sizes, sieves brush, weighing balances, a large pan and sieve shaker.

The BS 1377, Part 2 (1990) test procedure was followed with a caution taken to obtain an adequate result: Prolonged shaking by the sieve shaker was avoided as this may lead to severe breakdown of some particles (Malomo, 1977).

4.3.3.3 Hydrometer analysis

This analysis covers the quantitative determination of particles finer and smaller in diameter than 75 microns. The portion passing through the 75-micron sieve obtained from wet-sieving was oven-dried for a period of 24 hours. This was allowed to cool and 50 g was weighed. The apparatus used include a hydrometer, two sedimentation cylinders (1 000 cm^3), a constant temperature bath, thermometer and a stopwatch. The test procedure followed the stipulations of BS 1377, Part 2 (1990).

Important geotechnical parameters of soil obtainable from the grading curves that are useful in the classification of the soil include the following:

- Effective grain size (D_{10}).
- Coefficient of uniformity (Cu).

4.3.4 Consistency limits determination

Generally known as the Atterberg limits, it gives the plasticity characteristics of a cohesive soil. Four states of consistency were established by Atterberg, namely the liquid, plastic, semi-solid and solid states. Their separation boundaries are the liquid limit, plastic limit and the shrinkage limit, respectively (Liu and Evett, 2009). The consistency limits are expressed as a percentage of moisture content reckoned to the dry weight. The difference between the liquid and plastic limits gives the plasticity index, which is the range of moisture content over which the soil remains plastic. The plasticity index and liquid limit are very important in soil classification and forms the basis of Casagrande's plasticity chart.

4.3.4.1 Liquid limit determination

It is the water content corresponding to the arbitrary limit between the liquid and plastic state of consistency of a soil. It is the minimum water content at which the soil is still in the liquid state, but the soil can flow under its own weight, a condition in which it does not have shear strength any more.

4.3.4.2 Plastic limit determination

It is the minimum water content at which soil begins to crumble when rolled into a thread of approximately 3 mm in diameter.

Plastic limit: $$PL = \frac{Ww}{Wd} \times 100$$ *Equation 4.3*

Where:

W_w = Wet weight

W_d = Dry weight

4.3.4.3 Plasticity index

This is the amount of water required to change the soil from its plastic limits to its liquid limit.

Plasticity Index: $$PI = LL - PL$$ *Equation 4.4*

Where:

PI = Plasticity index

LL = Liquid limit

PL = Plastic limit

4.3.4.4 Shrinkage limit determination

The shrinkage limit is the maximum water content at which a further reduction in water content will not cause a decrease in the volume of a soil mass. It is the lowest water content at which the soil can still be completely saturated. It is the line that separates semisolids from the solid state. Another soil parameter obtained from the shrinkage limit is the shrinkage ratio; it is defined as the ratio of a given volume change, expressed as a percentage of the dry volume to the corresponding change in water content above the shrinkage limit expressed as a percentage of the mass of oven-dried soil.

Shrinkage ratio: $$R = \frac{\Delta V / Vo}{\Delta w / Wo}$$ *Equation 4.5*

Where:

ΔV = Soil volume change in cm^3

Vo = Volume of oven-dried sample

Δw = Change in water content in g

Wo = Weight of oven-dried soil in g

$$\Delta w = (\Delta w)(\gamma w)$$ *Equation 4.6*

Simplifying Equation 4.5 by substituting gives:

$$R = \frac{Wo}{Vo}$$ *Equation 4.7*

Shrinkage Limit: $$SL = w - \left(\frac{V - Vo}{Wo} \times 100\right)$$ *Equation 4.8*

Where:

w = water content of wet soil in %

V = volume of wet soil pat

V_0 = volume of oven-dried soil pat

W_0 = weight of oven-dried soil pat

All the Atterberg limit tests and parameters were carried out in accordance with BS 1377, Part 2 (1990).

4.3.5 Compaction and California bearing ratio tests

Soils in the loose state consist of particles, water and air. Compaction test is aimed at detecting the moisture content, which will produce maximum density of the soil. This moisture content is known as the 'optimum moisture content', while the corresponding maximum density is known as 'maximum dry density'.

Compaction can be carried out at standard proctor (sometimes referred to as AASHTO), modified proctor (AASTHO) and West African levels. The scope of this work is limited to the West African level, based on the peculiarity of the south-western lateritic soils in order to avoid over-compaction leading to degradation of the soil (Gidigasu, 1976; Adeyemi and Oyeyemi, 2000). The moisture–density relationship for each sample was determined using the West African–AASHTO level of compaction (or Modified AASHTO level of compaction). The Modified AASHTO level has the tendency to over-compact lateritic soil (De Graft-Johnson *et al*., 1972, cited in Gidigasu, 1976) and was thus recommended for laterite gravel. The test procedure here was modified after the BS 1377, Part 4 (1990) to suit the peculiarity of the lateritic soil under study.

Apparatus used included a cylindrical mould of volume 0.00212 m^3, a 4.5 kg rammer, an oven, tray, scoop, mallet, sample extruder, collar, weighing balance and 4.75 mm sieve.

4.3.5.1 Test procedure

The air-dried soil sample weighing 6 kg was mixed thoroughly with water. The soil was compacted into a pre-weighted mould in five layers with each layer subjected to 27 blows in the West African level from 4.5 kg (44.5 N) rammer falling through a height of 0.46 m.

The compaction mould and the soil were weighed when the collar was removed. The extruder was used to remove the compacted soil from the mould after moisture content determination was carried out on a representative sample from the top, middle and bottom of the specimen. The procedure is repeated after addition of 3% by weight until the weight of the compacted soil was known. The dry density was plotted against the moisture content from which the OMC and MDD were obtained.

The California Bearing Ratio (CBR) test which assesses strength and bearing capacity of soils, especially for subgrade soils in road construction. Soils samples for this test are normally remoulded (i.e. compacted). Samples are prepared compacted according to BS 1377. The CBR was carried out at 100% dry density and optimum moisture content.

Apparatus: compaction mould (6-inch diameter and 7-inch height), collar, adjustable stem and perforated plate, weights, penetration piston (3 in 2 in area), loading (compression) machine with two dial gauges, standard compaction hammer, scales, soaking tank and oven. A standard procedure as stipulated by BS 1377, Part 4 (1990), was followed, after which the following equations were applied to determine the percentage CBR at 2.5 and 5.0 mm penetration:

CBR at 25 mm $= \frac{Corrected\ load\ at\ 2.5\ mm}{13.44} \times 100$ *Equation 4.1*

CBR at 50 mm $= \frac{Corrected\ load\ at\ 5.0\ mm}{20.16} \times 100$ *Equation 4.2*

Where 13.44 and 20.16 are standard load values.

4.3.6 Strength test (direct shear test)

The direct shear test measures the shear strength of a soil in the laboratory. It can be performed on both cohesive and cohesionless soil, hence suitable for estimating the shear strength of lateritic soils. It evaluates both cohesion, (c) and internal friction (φ) of the soil.

The apparatus used included a direct shear device, porous stone, shear loading device, shear load measuring device, axial load measuring device, cutting ring, knife and compacting equipment.

The direct shear test was carried out in accordance with BS 1377, Part 7 (1990) standards for soil tests.

4.4 SOIL MINERALOGY

4.4.1 X-Ray diffraction analysis

The samples were prepared according to the standardised PANalytical backloading system, which provides nearly random distribution of the particles.

The samples were analysed using PANalytical X'Pert Pro powder diffractometer in θ–θ configuration with an X'Celerator detector and variable divergence and fixed receiving slits with Fe filtered Co-Kα radiation (λ=1.789Å). The phases were identified using X'Pert High-Score Plus Software (Panalytical B.V., The Netherlands). Graphical representations of the qualitative results were presented in Appendix B.

The relative phase amounts (weight percentage) were estimated using the Rietveld method (Autoquan Program). Errors are on the three-sigma level in the column to the right of the amount. Amorphous phases, if present, were not taken into consideration in the quantification.

4.4.2 X-Ray fluorescence analysis

The samples were milled in a tungsten-carbide milling pot to achieve particle sizes of <75 micron. The samples were dried at 100 °C and roasted at 1 000 °C to determine Loss On Ignition (LOI) values. A 1 g sample was mixed with 6 g Lithiumteraborate flux and fused at 1 050 °C to make a stable fused glass bead. For trace element analyses the sample was mixed with PVA binder and pressed in an aluminium cup at 10 tons.

The Thermo Fisher ARL Perform 'X Sequential XRF with OXSAS Software was used for the analysis. Blank and certified reference materials were analysed with each batch of samples.

4.5 COMPRESSED EARTH BRICK TESTS AND METHODOLOGY

4.5.1 Brick making

Brick making forms part of the assessment of the suitability of various lateritic soils taken from different locations. This was carried out with the aim of understanding their responses to stabilisation, compression and curing within the context of this research.

Soils for these bricks were used based on location and not based on different samples from the same locality. Firstly various samples from one locality were mixed (e.g. OS 1A and OS 1B mixed to form OS 1), followed by soil screening and pulverising after which 50 kg were measured out. Portland limestone cement 42.5R grade was used as stabiliser and 2.5 and 5.0 kg were added to the measured soil, depending on the soil texture and results from previous tests carried out in the laboratory. These were properly mixed in a dry state to form homogenous mass of consistent composition and texture, after which water was added. Water was added based on the optimum moisture content determined from the compaction tests. A drop test was carried out on the mixture before transferring it into the mould. The soil in this moist state was transferred into a manual CIRAM (QR 40) hand press mould with low pressure (about 2–4 MPa) and pressed about three times before removing the damp brick from the mould. The brick was immediately transferred to a curing section where it was moisture-cured for 14 days before exposing the bricks to air and sunlight for another 14 days. This procedure was repeated for all the bricks from the 15 locations with an average of four bricks per location. The pictorial description of the processes involved is given in Figure 4.3 illustrating the step-by-step procedure.

Source: Author’s own (2016).

Figure 4.3: ***Pictorial description of processes involved in brick making (starting from soil screening, stabilizer, mixing, compression, curing, stack piling and use)***

4.5.2 Compressive strength test for bricks

A dry compressive strength test was carried out based on the outlines of ARS (1998) and HB-95 (2011) specification of earth buildings.

- **Apparatus**
  Compression testing machine, scale for measuring brick dimension and bricks.
- **Procedure**
  The initial dimensions of the bricks were established and the cross-sectional area was calculated from these dimensions. After this, the brick sample was placed in-between the jaws of the compression testing machine and load was applied progressively until the brick sample started developing cracks, which signifies failing under the applied load. At this stage the machine was stopped and the applied load at failure was measured. The same procedure was repeated for the rest of the bricks.

The compressive strength of the bricks was calculated as follows:

Compressive Strength: $$\sigma = \frac{P}{A}$$ *Equation 4.3*

Where P is the maximum load at failure and A, is the cross-sectional area of the brick.

4.5.3 Exposure test

Representative samples of each brick from different locations were used to form a wall without cement mortar or any form of jointing. This was exposed in an open space to test for its resistance to weathering since January 2014. These bricks were expected to be exposed for a minimum duration of one year during which a complete cycle of climatic conditions would act on it. The quality and physical properties of these bricks in terms of weight and change in colour were evaluated after one year.

4.6 STATISTICAL EVALUATION

Different statistical approaches were employed to appreciate the impact of the various soil tests carried out on the strength and durability of the bricks, including the visual impacts.

Mean, median and mode expresses distribution characteristics of data.

Multivariate regression analyses and linear correlation were used to establish the relationships between various variables and to determine their rate of dependence on each other. This involved the correlation coefficient, T-test and F-test. Correlation coefficient measures the ratio of co-variance between two variables in relation to the product of their standard deviation. Pearson's correlation coefficient between variables ranges between positive unity (+1) and negative (–1). Correlation of +1 signifies a strong direct relationship between two variables in question, while a –1 signifies that a variable is inversely related to the other (Davis, 2002). A T-test and F-test were carried out at both 0.05 and 0.1 level of significance, at 95% and 90% confidence intervals. This was carefully chosen in line with this research objective. Most of the parameters in question can conveniently accommodate a degree of uncertainty which cannot be over-emphasised generally in engineering geology.

The study adopts a linear regression equation to evaluate the effect of X_1, X_2-----X_k on dependant variable Y. (Unless otherwise stated, the dependant variable Y is the brick strength and $X_1 \ldots . X_k$ are the variable parameters in terms of the index and engineering parameters.) The population regression equation for the (PRF) is stated as follows:

$Y = \beta_0 + \beta_1 X_1 + \beta_2 X_2 +$ -------------------- $+ \beta_k X_k + \mu$ --------- *Equation 4.4*

Since this research only covers a certain part of the whole population from where the representative samples were taken and conclusions drawn are based on the analyses carried out on the samples, generalisations can be made based on these conclusions. Therefore, the sample regression function (SRF) is as follows:

$Y = \beta_0 + \beta_1 X_1 + \beta_2 X_2 +$ ------------------- $+ \beta_k X_k + e$ ---------- *Equation 4.5*

The T-test and F-test for all samples were computed using the computer statistical software package called IBM SPSS® Statistics 20 Software (2011), which gives T-test and F-test values at a 95% confidence interval. The results were again tested at 90% confidence interval as well. This implies that any result less than 0.05 for 95% confidence interval and 0.1 for 90% confidence interval are significant, while any result greater than 0.05 and 0.1, respectively, is not significant.

The various mathematical formulae for the different parameters are given in Appendix A.

4.7 CONCLUSION

The present study adopts the simplest methodology to encourage the use of earth materials especially in Africa. The methodology presented here covers the basic and useful tests for soils utilised in brick making. Procedures for basic engineering soil tests were omitted in the thesis because they followed the same prescribed in the earlier stated BS standard for soil tests. The CBR was done with the aim of establishing its usefulness in brick making. The tests and methodology is regarded as useful to predict the behaviour of soils in brick making and are proposed as standard for tropical soils in earth brick making.

Chapter 5

RESULTS

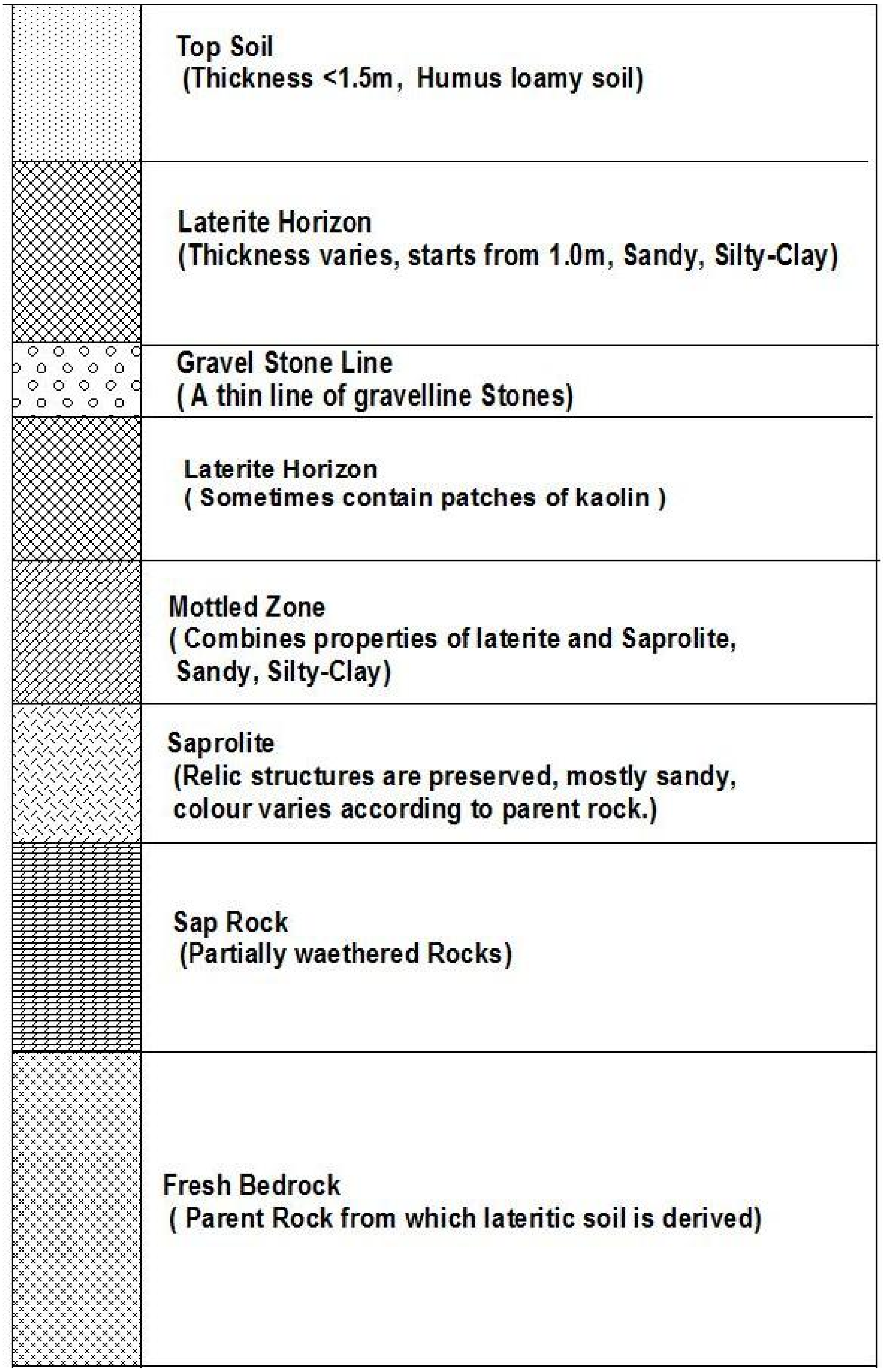
5.1 INTRODUCTION

In this chapter the findings from the field and laboratory studies are presented in the form of pictures, figures, summary tables and charts. The graphs and raw data from which these summaries were drawn are attached in Appendix B. Detail discussions of the results are presented in Chapter 6. Descriptive statistical analyses were used and regarded as necessary to come up with some of the results presented here. Results presented in this chapter include sample locations, geographic coordinates and elevations, rock types underlying each location, soil profile charts, grain size distribution, Atterberg Limits, specific gravity, shear strength, California Bearing Ratios (CBRs), X-ray diffraction analysis, X-ray fluorescence analysis, compressive strength of bricks and summary of regression analyses.

5.2 SAMPLE LOCATIONS, ROCK TYPES AND SOIL PROFILE

Figure 5.1 presents a generalised soil profile, reflecting the different horizons of the typical lateritic soils observed around the two study areas. Most of the sampling locations shared the same profile with minor differences, mainly in terms of the presence or absence of the gravel stone line.

A summary of the exact location of each sampling point, its lithology, colour and textural characteristics is provided in Table 5.1. As previously mentioned two samples were collected from some locations (Section 4.2.3 in Chapter 4).



Source: Author's own (2016).

Figure 5.1: ***Typical lateritic soil profile common to most sampling location***

TABLE 5.1: SAMPLE POINTS, ROCK TYPES AND GPS COORDINATES

S/N	Location	Northing (Latitude)	Easting (Longitude)	Altitude (m)	Sampling Points	Colour	Texture	Rock Type
1	ADK 1	07° 37.025[1]	005° 11.935	450	A	Whitish brown	Medium to Fine	Charnockite
					B	Rusty Red	Medium	
2	ADK 2	07° 36.968	005° 12.017	458		Rusty Red	Medium to Fine	Charnockite
3	ADK 3	07° 37.961	005° 11.637	481		Reddish	Medium to Coarse	Porphyritic Granite
4	ADK 4	07° 33.833	005° 12.702	421		Reddish	Medium to Coarse	Quartzite
5	ADK 5	07° 38.300	005° 11.996	440		Liver red	Medium to Coarse	Porphyritic Granite
6	ADK 6	07° 37.974	005° 11.893	462		Yellowish Red	Medium to Fine	Porphyritic Granite
7	ADK 7	07° 46.251	005° 15.097	573		Reddish	Medium to Coarse	Migmatite
8	ADK 8	07° 41.506	005° 15.170	406		Reddish brown	Medium to Coarse	Migmatite
9	ADK 9	07° 49.051	005° 20.936	575		Reddish	Medium to Fine	Charnockite
10	ADK 10	07° 47.613	005° 16.543	582		Yellowish red	Medium to Coarse	Migmatite
11	ADK 11	07° 43.741	004° 57.342	450		Reddish	Medium to Coarse	Quartzite
12	OS 1	07° 44.435	004° 31.119	344	A	Reddish Brown	Medium to Fine	Migmatite Gneiss
					B	Reddish	Medium to Fine	
13	OS 2	07° 50.764	004° 36.592	386	A	Liver red	Medium to Coarse	Amphibole Schist
					B	Yellowish brown	Medium to Coarse	

S/N	Location	Northing (Latitude)	Easting (Longitude)	Altitude (m)	Sampling Points	Colour	Texture	Rock Type
14	OS 3	07° 53.302	004° 39.249	395	A	Reddish	Medium to Fine	Amphibole Schist
					B	Reddish brown	Medium to Fine	
15	OS 4	07° 52.177	004° 38.764	391	A	Reddish brown	Medium to Fine	Amphibole Schist
					B	Yellowish brown	toMedium – Fine	
16	OS 5	07° 48.751	004° 35.206	370	A	Purple	Medium ro Coarse	Migmatite Gneiss
					B	Yellowish brown	Medium to Coarse	
17	OS 6	07° 47.032	004° 34.856	344		Yellowish brown	Medium to Coarse	Pegmatised Schist
18	OS 7	07° 46.044	004° 34.715	342	A	Whitish brown	Fine	Pegmatised Schist
					B	Brown	Medium to Fine	
19	OS 8	07° 45.721	004° 45.331	364		Reddish brown	Medium to Fine	Amphibole Schist

OS = Osogbo
ADK = Ado Ekiti.
Easting and Northing in degree and minutes.

5.3 INDEX PROPERTIES

Tables 5.2 and 5.3 gives a short summary of the results and deductions from the index tests. Presented in the tables are the specific gravity, grading results with Atterberg Limits which were used to classify the soils according to the AASHTO and USCS engineering soil classification systems. A detailed discussion of the result is presented in the subsequent section.

TABLE 5.2: SUMMARY TABLE OF THE INDEX PROPERTIES OF THE SOIL (ADK)

Index Properties	Specific Gravity	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Amount of Fines	Amount of Coarse	Liquid Limit	Natural Moisture Content(%)	Plastic Limit	Plasticity Index	Linear Shrinkage	AASHTO Classifica-tion	USCS
ADK 1	2.63	26.4	36.84	28.49	7.99	36.48	63.24	32.4	10.1	17.1	15.3	1	A-6	Clayey sand
ADK 2	2.71	25.64	36.14	29.73	8	37.73	61.78	26.01	17.1	12.8	13.21	0.8	A-6	Clayey sand
ADK 3	2.65	18.87	37.37	26.91	10.22	37.13	56.24	55.04	38.1	23.9	31.14	0.9	A-7-6	Clayey sand
ADK 4	2.25	37.69	44.89	11.71	2.77	14.48	82.58	37.33	12.7	22.2	15.13	1	A-2-6	Silty sand
ADK 5	2.72	20.76	36.89	39.35	3	42.35	57.65	46.2	38.5	18	28.2	0.9	A-7-6	Clayey sand
ADK 6	2.64	1.2	70.5	22	5	27	71.7	33.96	27.7	16.1	17.86	1.1	A-2-6	Silty sand
ADK 7	2.87	8.71	43	36.51	11.49	48	51.71	32.17	23.3	19.32	12.85	1	A-6	Clayey sand
ADK 8	2.64	21.2	44.42	34.66	0	34.66	65.62	40.17	18.6	17.6	22.57	1.1	A-6	Clayey sand
ADK 9	2.89	12.06	43.38	34.41	10	44.41	55.44	38.56	24.9	21.4	17.16	0.9	A-6	Clayey sand
ADK 10	2.9	4.14	46.16	35.65	13.89	49.54	50.3	31.56	12.4	18.17	13.39	0.8	A-6	Clayey sand
ADK 11	2.85	20.5	40.78	33.58	5.24	38.82	61.28	32.1	13.2	16.6	15.5	0.9	A-6	Silty sand

Source: Author's own (2016).

TABLE 5.3: SUMMARY TABLE OF THE INDEX PROPERTIES OF THE SOIL (OS)

Index Properties	Specific Gravity	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Amount of Fines	Number of Coarse	Liquid Limit	Natural Moisture Content(%)	Plastic Limit	Plasticity Index	Linear Shrinkage	AASHTO Classifica-tion	USCS
OS 1A	2.72	38.39	47.11	14.49	0	14.49	85.5	40.18	20.8	13.6	26.58	0.9	A-2-6	Silty sand
OS 1B	2.64	7.5	44.32	24.19	24.06	48.25	51.82	42.56	17.7	18.2	24.36	1.2	A-7-6	Clayey sand
OS 2A	2.72	3.81	55.47	23.7	16.4	40.1	59.28	51.82	37.8	35.2	16.62	1.1	A-7-5	Silty sand
OS 2B	2.62	25.96	43.54	26.16	2.54	28.7	69.5	44.56	16.8	32.5	12.06	1	A-2-7	Clayey sand
OS 3A	2.54	7.76	46.24	43	0	43	54	44.94	15.3	29.9	15.04	1.2	A-7-6	Silty sand
OS 3B	2.62	37.93	47.03	14.74	0	14.74	84.96	42.7	18.8	30.4	12.3	0.9	A-2-7	Silty sand
OS 4A	2.71	17.23	34.13	28.93	19.31	48.24	51.36	51.16	30.4	16.6	34.56	1.3	A-7-6	Clayey sand
OS 4B	2.56	2.07	53.89	21.78	22.26	44.04	55.96	43.92	20.1	19.8	24.12	1.2	A-7-6	Clayey sand
OS 5A	2.8	13.98	46.72	28.61	7.49	36.1	60.7	35.63	19.5	17.5	18.13	1.2	A-6	Clayey sand
OS 5B	2.76	23.54	41.89	25.14	7.24	32.38	65.43	38.6	20.0	33.7	4.9	1.1	A-2-4	Clayey sand
OS 6A	2.25	6.24	48.88	30.92	12.74	43.66	55.12	53.76	41.3	29.2	24.56	1.3	A-7-6	Clayey sand
OS 7A	2.74	31.42	41.1	23.51	0	23.51	72.52	47.53	18.2	21.9	25.63	1.2	A-2-7	Silty sand
OS 7B	2.5	12.16	39.95	29.52	17.7	47.22	52.11	36.26	13.4	17.4	18.86	0.9	A-6	Clayey sand
OS 8	2.7	8.71	43	36.51	11.49	48	51.71	40.79	14.9	18.1	22.69	1.2	A-7-6	Clayey sand

Source: Author's own (2016).

5.4 ENGINEERING PROPERTIES

Tables 5.4 and 5.5 summarises the engineering properties of the soil, including compaction characteristics (optimum moisture content and maximum dry density), California Bearing Ratio (CBR), shear strength (cohesion c and internal friction ϕ). CBR values at 95% compaction effort and different percentages of added cement are also presented in order to determine the response of the soil-to-cement content variation and establish if there is any correlation with the strength of bricks. Other parameters of interest are the reduction in strength due to soaking and increase in moisture content. These were all deduced from the results of CBR to ascertain the influence of ingress of water on the soil. Compaction characteristics at different cement percentages are presented with corresponding optimum moisture content and maximum dry density to identify a suitable cement content that gives the best OMC/MDD in relation to the proposed use of the soil.

TABLE 5.4: SUMMARY TABLE OF THE ENGINEERING PROPERTIES OF THE SOIL AT DIFFERENT CEMENT CONTENT (ADK)

	Engineering Properties	ADK 1	ADK 2	ADK 3	ADK 4	ADK 5	ADK 6	ADK 7	ADK 8
CBR at 0% Cement (%)	Unsoaked	34.15	15.63	22.47	22.40	14.88	50.30	17.56	31.25
	Soaked	10.64	7.96	8.33	10.57	6.62	13.99	6.70	11.68
	Reduction (%)	68.85	49.05	62.91	52.82	55.50	72.19	61.86	62.62
	Soaked MC	15.1	26.1	22.6	22.2	22.1	20.1	25.4	23.2
	Increase in MC	25.8	13.5	16.3	19.6	17.7	14.3	12.6	24.6
CBR at 5% Cement (%)	Unsoaked	37.35	24.63	30.36	25.97	17.34	54.91	22.77	45.76
	Soaked	11.46	12.35	10.49	11.61	7.74	15.33	8.41	15.03
	Reduction (%)	69.32	49.85	65.44	55.30	55.36	72.09	63.07	67.15
	Soaked MC	15.7	26.6	23.8	22.7	23.4	21.4	27.1	24.1
	Increase in MC	26.8	13.6	19.0	18.4	21.6	19.0	17.8	28.0
CBR at 10% Cement (%)	Unsoaked	46.88	46.21	35.42	56.70	21.35	53.27	31.62	45.46
	Soaked	14.43	10.19	10.12	14.96	9.08	15.10	10.86	14.14
	Reduction (%)	69.21	77.94	71.43	73.62	57.49	71.65	65.65	68.90
	Soaked MC	15.8	27.3	24.0	23.4	23.8	22.7	27.7	23.3
	Increase in MC	21.6	13.8	17.8	12.5	21.2	20.6	15.3	21.4
Compaction at 0% Cement	OMC (%)	12	23	19.4	18.6	18.8	17.6	22.6	18.6
	MDD (kg/m^3)	1 910	1 565	1 820.04	1 865.15	1 680	1 895	1 640	1 890
Compaction at 5% Cement	OMC (%)	12.4	23.4	20	19.2	19.2	18	23	18.8
	MDD (kg/m^3)	1 930	1 580.22	1 840	1 880	1 720	1 920.15	1 650	1 910
Compaction at 10% Cement	OMC (%)	13	24	20.4	20.8	19.6	18.8	24	19.2
	MDD (kg/m^3)	1 970.06	1 630	1 890	1 905	1 750	1 950	1 690.02	1 930
Shear Strength	C	29	80	35	110	34	150	120	144
	ϕ	40	41	42	38	40	24	38	36
	τ (kPa)	63.30	115.54	71.81	141.94	68.30	168.20	151.94	173.70

TABLE 5.5: SUMMARY TABLE OF THE ENGINEERING PROPERTIES OF THE SOIL AT DIFFERENT CEMENT CONTENT (OS)

	Engineering Properties	OS 1	OS 2	OS 3	OS 4	OS 5	OS 6	OS 7
CBR at 0% Cement (%)	Unsoaked	45.98	28.72	28.27	21.65	25.89	38.17	15.77
	Soaked	14.21	9.90	10.34	6.85	13.32	12.35	5.51
	Reduction (%)	69.09	65.54	63.42	68.38	48.56	67.64	65.09
	Soaked MC	19.8	21.0	19.2	25.8	20.0	19.0	22.0
	Increase in MC	20.5	19.4	18.3	16.3	20.6	14.6	22.1
CBR at 5% Cement (%)	Unsoaked	61.61	57.37	56.10	22.25	51.19	55.51	26.26
	Soaked	14.36	13.62	20.46	7.66	16.74	16.52	8.33
	Reduction (%)	76.69	76.26	63.53	65.55	67.30	70.24	68.27
	Soaked MC	21.5	23.9	20.3	26.1	21.3	20.8	22.6
	Increase in MC	26.4	32.6	22.5	14.4	26.5	21.2	20.2
CBR at 10% Cement (%)	Unsoaked	58.56	59.38	66.37	28.42	57.37	61.01	31.18
	Soaked	23.36	17.34	24.55	9.38	19.49	21.13	11.24
	Reduction (%)	60.10	70.80	63.00	67.02	66.02	65.37	63.96
	Soaked MC	22.5	24.1	22.0	26.6	20.4	22.1	23.2
	Increase in MC	28.0	27.9	28.1	12.6	18.3	22.8	20.9
Compaction at 0% Cement	OMC (%)	16.4	17.6	16.2	22.2	16.6	16.6	18
	MDD (kg/m^3)	1 830	1 740.1	1 835.06	1 630	1 730	1 799	1 758.1
Compaction at 5% Cement	OMC (%)	17	18	16.6	22.8	16.8	17.2	18.8
	MDD (kg/m^3)	1 860.2	1 765	1 850	1 650.1	1 750.1	1 820.05	1 805
Compaction at 10% Cement	OMC (%)	17.6	18.8	17.2	23.6	17.2	18	19.2
	MDD (kg/m^3)	1 910	1 790.1	1 880.12	1 690.15	1 805	1 870.1	1 840.02
Shear Strength	C	52	90	54	46	50	114	75
	ϕ	42	41	43	41	25	40	41
	τ (kPa)	88.81	125.54	92.12	81.54	69.06	148.30	110.54

5.5 SOIL MINERALOGY AND CHEMISTRY

The soil chemistry and mineralogy are summarised in Tables 5.6 and 5.7. Table 5.6 summarises the major oxides present in the soil, while Table 5.7 summarises the mineral compositions within each soil sample. Based on prior knowledge of the area and the kind of soil sample the important oxides and minerals considered in this study are emboldened.

Tables 5.8 to 5.11 present the statistical deductions from the results presented earlier. Different parameters were tested for their significance and/or influence on the strength of the bricks using the T-test and F-test. The T-test is an important statistical method in determining the difference between two sets of values to check if the given parameter is significant or not, while the F-test focuses on a set of parameters. Any degree of freedom has a specific values for 'T' or 'F' and is listed in the table at a given confidence limit. The basic rule is that when the calculated 'T' or 'F' is higher than the 'T' or 'F' in the table, the difference is adjudged to be significant. In this case just as explained earlier in Chapter 4 (section 4.6), the samples were tested both at 95% and 90% degrees of freedom.

TABLE 5.6: COMPLETE SUMMARY TABLE OF THE MAJOR OXIDES PRESENT IN SOILS (ADK AND OS)

%	SiO_2	TiO_2	Al_2O_3	Fe_2O_3	MnO	MgO	CaO	Na_2O	K_2O	P_2O_5	Cr_2O_3	NiO	V_2O_5	ZrO_2	CuO	LOI	TOTAL
Certified	**99.6**	**0.01**	**0.05**	**0.05**	**0.01**	**0.05**	**0.01**	**0.05**	**0.01**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**100**
Analysed	**99.56**	0.00	**0.01**	**0.01**	0.00	0.01	0.01	0.02	0.01	0.03	0.00	0.00	0.00	0.01	0.00	**0.10**	**99.77**
ADK1A	**48.33**	0.94	**34.20**	**2.94**	0.00	0.13	0.01	0.02	0.55	0.06	0.01	0.00	0.01	0.04	0.00	**12.17**	**99.42**
ADK1B	**43.94**	1.71	**25.75**	**13.61**	0.13	1.19	0.01	0.02	2.38	0.57	0.02	0.01	0.03	0.11	0.00	**10.42**	**99.91**
ADK2A	**46.22**	0.90	**35.18**	**2.72**	0.01	0.16	0.01	0.02	0.54	0.06	0.01	0.00	0.01	0.04	0.00	**14.07**	**99.97**
ADK3	**45.63**	0.91	**31.00**	**9.44**	0.05	0.17	0.01	0.02	0.29	0.08	0.01	0.00	0.02	0.05	0.00	**12.66**	**100.35**
ADK4	**43.05**	1.03	**29.09**	**12.87**	0.02	0.09	0.01	0.02	0.12	0.09	0.02	0.00	0.03	0.11	0.00	**12.90**	**99.46**
ADK5	**48.30**	0.81	**31.40**	**5.90**	0.02	0.26	0.01	0.02	1.47	0.07	0.00	0.00	0.01	0.18	0.00	**11.37**	**99.82**
ADK6	**62.00**	0.99	**21.78**	**5.74**	0.06	0.17	0.01	0.02	0.67	0.08	0.01	0.00	0.02	0.09	0.00	**8.60**	**100.24**
ADK7	**48.80**	0.95	**28.50**	**8.94**	*<0.01*	0.02	*<0.01*	*<0.01*	0.07	0.13	0.02	*<0.01*	0.01	*<0.01*	*<0.01*	**12.40**	**99.84**
ADK8	**43.44**	1.08	**26.73**	**18.13**	0.02	0.17	0.01	0.02	0.09	0.07	0.03	0.00	0.03	0.05	0.00	**12.04**	**101.93**
ADK9	**34.18**	1.80	**27.76**	**22.80**	0.04	0.07	0.01	0.02	0.07	0.17	0.02	0.01	0.05	0.07	0.00	**13.49**	**100.55**
ADK10	**41.20**	0.47	**32.19**	**12.07**	0.02	0.05	0.01	0.02	0.18	0.15	0.01	0.00	0.02	0.08	0.00	**13.43**	**99.89**
ADK11	**40.01**	1.70	**25.03**	**20.83**	0.04	0.36	0.01	0.02	0.38	0.11	0.02	0.00	0.06	0.05	0.00	**12.03**	**100.67**
OS1A	**51.83**	0.51	**28.42**	**5.48**	0.01	0.11	0.01	0.02	2.60	0.13	0.02	0.01	0.01	0.13	0.07	**10.35**	**100.69**
OS1B	**47.73**	0.79	**31.59**	**5.94**	0.01	0.13	0.01	0.02	1.28	0.10	0.01	0.01	0.01	0.09	0.00	**11.58**	**100.83**
OS2A	**42.77**	1.11	**31.86**	**10.23**	0.02	0.30	0.01	0.02	0.46	0.10	0.01	0.00	0.02	0.07	0.04	**12.87**	**99.32**
OS2B	**64.60**	0.10	**23.00**	**1.73**	*<0.01*	0.05	0.04	*<0.01*	2.84	0.06	*<0.01*	*<0.01*	*<0.01*	*<0.01*	*<0.01*	**7.41**	**99.82**
OS3A	**40.07**	1.30	**28.46**	**18.38**	0.05	0.13	0.01	0.02	0.26	0.12	0.02	0.00	0.05	0.07	0.00	**11.75**	**100.15**
OS3B	**38.84**	1.24	**28.35**	**18.53**	0.05	0.16	0.01	0.02	0.14	0.09	0.04	0.02	0.04	0.07	0.03	**13.18**	**100.97**
OS4A	**45.67**	0.16	**35.76**	**2.53**	0.01	0.09	0.01	0.02	0.73	0.07	0.05	0.03	0.01	0.01	0.00	**14.16**	**99.69**
OS4B	**41.75**	0.82	**27.92**	**16.91**	0.03	0.27	0.01	0.02	0.06	0.14	0.03	0.01	0.03	0.06	0.02	**12.08**	**99.90**
OS5A	**45.08**	1.80	**24.79**	**16.37**	0.02	0.38	0.01	0.02	2.15	0.19	0.04	0.01	0.04	0.06	0.00	**10.02**	**99.58**
OS5B	**72.60**	0.02	**18.50**	**0.63**	*<0.01*	0.05	*<0.01*	0.18	3.79	*<0.01*	*<0.01*	*<0.01*	*<0.01*	*<0.01*	*<0.01*	**4.12**	**99.90**
OS6A	**46.14**	0.47	**28.51**	**9.61**	0.03	0.17	0.01	0.02	3.26	0.06	0.01	0.01	0.02	0.03	0.00	**11.36**	**99.91**
OS7A	**49.73**	0.05	**33.10**	**0.87**	0.01	0.04	0.01	0.03	4.18	0.11	0.00	0.01	0.00	0.01	0.00	**11.76**	**99.97**
OS7B	**43.72**	1.08	**31.68**	**9.73**	0.01	0.15	0.01	0.02	0.29	0.08	0.04	0.01	0.03	0.04	0.00	**12.70**	**0.00**
OS08	**41.49**	1.07	**25.20**	**20.83**	0.03	0.13	0.01	0.02	1.03	0.11	0.02	0.00	0.04	0.07	0.00	**10.57**	**100.62**

Main oxides considered are in bold together with LOI.

TABLE 5.7: DETECTED MINERAL COMPOSITION OF SOILS (ADK AND OS)

	Kaolinite %	Muscovite %	Quartz %	Montmorillo-nite %	Geothite %	Heamatite %	Microcline %	Anatase %	Rutile %	Talc %
ADK 1A	**84.3**	**7.55**	**8.05**	0	**0**	**0**	0	0	0	0
ADK 1B	**67.82**	**6.55**	**11.5**	0	**0**	**3.35**	9.19	0	0	1.61
ADK 2	**91.6**	**2.46**	**5.93**	0	**0**	**0**	0	0	0	0
ADK 3	**77.22**	**3.89**	**8.1**	0	**6.96**	**2.73**	0	1.1	0	0
ADK 4	**70.34**	**10.85**	**9.67**	0	**3.49**	**3.42**	0	2.23	0	0
ADK 5	**76.35**	**6.66**	**8.35**	0	**0**	**0**	6.06	1.09	1.48	0
ADK 6	**54.74**	**0**	**39.65**	0	**0**	**2.11**	0	2.41	1.09	0
ADK 7	**66.39**	**0**	**20.2**	0	**12.15**	**0**	0	1.25	0	0
ADK 8	**78.33**	**0**	**9.21**	0	**4.45**	**4.49**	0	3.53	0	0
ADK 9	**80.58**	**0**	**7.03**	0	**6.3**	**6.09**	0	0	0	0
ADK 10	**91.66**	**0**	**3.13**	0	**3.04**	**2.17**	0	0	0	0
ADK 11	**67.17**	**0**	**17.15**	0	**6.37**	**6.24**	0	3.08	0	0
OS 1A	**51.44**	**30.85**	**16.95**	0	**4**	**0.77**	0	0	0	0
OS 1B	**71.11**	**16.91**	**10.37**	0	**0**	**1.61**	0	0	0	0
OS 2A	**89.03**	**4.53**	**3.29**	0	**0**	**3.16**	0	0	0	0
OS 2B	**40.96**	**3.35**	**36.62**	0	**0**	**0**	13.46	0	0	0
OS 3A	**73.62**	**2.75**	**7.05**	5.97	**7.16**	**3.45**	0	0	0	0
OS 3B	**76.87**	**0**	**9.03**	0	**10.55**	**3.55**	0	0	0	0
OS 4A	**85.25**	**7.39**	**5.91**	0	**1.46**	**0**	0	0	0	0
OS 4B	**83.45**	**0.83**	**5.94**	0	**8.73**	**1.05**	0	0	0	0
OS 5A	**63.62**	**9.36**	**14.98**	0	**8.74**	**1.96**	0	1.34	0	0
OS 5B	**11.66**	**27.72**	**60.61**	0	**0**	**0**	0	0	0	0
OS 6A	**70.97**	**3.7**	**6.23**	0	**4.33**	**0**	14.78	0	0	0
OS 7A	**68.26**	**5.6**	**2.05**	0	**0.76**	**0**	22.78	0	0.57	0
OS 7B	**88.32**	**2.8**	**6.46**	0	**0**	**2.43**	0	0	0	0
OS 8	**55.67**	**10.51**	**19.8**	0	**6.86**	**7.15**	0	0	0	0

Important minerals considered in bold.

TABLE 5.8: REGRESSION ANALYSIS BETWEEN ENGINEERING PROPERTIES AND BRICK STRENGTH

Variables	Correlation Coefficient	t-statistic	T-value
(Constant)	-19.966	-1.157	0.331
CBR@0% (X1)	0.094	1.638	0.200
CBR@5% (X2)	0.002	0.030	0.978
CBR@10% (X3)	-0.056	-1.046	0.373
0% OMC (X4)	6.228	1.596	0.209
0% MDD (X5)	-0.239**	-4.169	0.025
5% OMC (X6)	-10.755	-2.035	0.135
5% MDD (X7)	0.203**	4.723	0.018
10% OMC (X8)	4.664*	2.490	0.088
10% MDD (X9)	0.044	0.934	0.419
c (X10)	-0.042**	-4.201	0.025
¤(X11)	0.209*	2.984	0.058
	R = 0.990	R^2 =0.979	F = 13.013 (0.029)**

Note:

* significant at 10% and

** significant at 5%.

Dependant variable: Brick Strength

TABLE 5.9: REGRESSION ANALYSIS BETWEEN INDEX PROPERTIES AND BRICK STRENGTH

Variables	**Coefficient**	**t-statistic**	**p-value**
(Constant)	-32.955	-1.025	0.317
Amount of Fines (X1)	0.384	1.246	0.227
Amount of Coarse (X2)	0.391	1.232	0.232
Liquid Limit (X3)	0.012	0.145	0.886
Plasticity Index (X4)	0.200**	2.335	0.030
	R = 0.583	**R^2 =0.339**	**F = 2.569 (0.070)***

Note:
* significant at 10%
** significant at 5%.
Dependant variable: Brick strength

TABLE 5.10: REGRESSION ANALYSIS BETWEEN MAJOR OXIDES AND BRICK STRENGTH

Variables	Coefficient	t - statistic	p - value
Constant	79.790	2.024	0.056
SiO_2 (X1)	–.834*	–2.062	0.052
Al_2O_3 (X2)	–.188	–.296	0.770
Fe_2O_3 (X3)	–.690**	–1.566	0.032
LOI (X4)	–1.553**	–2.075	0.050
	R = 0.609	**R^2 =0.370**	**F = 3.089 (0.03)****

Note:

* significant at 10%

** significant at 5%.

Dependant variable: Brick strength

TABLE 5.11: REGRESSION ANALYSIS BETWEEN MINERAL COMPOSITION AND BRICK STRENGTH

Variables	Coefficient	t - statistic	p – value
Constant	21.503	3.028	0.007
Kaolinite (X5)	-.107	-1.390	0.180
Muscovite (X6)	0.038	.465	0.647
Quartz (X7)	-.214**	-2.413	0.026
Geothite (X8)	-.360**	-2.714	0.013
Heamatite (X9)	0.093	0.415	0.682
	R = 0.673	**R^2 =0.453**	**F = 3.312 (0.024)****

Note:

* significant at 10%

** significant at 5%

Dependant variable: Brick strength

5.6 CONCLUSION

As mentioned earlier, the focus of this chapter was to present a general overview of the results which forms the basis from which deductions in the next chapter are based.

Chapter 6

DISCUSSION

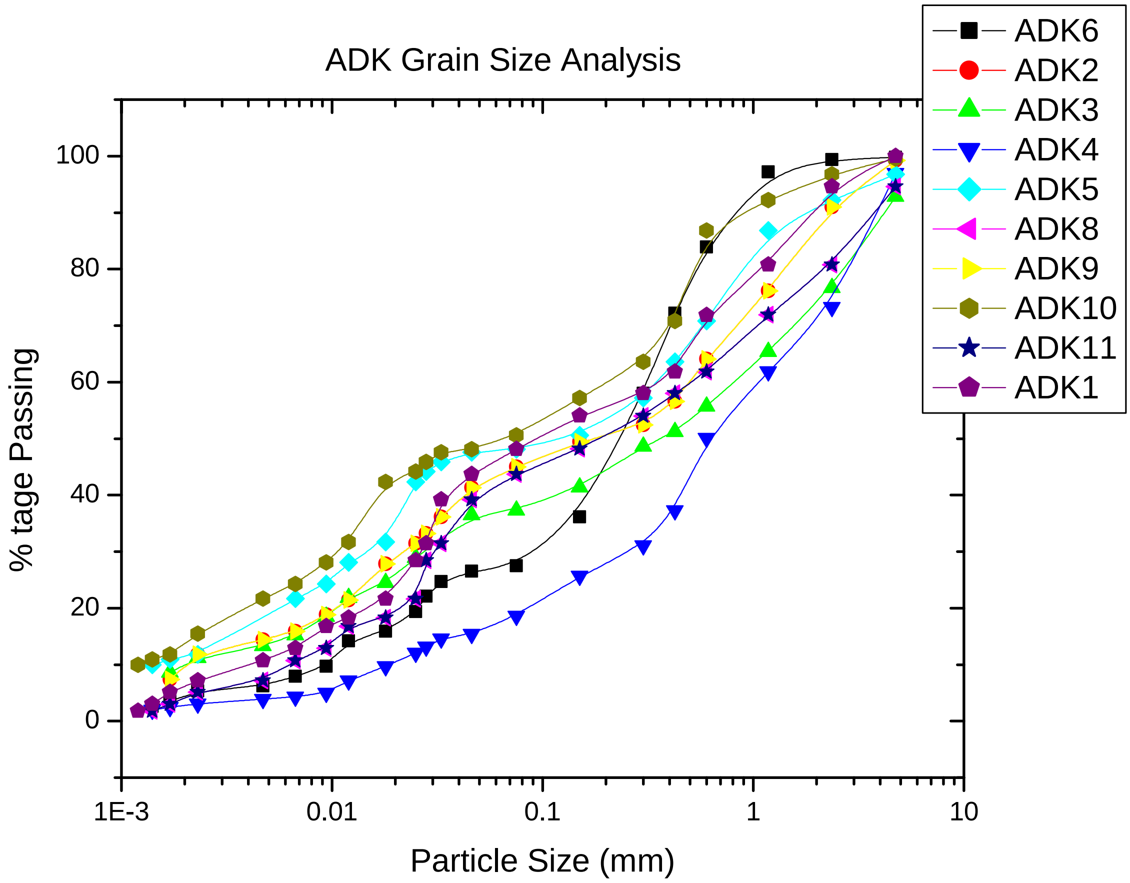
6.1 INTRODUCTION

One of the objectives of this research is to understand the overall influence of geology on tropical residual soil properties and by extension on the bricks made from these soils. In this chapter the results of the various test performed on the soils are discussed with a view of establishing the applicability of each test in the suitability or otherwise of tropical soils as material in brick making for sustainable housing construction in Africa.

6.2 GRAIN SIZE DISTRIBUTION

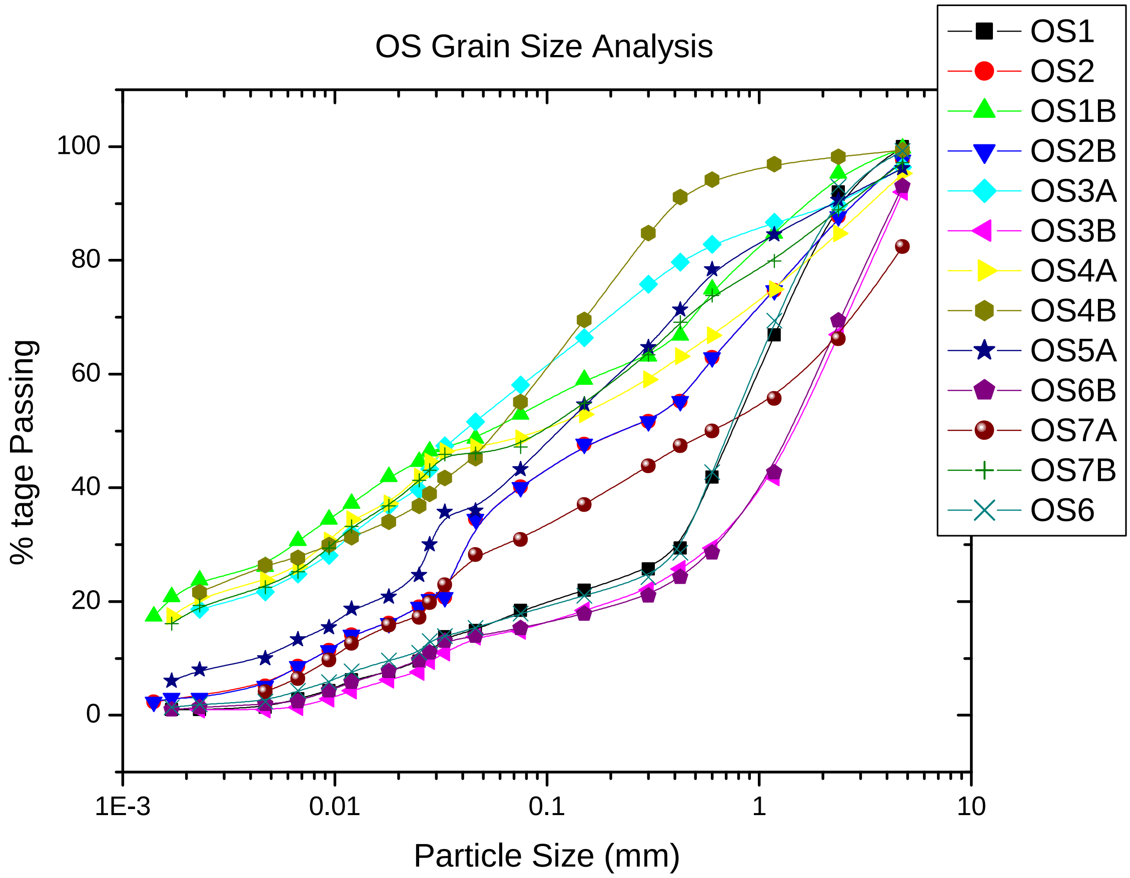
Grading characteristics of a soil form an important evaluation and classification parameter for soil materials in building construction (Oyelami and Van Rooy, 2016b). Soils from the study areas are well-graded and are expected to have a good compaction characteristic in terms of low porosity and low permeability. This is an indicator that the soils may possess good engineering properties. Presence of fine and coarse particles will naturally increase the efficiency of soil when compacted; clay particles tend to fill the void spaces between coarse portions of the soil at compaction. This is very relevant in the making of compressed earth bricks, because CEBs derive its compressive strength mainly by compaction.

The results presented in Tables 5.2 and 5.3 show a well-graded and competent soil in terms of texture at both sample locations which is evident in the particle size distribution curves presented in Figures 6.1 and 6.2. The amount of fines generally ranges between 14.48% and 49.54%, with an average for ADK at 37.3%, while that of OS is 36.6%. The coarse particle (sand and gravel) content ranges between 50.3 and 85.5%, with their averages at 61.6 and 62.1% for ADK and OS, respectively. This is considered good for various engineering constructions in line with already established literature (Adeyemi and Wahab, 2008; Adeyemi *et al.*, 2015; Giorgis *et al.*, 2014; Houben and Guillaud, 1994; Delgado and Guerrero, 2007; Oyediran and Okosun, 2013; Rigassi, 1995). Results from these locations are typical of soils derived from rocks that are felsic in origin. Soils from both locations reflect the mineralogical and textural characteristics of the parent rock (Adeyemi and Oyeyemi, 2000; Millogo *et al*., 2008b).



Source: Author's own (2016).

Figure 6.1: Grain size distribution of representative samples from ADK location



Source: Author's own (2016).

Figure 6.2: ***Grain size distribution of representative samples from OS location***

In addition, the grading characteristics of soils from the study areas reflect the influence of parent rock, which is visible in varying portions of the percentage of fine soil (clay and silt sizes). The amount of clay present in a soil is a function of weathering and the degree of laterisation.

In general, there is no unification in the different standards, each researcher recommends according to his region and peculiarity of the soils considered. Nevertheless, from the result of the present research, it could be concluded based on the earlier established literature, that a well-graded soil is required for the production of CEBs, and a minimum content of clay as suggested by Delgado and Guerrero (2007) is required. An adequate mix of sand and gravel has been found to be equally important. Judging from the aforementioned, the soil samples studied are adjudged suitable for the production of CEBs.

In order to further evaluate the characteristics of the soil, classification of soil was carried out based on two geotechnical systems of classification: AASHTO and USCS. Although no standard classification system has been adopted in earth construction materials, these were selected based on the earlier works of Houben and Guillaud (1994). It was found to be suitable for earth building classification, because it considers most of the salient parameters of the index properties of soil, namely the grain size distribution, the plasticity, cohesiveness, compactability and quality of soil (i.e. organic soil or otherwise).

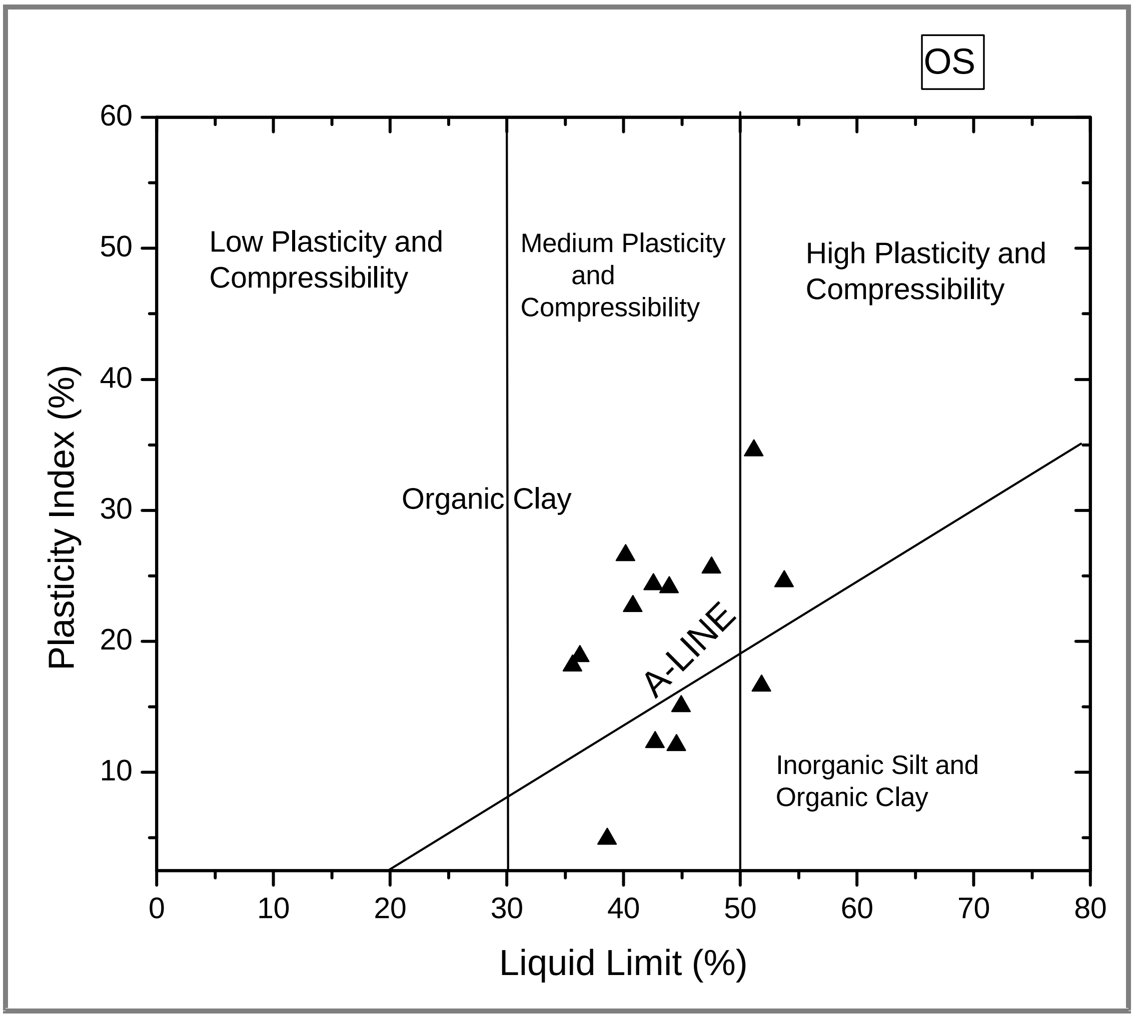
According to AASHTO classification system, the majority of the soil samples fall within the silty–clayey soils, between A6, A-7-5 and A-7-6 group, with a few that falls within the silty–clayey sands, between A-2-4, A-2-6 and A-2-7 groups. They are rated fair to good soils in geotechnical material rating. The USCS classification place soils within the silty sand (SM) and clayey sand (SC). Details of individual soil categories are found in Tables 5.2 and 5.3.

In terms of the influence of parent rock, micas which were present in the soils have structures similar to clay, thereby forming flat pseudo-hexagonal sheets that significantly influence the quality of aggregates (Loberg, 1999), and quartz-rich parent rocks which produce gravelly materials containing coarser quartz particles than the concretionary ones (Gidigasu, 1974).

6.3 CONSISTENCY LIMITS OF SOILS (MOISTURE CONTENTS)

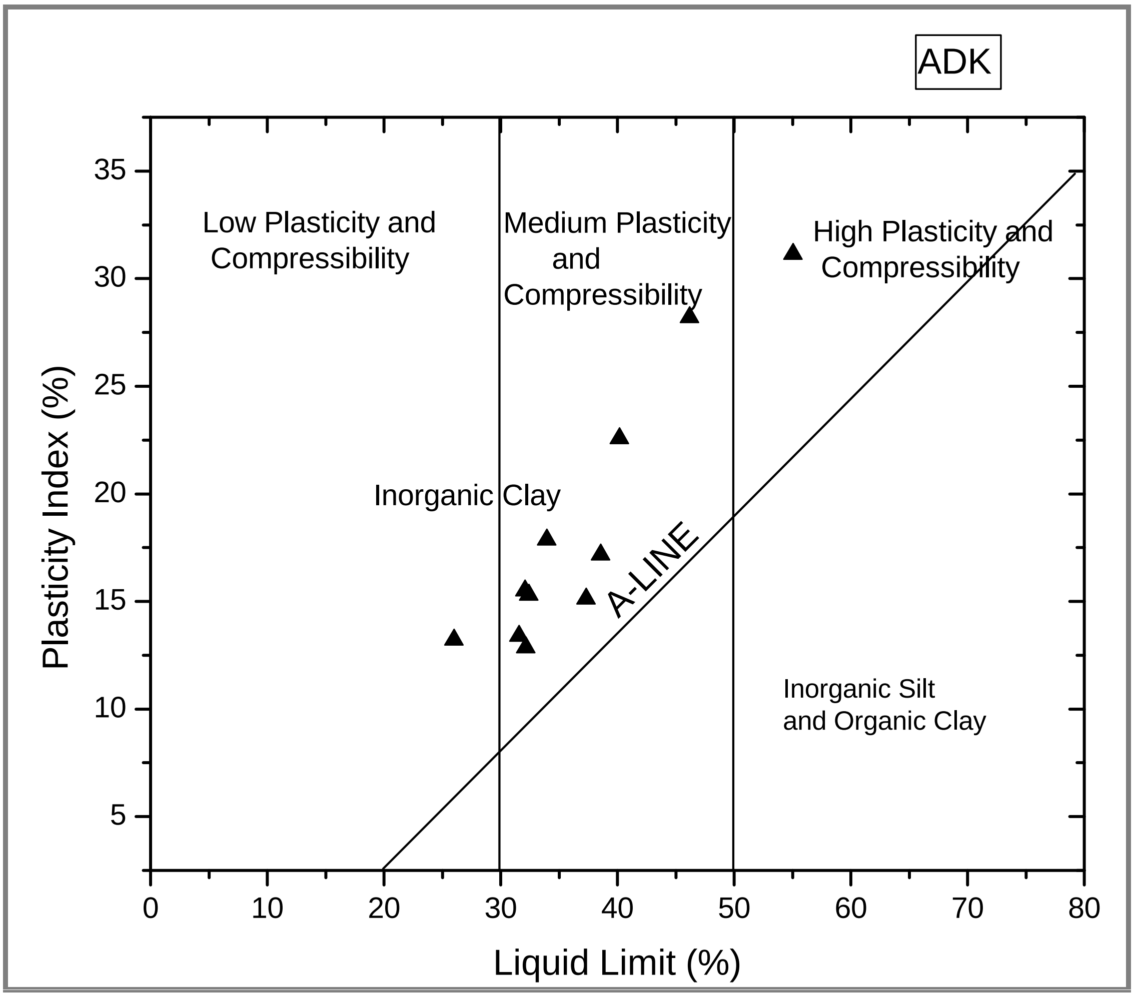
Consistency limits are particularly useful in describing, identifying, and classifying cohesive soils and serve as a tool in understanding their mechanical properties.

The soils from both locations have the following range of consistency: Liquid limit ranged between 26.01 and 55.04, plastic limit between 12.8 and 35.2, while plasticity index ranged from 4.9 to 34.56. Plasticity charts illustrated in Figures 6.3 and 6.4 show that most of the soil samples from both locations were inorganic clay with medium plasticity and compressibility. Basically, soil samples with liquid limits of <30% are considered to be of low plasticity and compressibility, those with liquid limits between 30% and 50% exhibit medium plasticity, while those with liquid limits of >50% exhibit high plasticity and compressibility. This shows that these soils are suitable for earth building as it was previously reported that most soils for earth building usually fall within the inorganic clay of medium plasticity, as well as inorganic silts of low and medium compressibility in the Casagrande chart of plasticity (Delgado and Guerrero, 2007). The results were in agreement with the earlier literatures especially with the proposed liquid limits and plasticity indices according to Houben and Guillaud, (1994), Adam and Agib, (2001) and Centre for Development of Industry (CDI), African Regional Organization for Standardization ARSO (1998).



Source: Author's own (2016).

Figure 6.3: Plasticity chart of soil samples from OS location



Source: Author's own (2016).

Figure 6.4: Plasticity chart of soil samples from ADK location

Based on the earlier established literature in section 2.3.8.2, (Table 2.1) the soils under study possess low to moderate swelling potentials, but again, this reflects the mineralogy of the soil based on the type of clay present. The statistical evaluation reflects a positive correlation between plasticity index and the angle of internal friction of the soils. This shows that there is a direct relationship between the PI and angle of internal friction. This is in line with the positions of earlier authors, where more clay in soil has been attributed to a higher plasticity leading to a greater potential shrinkage and swell, lower hydraulic conductivity, higher compressibility, higher cohesion and lower angle of internal friction (Mitchell and Soga, 2005; Rigo *et al*., 2006; Millogo *et al*., 2008a). Judging from the results of grain size analysis and the consistency of the soils studied, it could be suggested that up to 35% fines could be suitable for soils in CEB production, and to prevent excessive waste of stabiliser, a minimum amount of about 10% fines may be required for these soils.

6.4 SPECIFIC GRAVITY

The specific gravity of the lateritic soils from the two locations fall within a range of about 2.25–2.9, with an average specific gravity of 2.71 for ADK and 2.6 for OS. These results confirm that most of them are lateritic soils. The various values of SG are shown in Tables 5.2 and 5.3. The higher the specific gravity of a soil and the degree of laterisation, the stronger the soil would be (Adeyemi *et al.*, 2003*)*. Laterisation in form of sesquioxide coating around the study areas was found to be responsible for the increase in specific gravity and strength of soil, this confirmed that the higher specific gravity in the lateritic soils were responsible for their hardness, hence the higher strength observed in bricks. This is in agreement with earlier literatures that attributed to the presence of oxides with heavy minerals (for example, Fe_2O_3, FeO(OH)) with higher specific gravity and higher strength in soils (Gidigasu, 1974; Kasthurba et al., 2007).

6.5 COMPACTION CHARACTERISTICS AND CBR

Compaction is an important evaluation criterion in soils for brick making; this is because strength of the soil is positively related to brick strength. The present research established an improvement in compaction characteristics of soil with an increasing addition of cement. Nevertheless, at a certain point the improvement seems marginal. Compaction test at three percentages, 0%, 5% and 10% cement, gave optimum moisture content ranges of 12–23, 12.4–23, 13–24% and maximum dry densities of 1 476–1 910, 1 580–1930, 1 630–1 970.06 kg/m^3, respectively. The impact of cement increment is not reflected in the optimum moisture content as such, because water initially softens the soil particles and allow for free adjustment of particles. This effect on the particle increases the dry density until a point is reached whereby increase in water has no effect on the dry density, and further addition at this point leads to reduction in dry density of the soil material. As such the effect of water on cement may not be significant; it rather gives an idea of the quantity of water required in the production of CEB. The addition of cement, however, increases the maximum dry density of the soil.

Generally speaking, higher MDD observed signifies more densely packed, low permeable and higher strength soil materials. This is not significantly different from the findings of Bachar *et al*. (2014) who established variations in water content and maximum dry density as a function of cement content. Therefore, soil materials in brick making are required to have a significantly moderate MDD. There are no specified standards for compaction characteristics of soils for CEB, but there is a maximum density prescribed for CEBs which has to do with the density of the brick after compression. Nevertheless, an insight into to the compaction characteristics of soil for CEB making, gives a good idea of what to expect in the process of compression following the laboratory compaction which specifies the OMC and MDD that dictates the amount of water a particular soil type could accommodate in order to achieve a maximum density of bricks. According to established literature, a maximum of 2300 kg/m^3 has been recommended for CEBs, this implied that all the soil samples considered from both locations were suitable in terms of compaction for CEB brick production.

Compaction test results at 5% addition of cement was found to be significant at a 95% confidence limit (Table 5.8). This implies that a 5% addition of cement has a good relationship in attaining the required strength of CEBs. It is therefore recommended that a 5% cement addition may just be suitable for required compressive strength in CEBs.

The California bearing ratio test, though not quite popular as a soil evaluation test in brick making, was carried out to ascertain if there is any relationship or correlation between the test and either brick strength or soil strength parameters. CBR was carried out at 95 - 100% MDD and OMC with an increasing addition of cement at 0%, 5% and 10% at both unsoaked and soaked states of the soil. This was carried out to predict the response/behaviour of soil to ingress of water under soaking conditions.

The summary of the results from the test revealed a reduction in strength due to soaking and an improvement in the resistance of soil due to the increasing addition of cement. The unsoaked CBR result at 0%, 5% and 10% returned the following ranges: 14.88–50.30, 17.34–61.61 and 21.35–66.37, respectively, while soaked CBR at the same cement contents returned 6.62–14.21, 7.74–20.46 and 9.08–24.55, respectively. The reductions in strength due to soaking were given for each sample in Tables 5.4. and 5.5. It was obvious from the result that all the soil samples experienced a considerable reduction due to soaking; this could affect their performance when used as bricks in a building situated in a water-logged environment or an environment that is not well-drained. To mitigate this challenge, a good foundation structure should be recommended wherever this kind of soils were to be used, especially when the environment has a poor drainage system. In addition, a good drainage system is to be provided so as to minimise long time exposure of bricks made from this kind of soil to water.

Another observation from the results reflected a better performance in soils from the OS location after adding cement. They offer better resistance to load (plunger) at 5% and 10% cement content. Judging from these results, this test could be suitable for soils in earth brick production in predicting the following: the possible content of cement in soil that offers best resistance to load, the likely response of bricks to increasing moisture content, especially in a water logged environment, and an idea of the response of the soil to stabilisation. CBR, as established by statistical tests, may not have a direct impact on brick strength, but it has been shown to be valuable in the evaluation of soils to be used as materials in brick making.

6.6 STRENGTH OF SOILS

A summary of the shear strength parameters revealed moderately high values of cohesion which may be due to the general percentage of fines, and hence confirmed the soil samples as partly cohesive. The shear strength parameters from the tested soils from ADK show cohesion values between 29 and 150 kPa and from the OS site the cohesion falls between 46 and 114 kPa. Corresponding internal friction angles for both sites falls between 24^{o} and 42^{o} while the shear strength (τ) of soil range from 63.30 to 173.70kPa and 69.06 to 148.30 for ADK and OS respectively. A critical look at the result reflects the impact of the percentage of fines on the cohesion within some locations. This could be attributed to the influence of parent rock on the strength parameters of the soil, the presence of coarse low-friction platy minerals around the Osogbo axis of the study area may be responsible for the partly low shear strength and cohesion observed from their results as presented in Table 5.4.

Adequate attention has not been given to the role of strength parameters of soil materials (c and ϕ) in the strength and durability of earth bricks. The results of this present study have been able to indicate the significance of these parameters on the strength of bricks. This is established by the result of the T-test carried out, which shows that cohesion of soil is significant at a 95% interval, while the angle of internal friction was found significant at a 90% level of interval (Table 5.8). This reflects the role of clay as binding agent in the material and sand material as a useful and indispensable constituent as well.

An adequate mix of coarse and fine material is more important in soil selection for brick making than in any other construction materials. This is because fine materials play an initial role in the binding process and aid the compaction properties of a soil; coarse particles are responsible for the long-term durability of the brick and they react easily with cement to form a lasting cohesion (Muntohar, 2011). Adequate combination of clay and sand naturally complement each other by clay/finer materials occupying void spaces present in sand, leading to reduction in permeability, water absorption, more densely packed soil and greater shear strength parameters. Strength of soils, especially tropical soil, is equally dependent on their mineralogy and composition.

6.7 IMPACT OF SOIL MINERALOGY ON BRICK STRENGTH

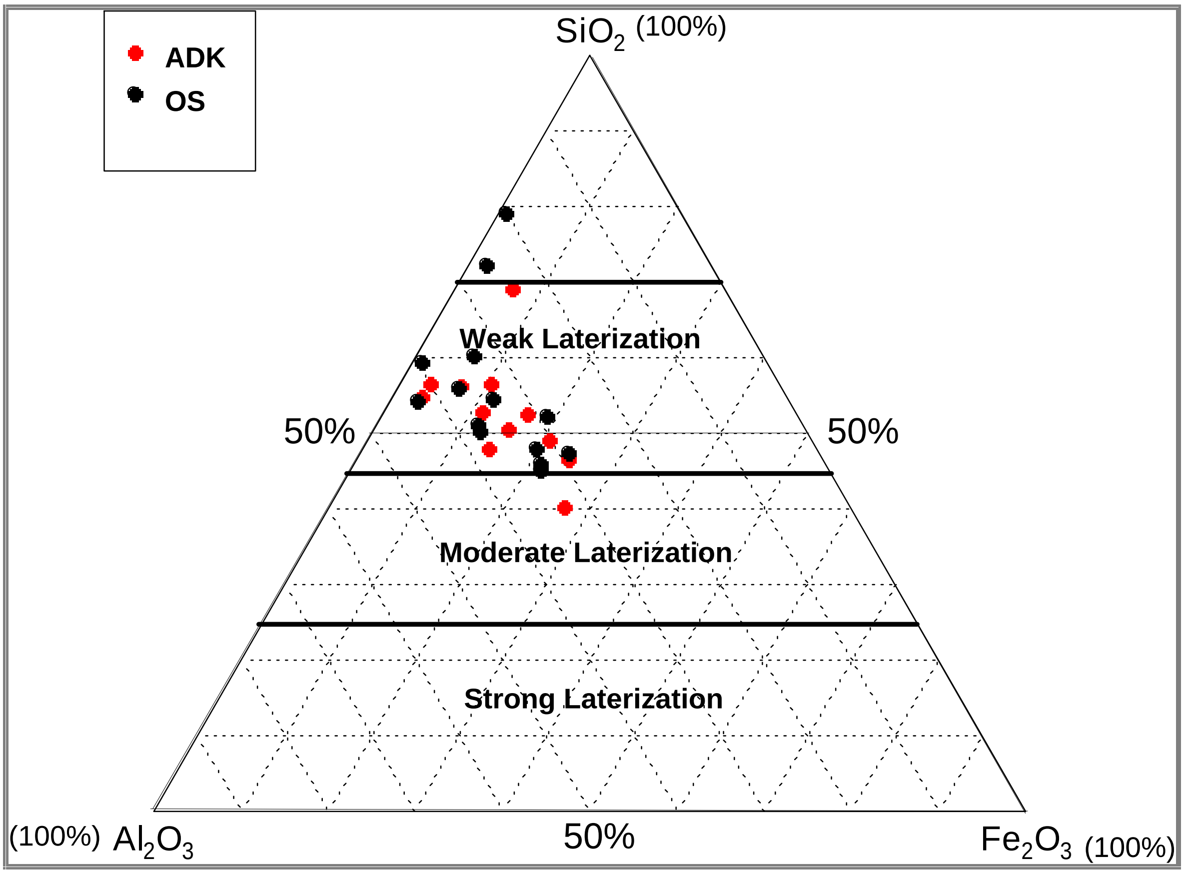
Millogo *et al.* (2008b) recommended soil mineralogy as an important evaluation for lateritic soil, because of their changing characteristics with time which had made it difficult to rely only on their geotechnical and mechanical evaluations. Tables 5.6 and 5.7 in the previous chapter give the details of the mineral composition and chemistry of the soils studied. The summary of the relevant minerals and oxides that are important in brick strength are listed below:

- Kaolinite $Al_4(Si_4O_{10})OH_8$
- Quartz SiO_2
- Muscovite $KAl_3Si_3O_{10}(OH)_2$
- Goethite FeO(OH)
- Heamatite Fe_2O_3
- Microcline $KAlSi_3O_8$
- Montmorillonite $(Na,Ca)0.3(Al,Mg)_2Si_4O_{10}(OH)2.n(H20)$
- Anatase TiO_2
- Rutile $(Ti,Fe)O_2$

In terms of major oxides, SiO_2, Al_2O_3, Fe_2O_3 and others, were present. Soil mineralogy in the present study partly reflects the composition of the underlying rocks, and their variations mostly depends on the type and genesis of the weathered regolith (Tijani *et al.*, 2006). Abundance of kaolinite in the soil samples confirms their lateritic nature and reflects their degree of laterisation. This was confirmed based on the ternary plot shown in Figure 6.5, as proposed by Schellmann (1981), where most of the soil samples fall within the region of weak laterisation. Major oxide proportions are in the range of 34.18–72.60% SiO_2, 18.50–35.76% Al_2O_3, 0.63–22.8% Fe_2O_3, with 4.12–14.16 loss on ignition (LOI).

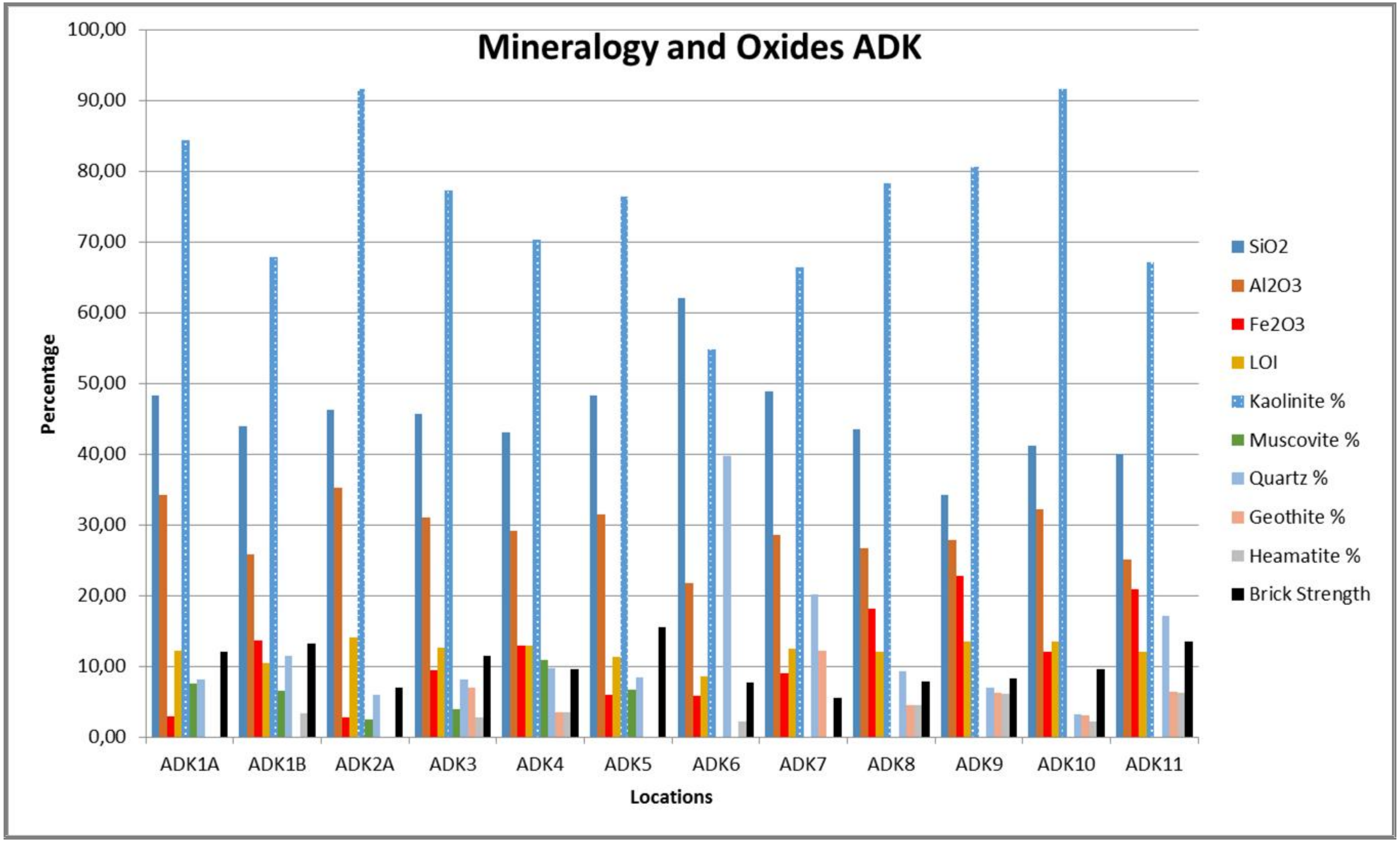
Laterisation, among other factors, are characterised by chemical weathering with progressing loss of mobile elements in the form of alkalis (for example K_2O, MgO, CaO) and gradual enrichment of sesquioxides (for example Fe_2O_3, Al_2O_3, FeO) as proposed by earlier authors (Tijani *et al.*, 2006). These sesquioxides plays a significant role in the hardening of laterites. Induration in lateritic soils on exposure to the atmosphere has been attributed to these sesquioxides which is often reflected in their typical profile with the presence of sesquioxides in the upper part and kaolinites occurring at the lower part of the profile (Kasthurba *et al.*, 2007, 2008). The relevance of this was studied in the present research, especially as it affects the strength of bricks made from lateritic soil. The results revealed an important role played by sesquioxides, not just in soil strength as proposed by Kasthurba *et al.* (2008), but more importantly in the strength of bricks.

As a follow-up to the visual comparison presented in the histogram (Figure 6.6), a multivariate regression analysis was carried out which revealed a positive correlation between the sesquioxides and the brick strength (Table 5.10). More importantly, the T-test revealed an influence of sesquioxides, particularly Fe_2O_3 and SiO_2, on the brick strength at 90 and 95% confidence limit (Table 5.10). This is in agreement with the findings of Gidigasu (1974), Kasthurba *et al.* (2008) and Maegnein (1966) that engineering properties of lateritic soils depend on its degree of laterisation, chemical, mineralogical composition and its induration on the quantity and nature of iron present in it. Goethite and heamatite were present as hydroxide and oxide of iron in the present study and they have been found to contribute more to the strength of the bricks because of their higher specific gravities and hardness. Quartz (SiO_2), being a resistant mineral and a natural binding agent, also contributed to the strength parameter of the bricks.



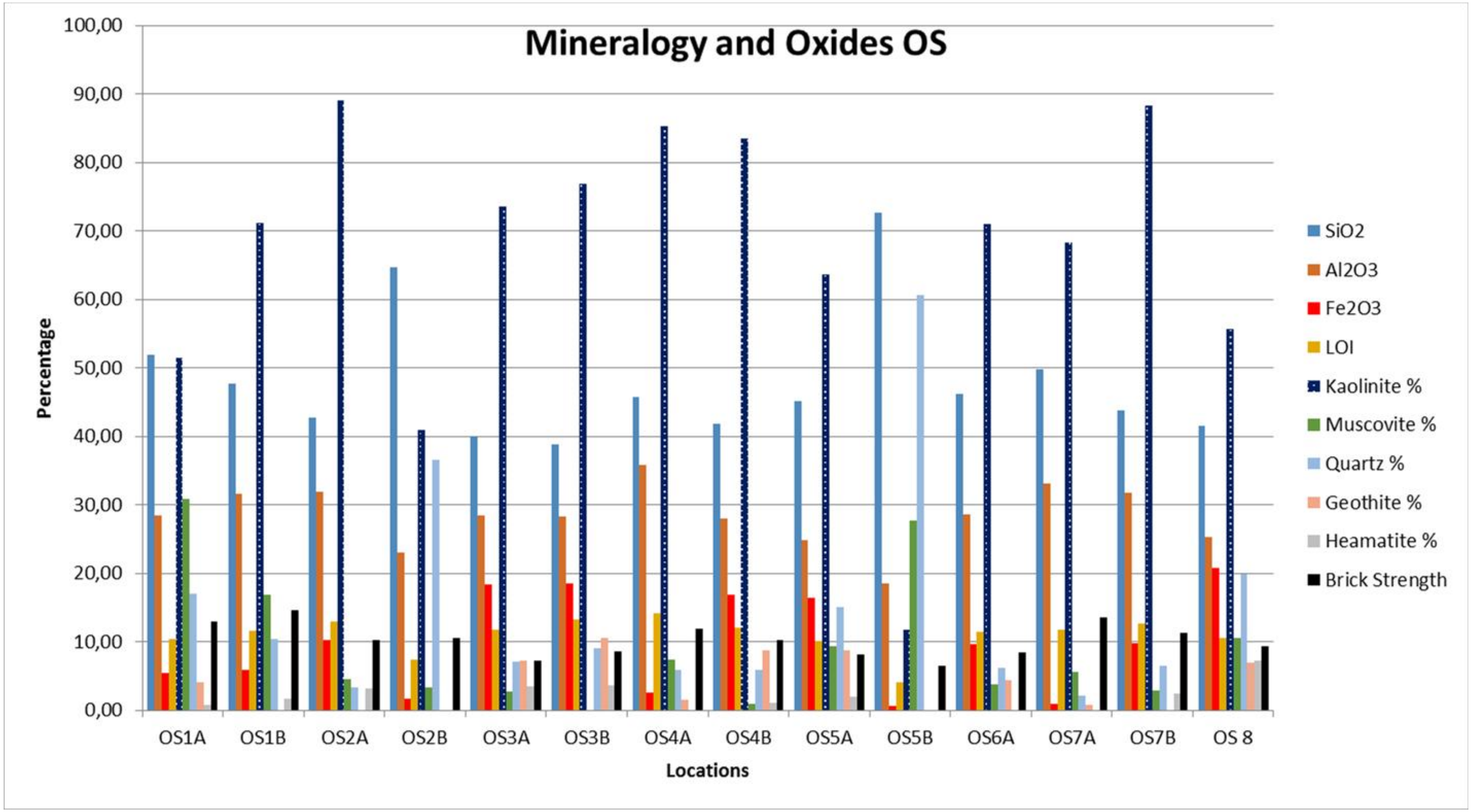
Source: Modified after Schellman (1981)

Figure 6.5: ***A ternary plot showing the degree of laterisation***



Source: Author's own (2016).

Figure 6.6: Histogram showing the relationship between oxides, minerals and brick strength for ADK location



Source: Author's own (2016).

Figure 6.7: Histogram showing the relationship between oxides, minerals and brick strength for OS location

The importance of soil mineralogy in brick making cannot be over-emphasised, because most of the strength parameters and tests derived in the laboratory related only to the strength of the soil which has little to do with the durability of the soil in the long process. Mineralogy goes a step further in the prediction of the behaviour of soil and its sustainability based on the nature and kind of minerals and/or major oxides present. Presently, there are no standards that specifies the minimum requirements in terms of either mineralogy or major oxide in soils for earth construction; therefore, this present research can only recommend based on the findings peculiar to the location where the work is carried out. It is worthy to note that lateritic soils are heterogeneous and their properties vary even within the shortest distance (Kasthurba *et al*., 2007; Malomo *et al*., 1983), therefore, a general statement or conclusion cannot be made for basic standard of soil mineralogy to be adopted in brick making.

6.8 DURABILITY OF COMPRESSED EARTH BRICKS

Durability of earth materials can best be estimated in terms of resistance to extreme weather conditions in the natural state. These conditions include heavy rainfall, strong and mild winds, frost and extreme heat and cold conditions. Houben and Guillaud (1994) recommended a practical performance evaluation of durability by exposing bricks to varying weather conditions at least for a complete season. Generally speaking, the durability of bricks produced from lateritic soil could equally depend on the silica/sesquioxide ratio as proposed by Millogo *et al.* (2008b). This is because of the resistance offered by silica to weathering, coupled by the hardness contributed by sesquioxides.

The report of the durability test carried out on the bricks reflected the influence and impact of mineralogy on the bricks after a complete cycle of climatic conditions over a period of about 18 months of heavy rain fall, intense heat and a series of strong winds. Figure 6.8 shows the picture of the bricks at different periods. The exposed bricks were further evaluated in terms of their physical change and change in weight. Dry compressive strength and weight of the bricks at the initial stage, compared to their weight after being exposed for 18 months, is presented in Table 6.1 below. Figure 6.8 below shows the brick pile after 28 days after both moisture and air curing and at six months and after one year.

TABLE 6.1: DRY COMPRESSIVE STRENGTH ON BRICKS AND CHANGE IN WEIGHT AFTER 18 MONTHS

Sample No	Weight (kg)	Density (g/cm^3)	Load@Failure (KN)	Strength (Mpa)	% cement	Drop Test	Weight after 18 months	% Weight loss
ADK 1a	7.94	1.72	50.7	12.07	5	Smashed	7.45	6.1
ADK 1b	8.04	1.74	55.5	13.21	5	Smashed	7.68	4.5
ADK 2a	6.42	1.4	26.6	6.33	5	Smashed	5.32	17.1
ADK 2b	6.47	1.39	30.3	7.21	5	Smashed	5.64	12.8
ADK 3a	7.69	1.66	48.3	11.5	5	Slightly smashed	7.42	3.5
ADK 3b	7.27	1.57	43.2	10.29	5	Slightly smashed	6.86	5.6
ADK 4a	7.41	1.6	36.3	8.64	5	Slightly smashed	6.95	6.2
ADK 4b	7.61	1.65	40.4	9.62	5	Slightly smashed	7.12	6.4
ADK 5a	7.20	1.56	65.4	15.57	10	Not smashed	6.96	3.3
ADK 5b	7.32	1.58	53.6	12.76	10	Not smashed	6.89	5.9
ADK 6a	7.21	1.56	29.9	7.12	5	Smashed	6.74	6.5
ADK 6b	7.80	1.69	32	7.62	5	Smashed	7.26	6.9
ADK 7a	6.63	1.44	23	5.48	5	Not smashed	6.02	9.2
ADK 7b	6.40	1.39	22.4	5.33	5	Not smashed	5.6	12.5
ADK 8a	6.74	1.46	26.9	6.4	5	Not smashed	6.22	7.7
ADK 8b	6.87	1.49	33	7.86	5	Not smashed	6.25	9
OS 1a	8.56	1.85	54.4	12.95	5	Smashed	7.99	6.7

OS 1b	8.34	1.81	61.5	14.64	5	Smashed	7.85	5.9
OS 2a	7.27	1.57	42.8	10.19	5	Not smashed	6.46	11.1
OS 2b	7.08	1.53	44	10.48	5	Not smashed	6.32	10.7
OS 3a	7.17	1.55	30.1	7.17	5	Not smashed	6.45	10
OS 3b	7.12	1.54	36.3	8.64	5	Not smashed	6.51	8.6
OS 4a	7.02	1.52	50	11.9	10	Not smashed	6.23	11.2
OS 4b	6.91	1.5	43.2	10.29	10	Not smashed	6.08	12
OS 5a	7.25	1.57	34.2	8.14	5	Not smashed	6.49	10.5
OS 5b	7.15	1.55	27.3	6.54	5	Not smashed	6.24	10.2
OS 6a	7.14	1.55	35.4	8.43	5	Not smashed	6.39	10.5
OS 6b	7.07	1.53	39	9.29	5	Not smashed	6.33	10.5
OS 7a	7.39	1.6	56.7	13.5	10	Not smashed	6.57	11
OS 7b	7.39	1.6	47.3	11.26	10	Not smashed	6.69	9.47

Source: Author's own (2016).

Bricks after 28-day curing, exposed to test for its durability against intense weather, erosion and wind

After six months of exposure to intense weather, rainfall and wind

After more than one year of exposure to intense weather

Source: Author's own (2016).

Figure 6.8: Compressed earth bricks at different periods after exposure to weather.

6.9 CONCLUSION

After careful evaluation of the various results and discussion, the summary of the statistical evaluation of the different parameters varying from the index, engineering and mineralogical parameters tested against the brick strength with the aim of identifying their influence or significance on the strength of bricks is given below:

TABLE 6.2: SUMMARY OF STATISTICAL EVALUATION OF DIFFERENT PARAMETERS AGAINST BRICK STRENGTH

Parameter	Significance	Level of Significance and Value
Amount of Fines	Not Significant	0.317
Amount of Coarse	Not Significant	0.227
Liquid Limit	Not Significant	0.232
Plasticity Index	Significant	0.03 (95%)
Index Parameters (F-value)	**Significant**	**0.07 (90%)**
0% MDD	Significant	0.025 (95%)
5% MDD	Significant	0.018 (95%)
10% MDD	Not Significant	0.4
0% OMC	Not Significant	0.2
5% OMC	Not Significant	0.14
10% OMC	Significant	0.08 (90%)
Cohesion, c	Significant	0.025 (95%)
Internal Friction φ	Significant	0.058 (90%)
Engineering Parameters (F-value)	**Significant**	**0.029 (95%)**
SiO_2	Significant	0.056 (90%)
Al_2O_3	Not Significant	0.770
Fe_2O_3	Significant	0.03 (95%)
LOI	Significant	0.05 (95%)
Major Oxides (F-value)	Significant	0.03 (95%)
Kaolinite	Not Significant	0.180
Muscovite	Not Significant	0.647
Quartz	Significant	0.026 (95%)
Goethite	Significant	0.013 (95%)
Haematite	Not Significant	0.682
Minerals (F-Value)	**Significant**	**0.024 (95%)**

Chapter 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 CONCLUSIONS

The following conclusions were drawn after a careful study of the results and based on some earlier literature:

- Lateritic soils have proved to be a suitable material in the making of compressed earth bricks. Despite the difference in location and the underlying geology, soil samples from the two locations were found suitable for CEB. Based on the index properties of the lateritic soils studied, it is suggested that up to 35% fines could be suitable for soils in CEB production, and to prevent excessive waste of stabiliser, a minimum amount of about 10% fines is required for these soils. Again, the type of soil and the geographic location may play a significant role in their index properties. The above suggestion was based on the present study on tropical lateritic soil.

- The importance of parent rock on the grain size characteristics of residual lateritic soils have been established, they were found to have a significant influence on the mineralogy of the resultant soil, which play an important role in strength and durability of bricks. Products of rocks from the study area are well-graded lateritic soils with good compaction characteristics, low porosity and permeability which make them excellent material in brick making.

- Lateritic soils are readily available even in its natural form as material for brick making, and from the present study it has been established that with even a minimal addition of cement (about 5%) a strong brick of an average strength of 7 MPa can be produced. This is suitable for the required compressive strength in CEBs for low rise or bungalow residential buildings, single-storey buildings based on the recommended values of 2.8 and 5.2 MN/m^2 for precast concrete blocks and load bearing fired clay bricks, respectively (BS 771-3, 1990).

- Specific gravity of tropical soils may serve as an important clue of their strength and durability since specific gravity is influenced mainly by the types of minerals and oxides present in these soils.
- CBR tests could also be relevant for soils in earth brick production because it gives a useful clue to the possible cement content required in soil that offers best resistance to load; to foretell the response of bricks to increasing moisture content, especially due to ingress of water; and to give an idea of the response of the soil to stabilisation.
- Compaction characteristics of lateritic soils in CEB production serves as a pointer to what is expected in the process of compression, i.e. the handability of bricks in the mould and the ease of extruding bricks after compression. It dictates the amount of water a particular soil type could accommodate in order to achieve a maximum density in bricks.
- The importance of the shear strength parameters of soil (c and ϕ) cannot be overemphasised in achieving the best strength quality and durability of bricks. It reveals the pivotal role of clay materials as binding agent and sand as a useful and indispensable constituent in achieving long-lasting strength quality of CEB.
- The mineralogy of laterites was found to play a significant role in the cohesion and strength of bricks, and therefore, the resultant bricks were able to stand the test of time over a long period in unfavourable weather and climatic conditions. It is strongly believed with this experiment that they could withstand even more adverse climatic conditions when they are properly joined with cement mortar.
- Mineralogy goes a step further in the prediction of the behaviour of soil and its sustainability based on the nature and kind of minerals and/or major oxides present.
- An adequate mix of coarse and fine material is more important in soil selection for brick making than in any other construction materials. This is because fine materials play an initial role in the binding process and aid the compaction properties of a soil; coarse particles are responsible for the long-term durability of the brick and they react easily with cement to form a lasting cohesion.
- Finally, for the sake of the environment, firing of clay bricks should be discouraged as much as possible since adequate stabilization methods could be employed in

achieving any required strength of bricks for various construction purposes. Equally, the use of fibres have proven efficient in improving strength and durability of bricks.

7.2 RECOMMENDATIONS

One of the main challenges in the production of CEBs is the measurement and mixing of cement and soil. A guide is provided that also simplifies the conversion from weight measurement to volume which will go a long way in helping professionals and the man on the street or in the field on how to achieve a proper mixture for soil/cement stabilised brick production.

7.2.1 Proposed soil/cement measurement and mixing based on weight/volume

The proposed method of achieving either 5% or 10% stabilisation is based on the extensive fieldwork and laboratory analyses, coupled with a brick making experimentation, during this research project. This is concerned with understanding the proper way of measuring proportions of soil and cement to achieve the required stabilisation. The man on the street is used to a volumetric measurement of materials in brick making; therefore, based on this research, the following were deduced to produce an efficient brick based on volumetric measurement:

A standard bag of cement weighs 50 kg and a Standard builders' wheelbarrow is equivalent to 65 litres. Based on calculations using density and specific gravity, 50 kg of cement is approximately equivalent to about 33 litres. Therefore, 1 000 kg of sand is equivalent to 660 litres = 10 wheelbarrows of soil. Therefore, to produce approximately 90 earth bricks at 5% cement stabilization, the following volumes of ingredients are needed:

1. One bag of cement (standard bag = 50 kg)

2. 10 wheelbarrows of sand (equivalent to 1000kg of sand)

For 10% cement stabilization to produce approximately 50 earth bricks:

1. One bag of cement

2. Five wheelbarrows of sand (equivalent to 500kg of sand)

7.2.2 Other recommendations

- It is recommended that governments, especially in African countries, should make adequate regulations/specifications on the proper use of earth materials. This will promote a greener environment and bridge the existing deficit in housing around the continent.
- As attention is beginning to shift from the sandcrete/concrete blocks to earth bricks, because of the energy consumption of the former as compared to the latter, it is necessary to do further research into the possibility of further reducing the amount of cement in brick making by exploring the possibility of geomaterials, for instance, tar sand or bitumen and other possible fibre products.

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APPENDIX A

EQUATIONS AND FORMULAS

FORMULAE

A-1 Plasticity Index

PI = L.L – P. L

Where PI= Plasticity Index

LL= Liquid Limit,

PL= Plastic Limit.

A-2 Uniformity Coefficient

Cu= D60/D10

A-3 Moisture Content

w= **[**$(W_2$-$W_3)$ – $(W_3$-$W_1)$] X 100

Where w= Moisture Content

W_1= Mass of Can (g)

W_2= Mass of can + wet soil (g)

W_3=Mass of can + Dry soil (g)

W_2-W_3= Mass of mass moisture (g)

W_3-W_1= Mass of Dry Soil (g)

A-4 Specific Gravity

$G_s = M_2 - M_1 / (M_4 - M_1) - (M_3 - M_2)$

Where G_s= Specific Gravity

M_1=Mass of Dry pycnometer

M_2= Mass of Pycnometer + dry soil

M_3= Mass of pycnometer + soil + water

M_4= Mass of pycnometer + water

A-5 Linear Shrinkage

$\mathbf{S_L} = [1 - (L_1/L_0) \times 100]$

Where S_L = Linear Shrinkage

L_0 = Initial length of specimen (mm)

L_1 = Length of oven dried specimen (mm)

A-6 Activity

A = PI / %Clay fraction

Where A= Activity

PI= Plasticity Index

A-7 CALIFORNIA BEARING RATIO (CBR)

(Each unit of load * 30) / 1000 = Load in kN

CBR = Penetration of plunger at 2.5mm*(100/13.24)

= Penetration of plunger at 5.0mm*(100/19.96)

Strength Reduction

%Strength Reduction = (Unsoaked CBR – Soaked CBR/ Unsoaked CBR) X 100

CBR= California Bearing Ratio

Increase in Moisture Content

% Increase in Moisture Content = (Soaked M.C – OMC) / OMC

MC- Moisture Content, OMC- Optimum Moisture Content

A-8 SHEAR STRENGTH

Unit strain = deformation/initial length (l_o)

l_o=76mm

Area cylinder (A_o) =$\pi d^2 h/4$

D= Diammeter of Mould=38mm

Corrected area (Ac) =A_O/ (1-ϵ) 10^{-3}

ϵ = strain

Stress = Load/Corrected Area

Shear Strength, $\tau = c + (\sigma_n - u) \tan \phi$

A-9 HYDROMETER TEST

t = elapsed time

r = Hydrometer reading

R =1000(r-1)

Rw = -2.5

N % = $K_1(R-R_W)$

$K_1 = 2G_S/G_S$-1

K_2= 0.012

K_1= (2*2.66)/ (2.66-1) =3.20

h =submerged hydrometer length

Zr= 5.07+h

D =particle size

= k_2* SORT (Z_r/t)

N^I = % finer

= % passing 0.063mm* N^1%

A-10 Permeability Test

k= (2.3026 /A) * ($\log_{10}H_1$-$\log_{10}H_2$) / (t_2-t_1)

Where k= coefficient of permeability (mm/secs)

A= Bore area of standpipe tube

L=length of sample under test (mm)

H_1= Initial head of water (mm)

H_2= Head of water (mm) indicated at end of a particular period of time

t_1= Initial time

t_2= Time corresponding to water (sec)

A-11 Coefficient of Variation

C_v = [S.D /(∑x/n)] X 100

S.D= $[(x- x)^2/ n]^{1/2}$

Where C_v= Coefficient of variation

S.D= Standard Deviation

X= Statistical mean from data

n= Number of events or trials

A-12 T-test and f-test

$$t = \frac{\bar{x}_1 - \bar{x}_2}{\sqrt{\frac{S_1^2}{n_1} + \frac{S_2^2}{n_2}}}$$

Where,

$\bar{x}_1$= Mean of first set of values

$\bar{x}_2$= Mean of second set of values

S_1 = Standard deviation of first set of values

S_2 = Standard deviation of second set of values

n_1 = Total number of values in first set

n_2 = Total number of values in second set.

Variance is given as:

$$S = \sqrt{\frac{\sum(x - \bar{x})^2}{n - 1}}$$

Where,

x = Given values

$\bar{x}$ = Mean

n = Total number of values.

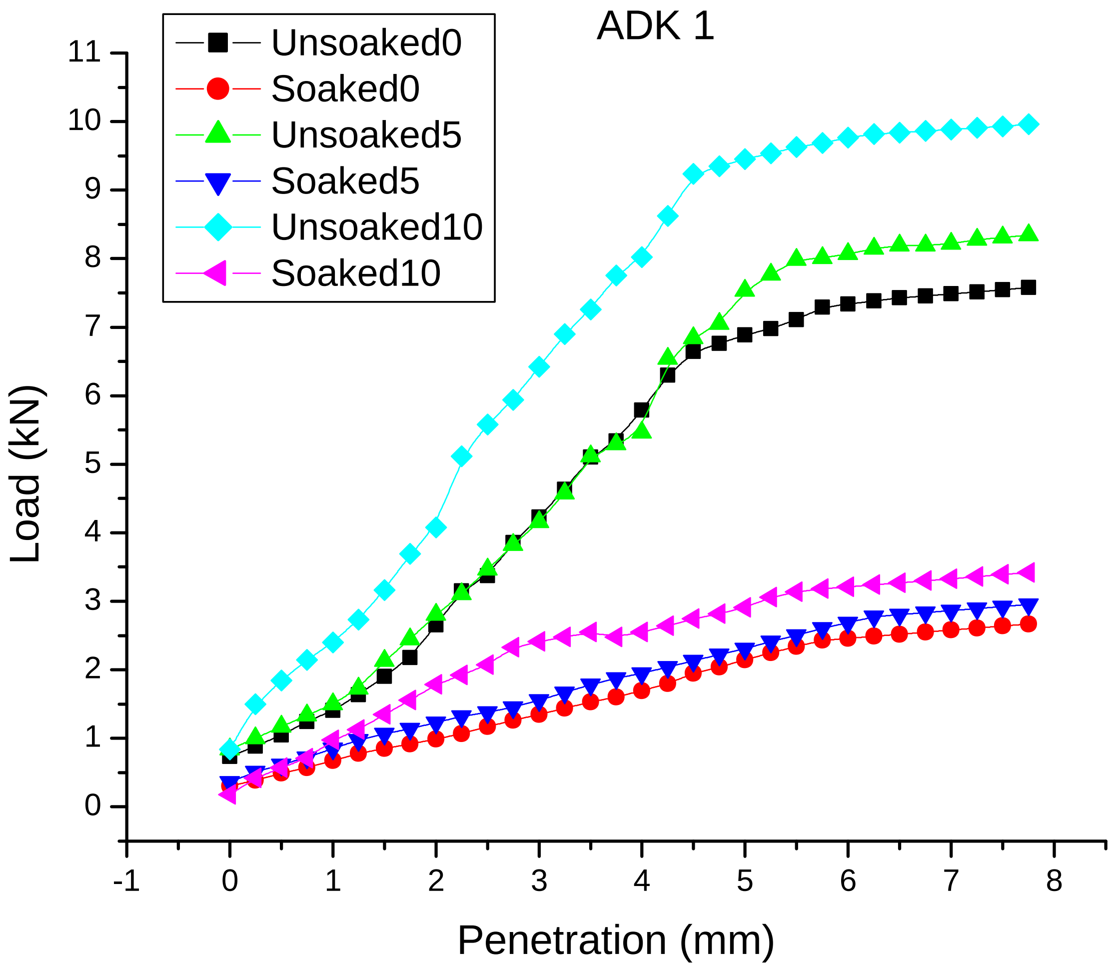
Therefore, f-distribution is calculated as:

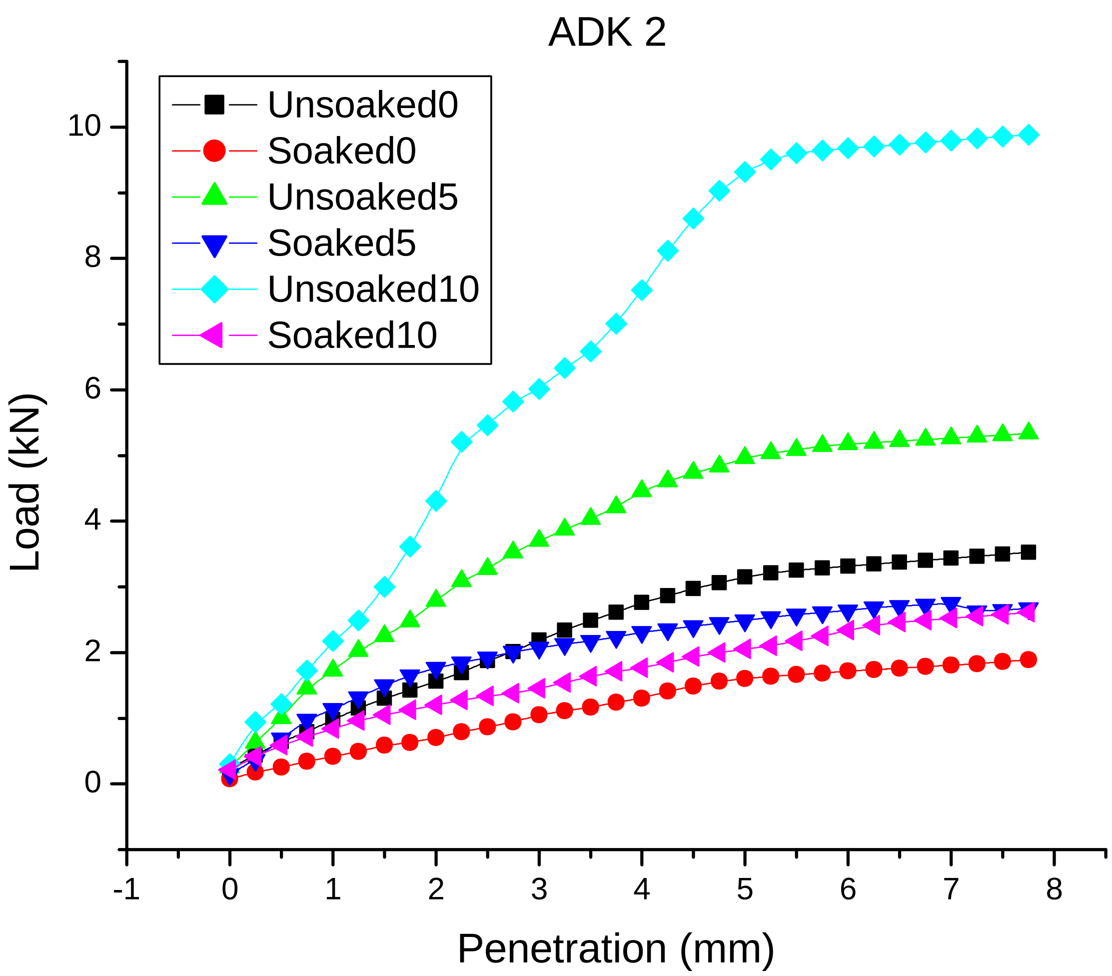
$$F\ value = \frac{variance\ 1}{variance\ 2} = \frac{\sigma_1^2}{\sigma_2^2}$$

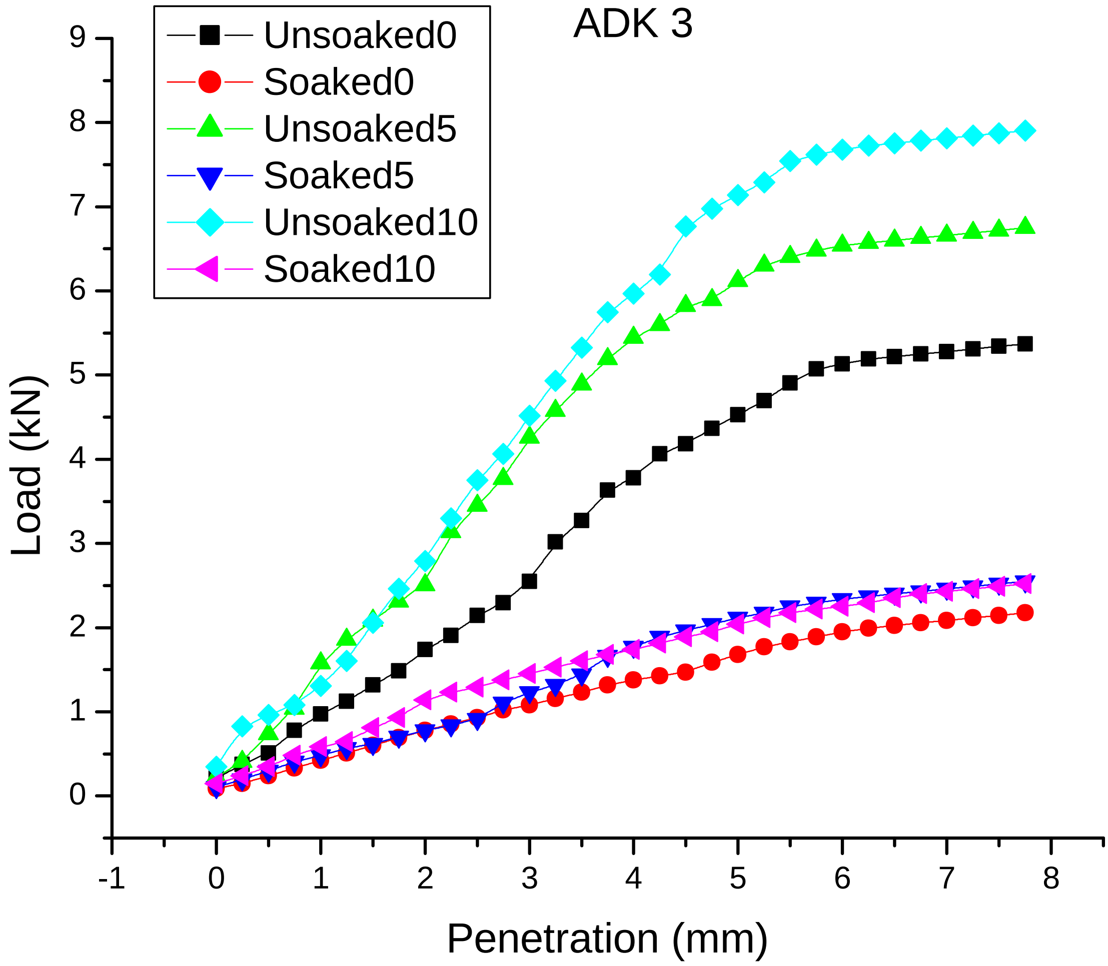
APPENDIX B

GRAPHS AND TABLES

B-1 California Bearing Ratio (CBR) Plots




ADK 2
Unsoaked0
Soaked0
Unsoaked5
Soaked5
Unsoaked10
Soaked10
Load (kN)
Penetration (mm)
10
8
6
4
2
0
-1
0
1
2
3
4
5
6
7
8


ADK 3
Unsoaked0
Soaked0
Unsoaked5
Soaked5
Unsoaked10
Soaked10
Load (kN)
Penetration (mm)
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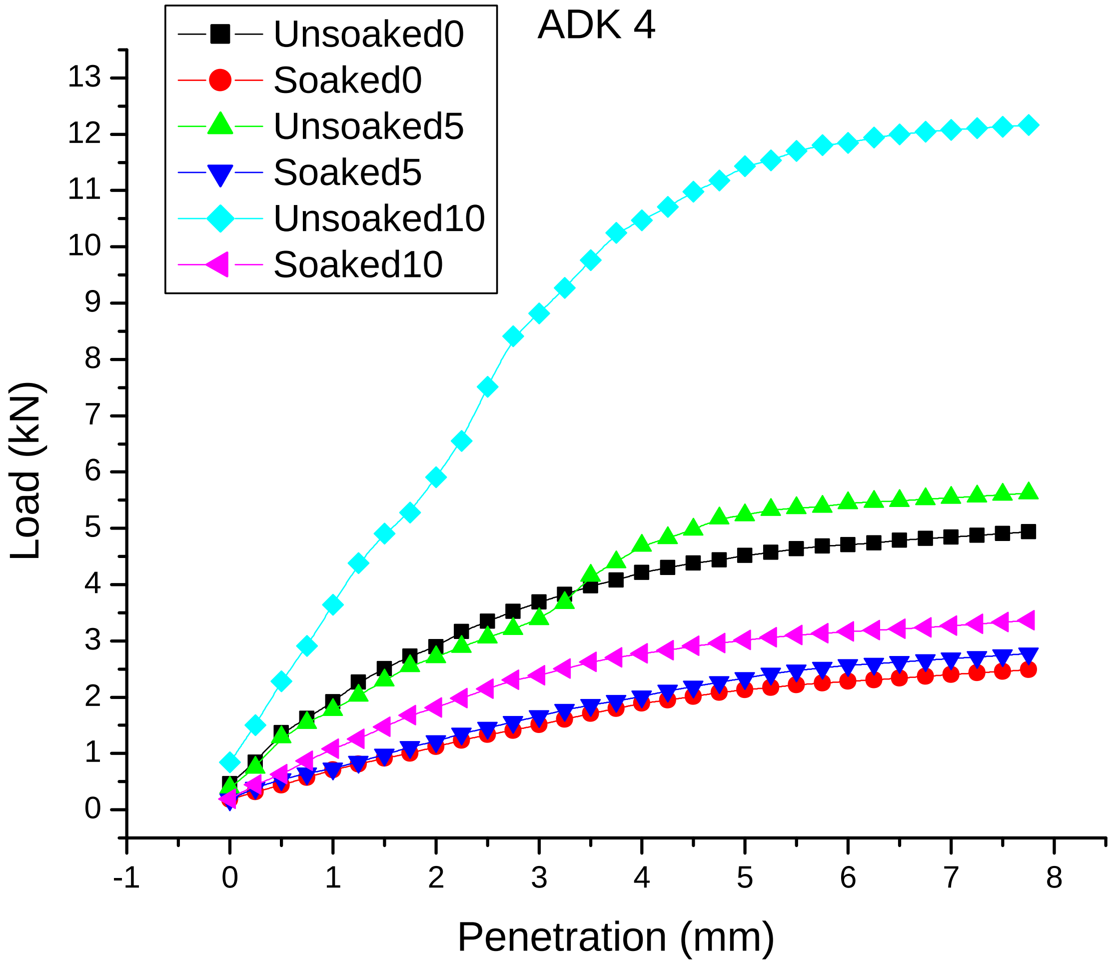
6
7
8
9
-1


ADK 4
Unsoaked0
Soaked0
Unsoaked5
Soaked5
Unsoaked10
Soaked10
Load (kN)
13
12
11
10
9
8
7
6
5
4
3
2
1
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-1
0
1
2
3
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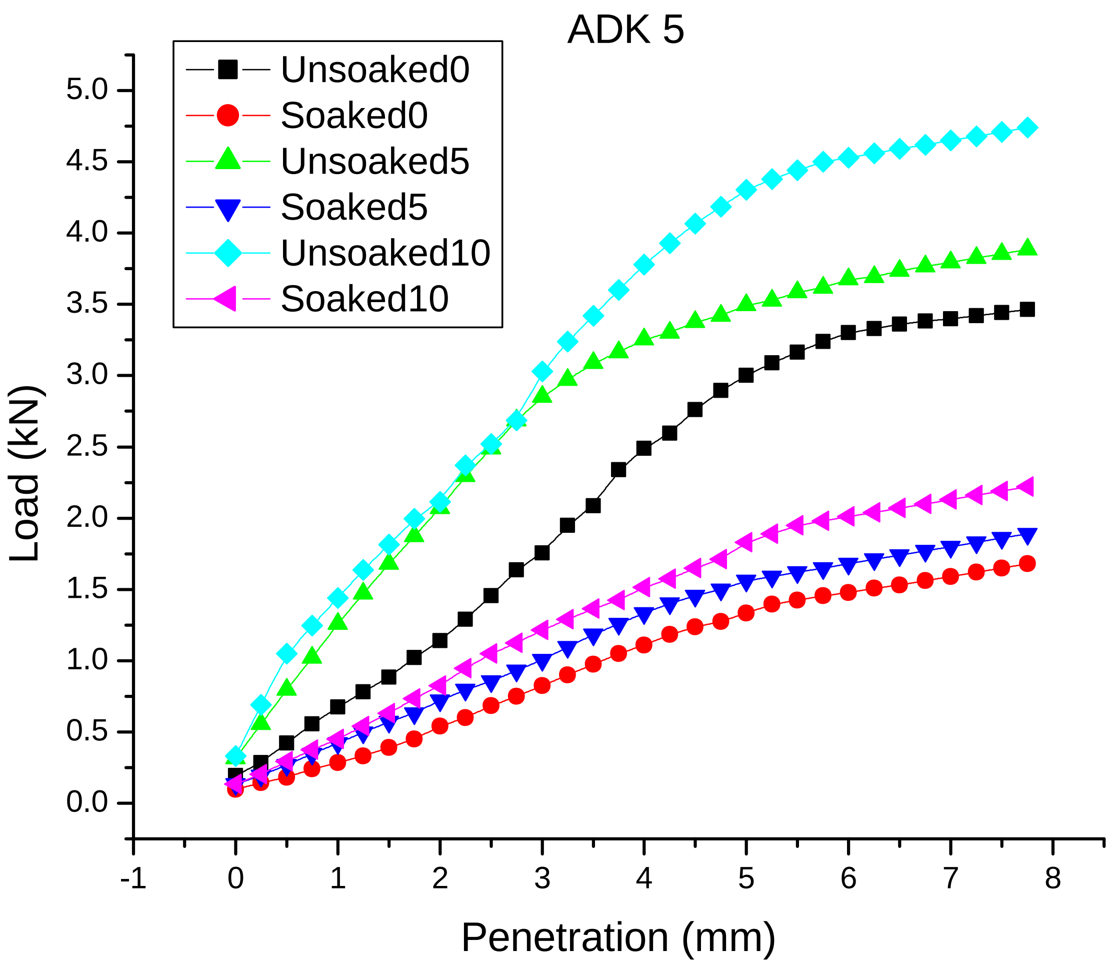
5
6
7
8
Penetration (mm)



ADK 5
Unsoaked0
Soaked0
Unsoaked5
Soaked5
Unsoaked10
Soaked10
Load (kN)
Penetration (mm)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
-1
0
1
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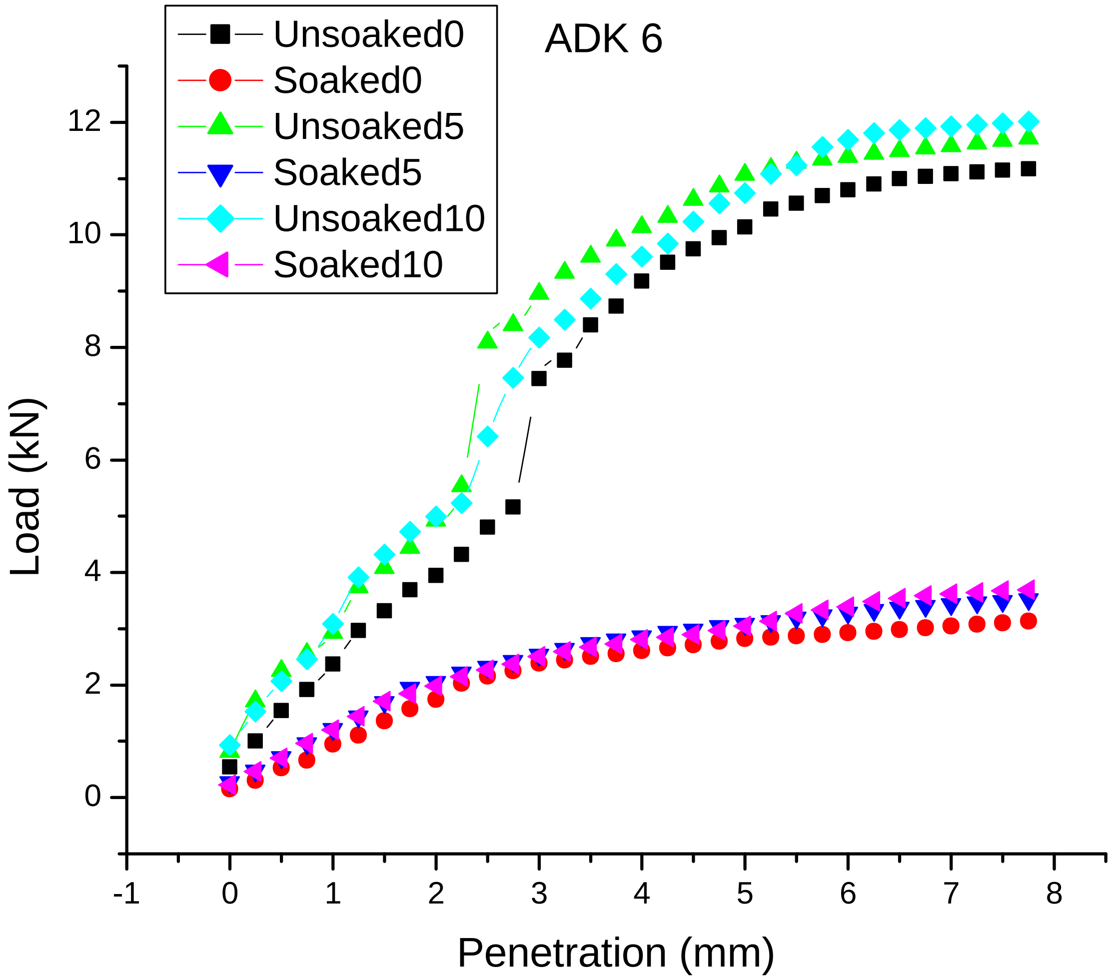
4
5
6
7
8


ADK 6
Unsoaked0
Soaked0
Unsoaked5
Soaked5
Unsoaked10
Soaked10
Load (kN)
Penetration (mm)
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3
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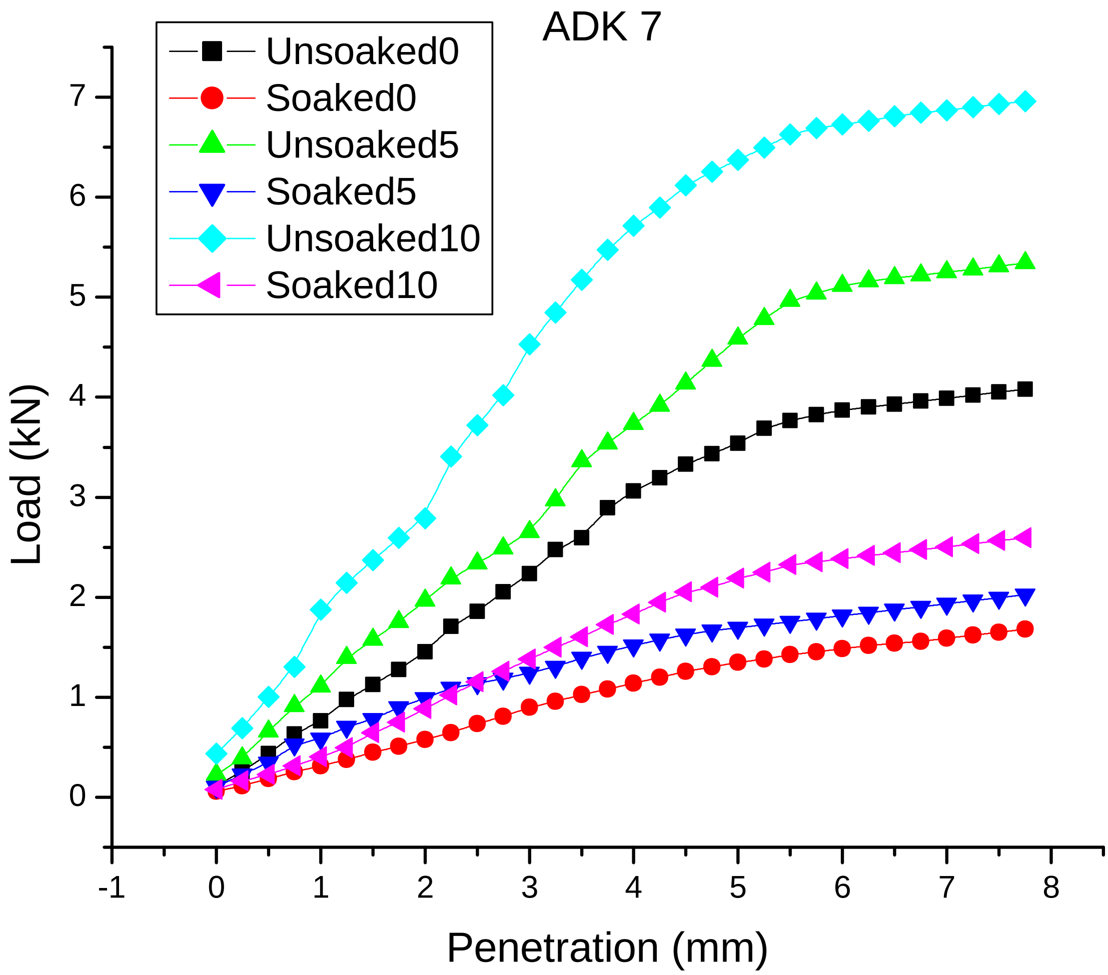
6
7
8
0
2
4
6
8
10
12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Unsoaked0
Soaked0
Unsoaked5
Soaked5
Unsoaked10
Soaked10
Load (kN)
Penetration (mm)
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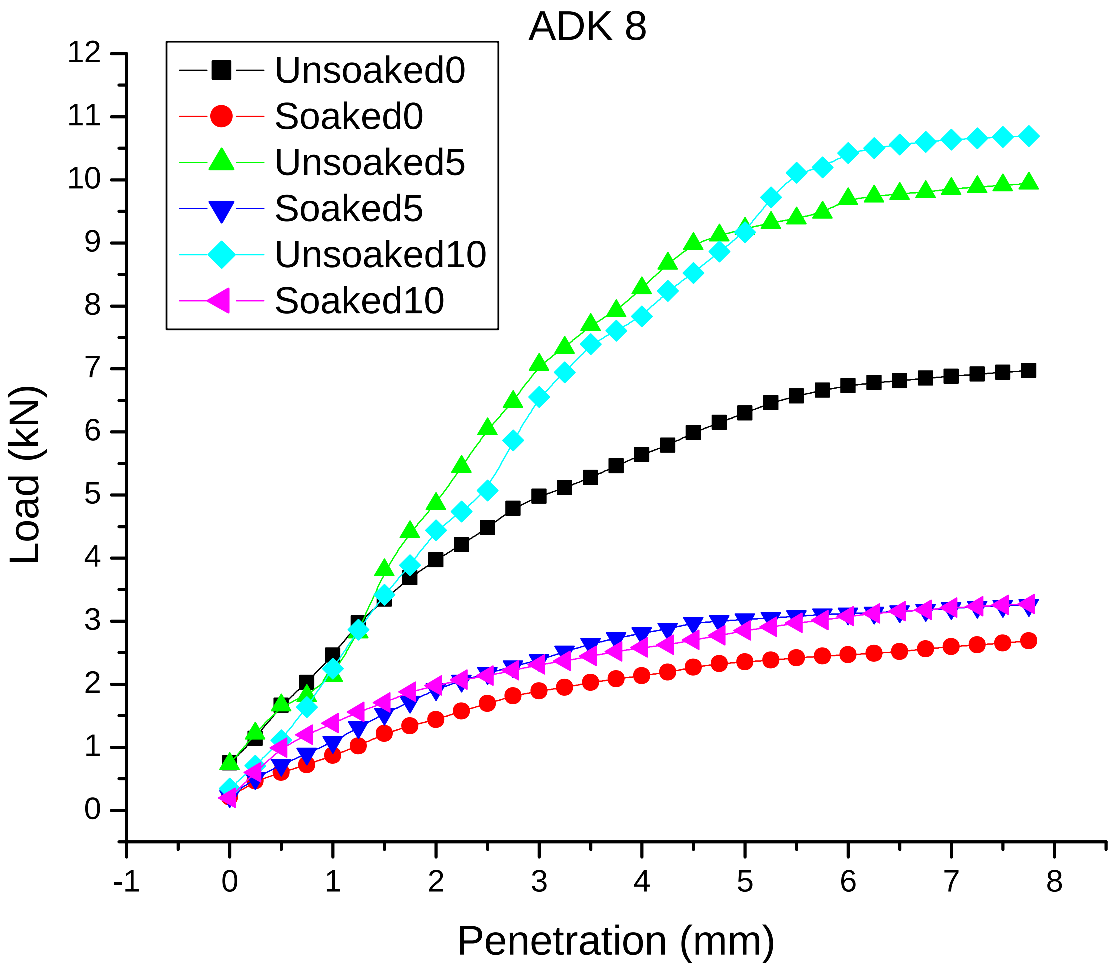
6
7
-1
8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Unsoaked0
Soaked0
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Unsoaked10
Soaked10
Load (kN)
Penetration (mm)
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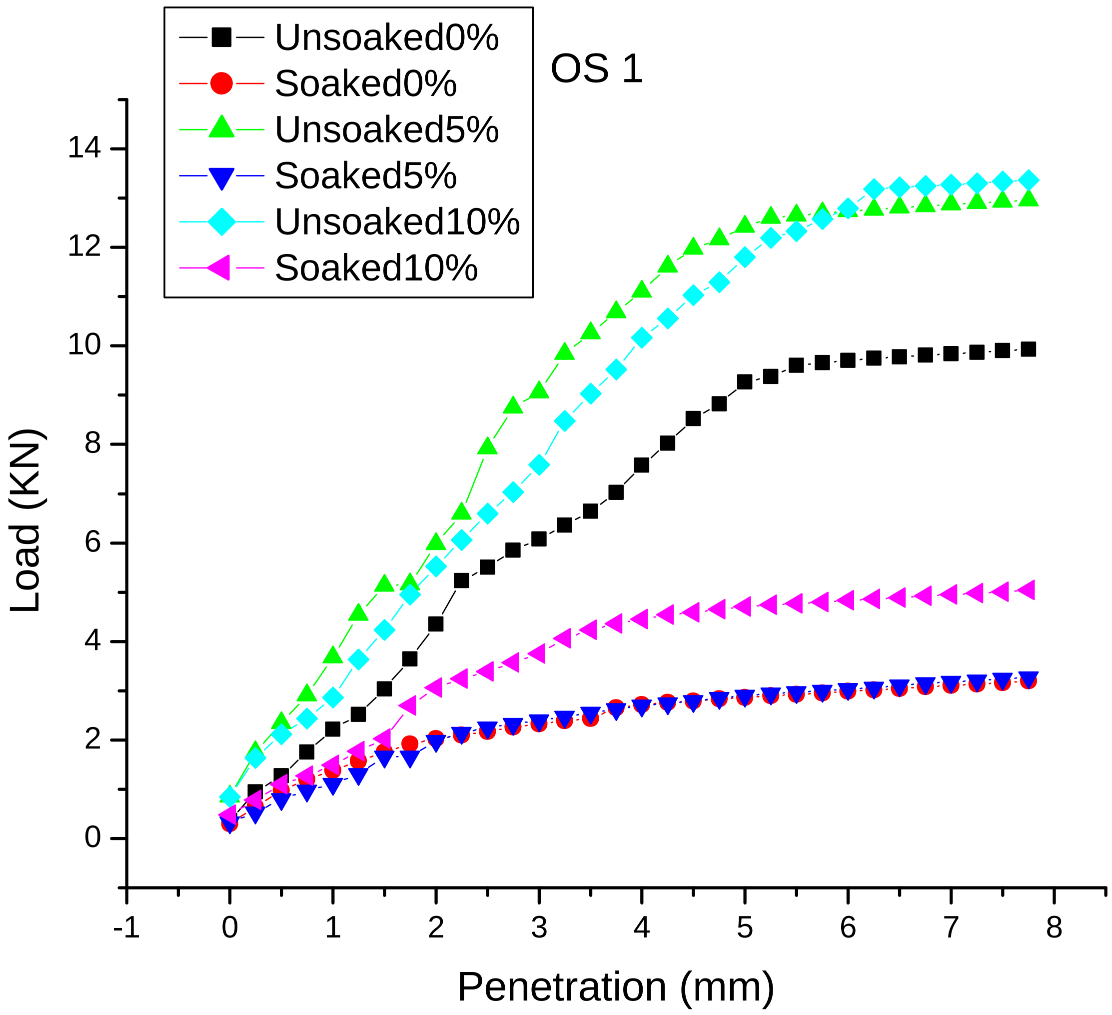
Unsoaked0%
Soaked0%
Unsoaked5%
Soaked5%
Unsoaked10%
Soaked10%
OS 1
Load (KN)
Penetration (mm)
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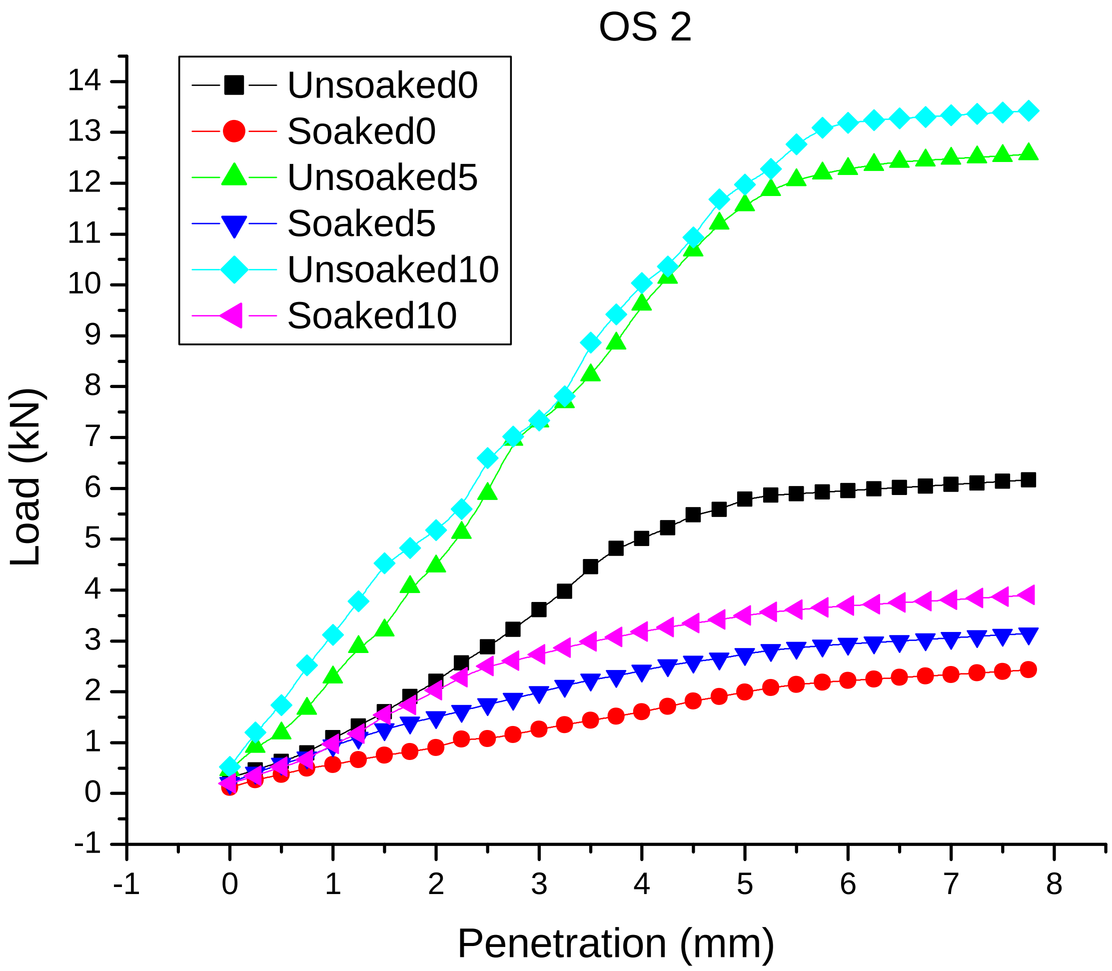
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Unsoaked0
Soaked0
Unsoaked5
Soaked5
Unsoaked10
Soaked10
Load (kN)
Penetration (mm)
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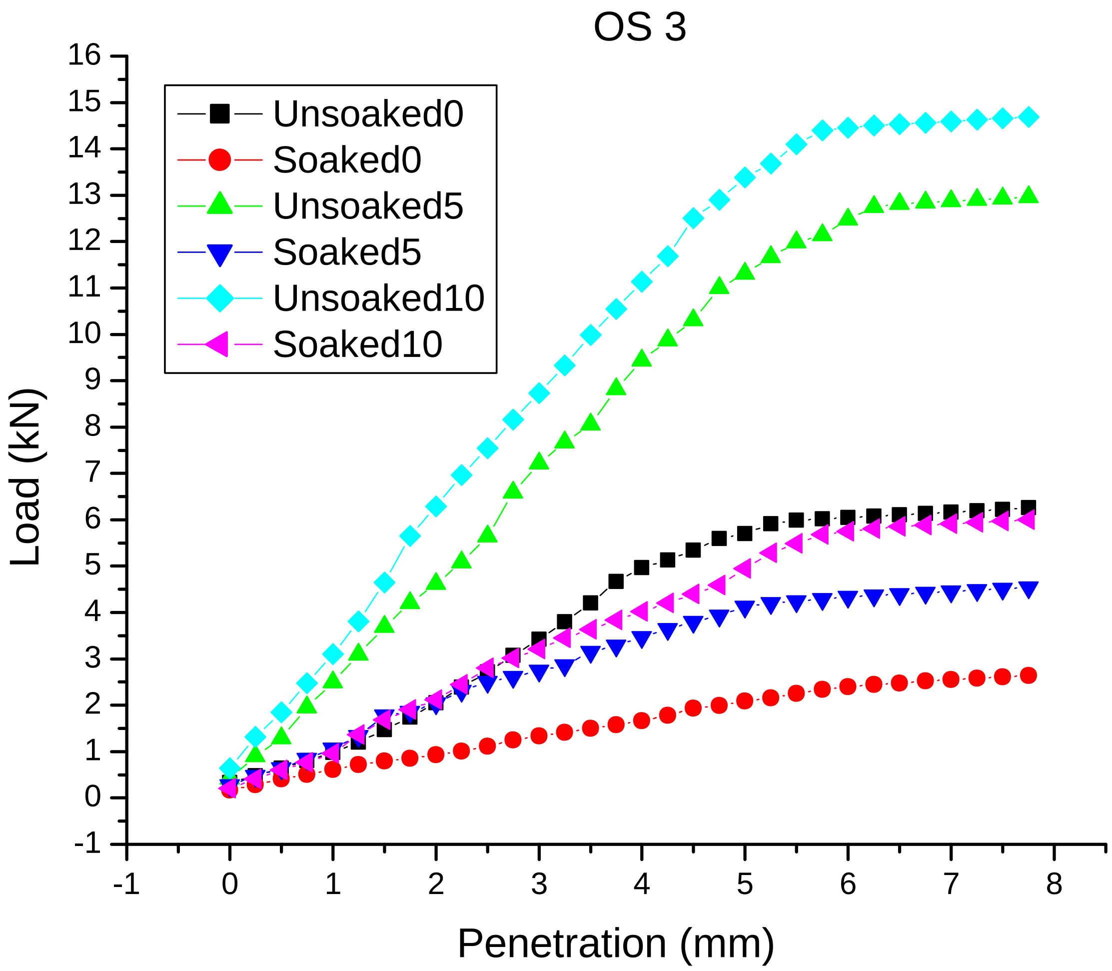
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Soaked0
Unsoaked5
Soaked5
Unsoaked10
Soaked10
Load (kN)
Penetration (mm)
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12
11
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9
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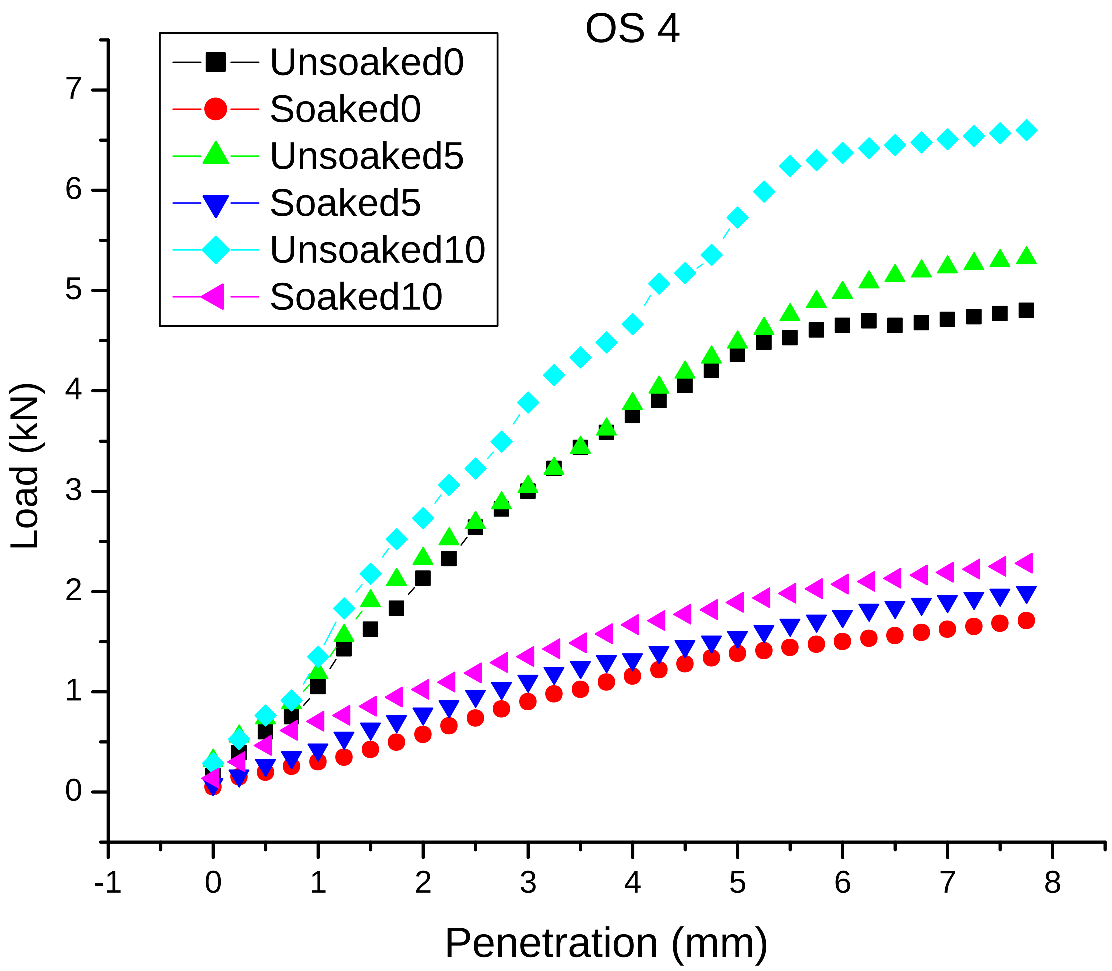
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Unsoaked0
Soaked0
Unsoaked5
Soaked5
Unsoaked10
Soaked10
Load (kN)
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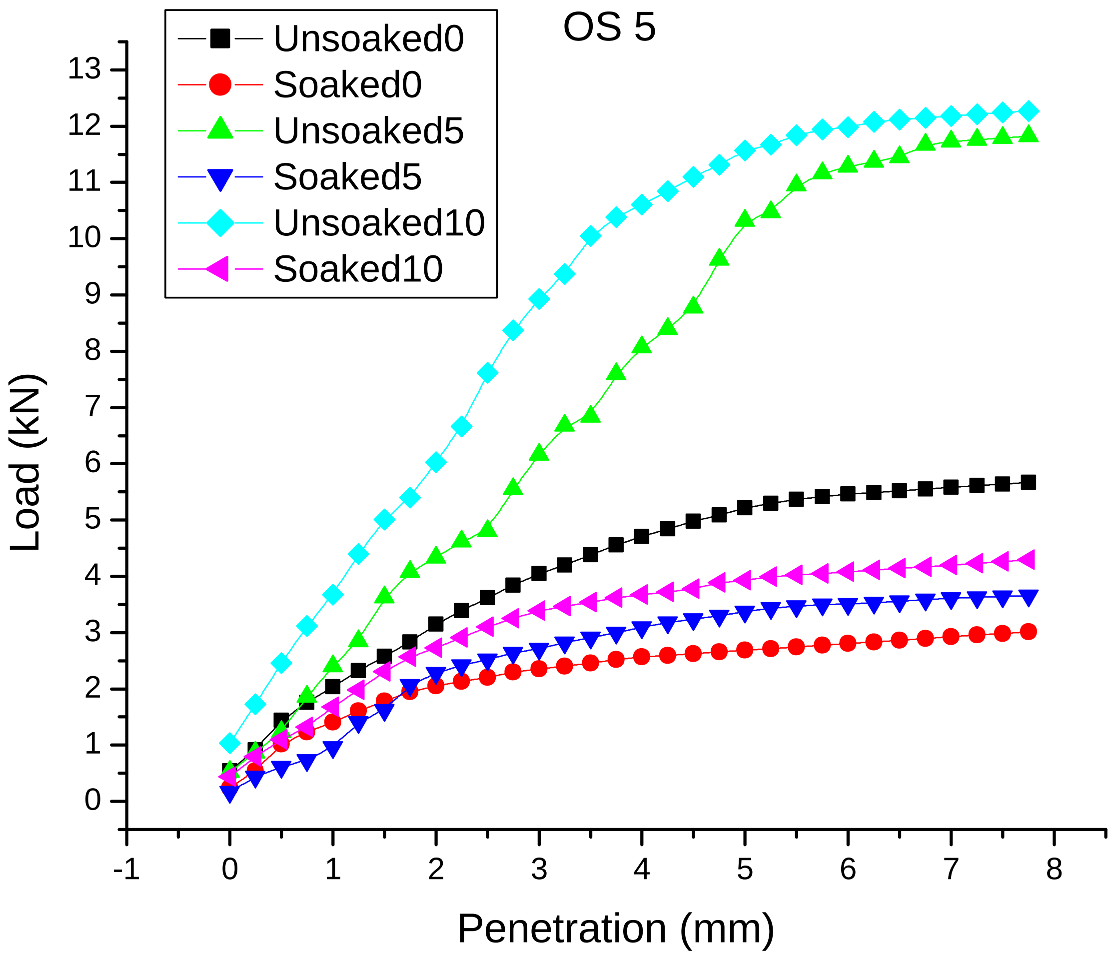
8
Penetration (mm)


OS 5
Unsoaked0
Soaked0
Unsoaked5
Soaked5
Unsoaked10
Soaked10
Load (kN)
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-1
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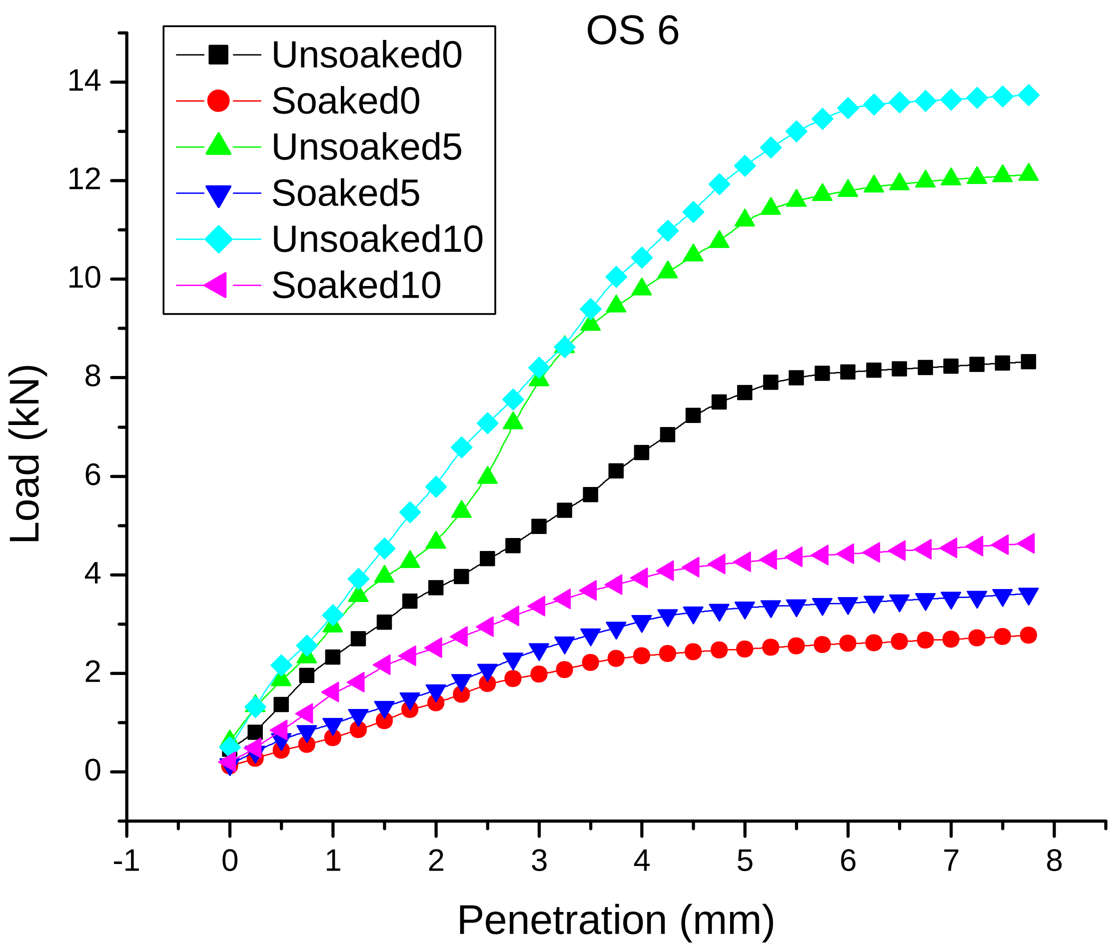
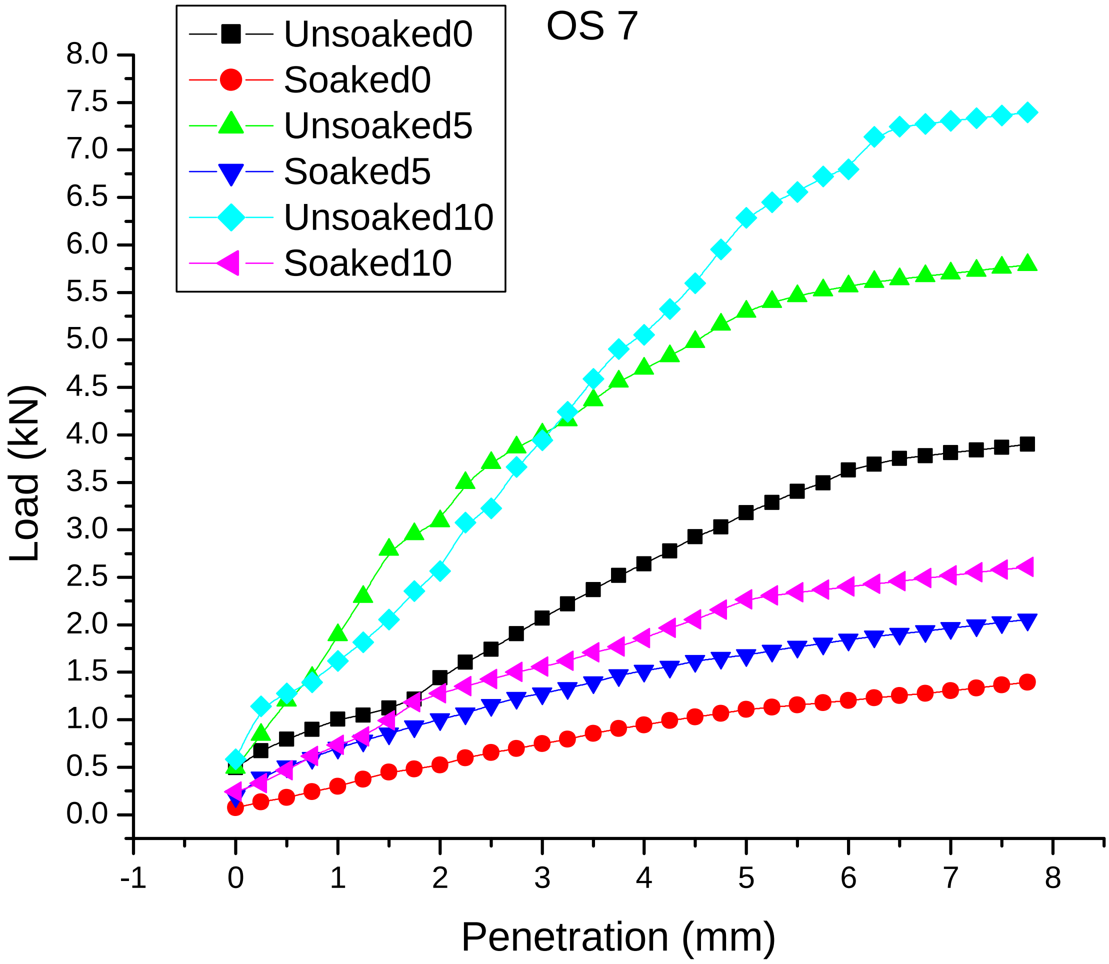
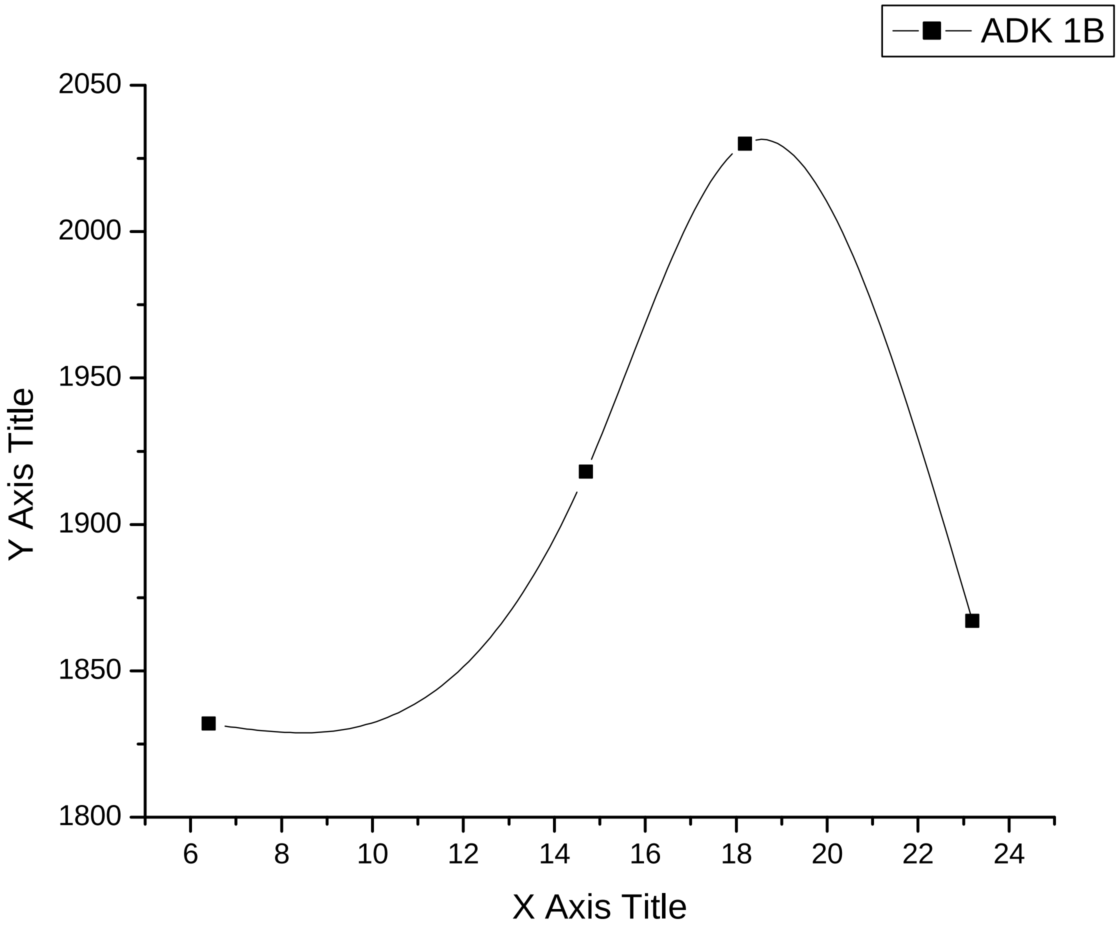
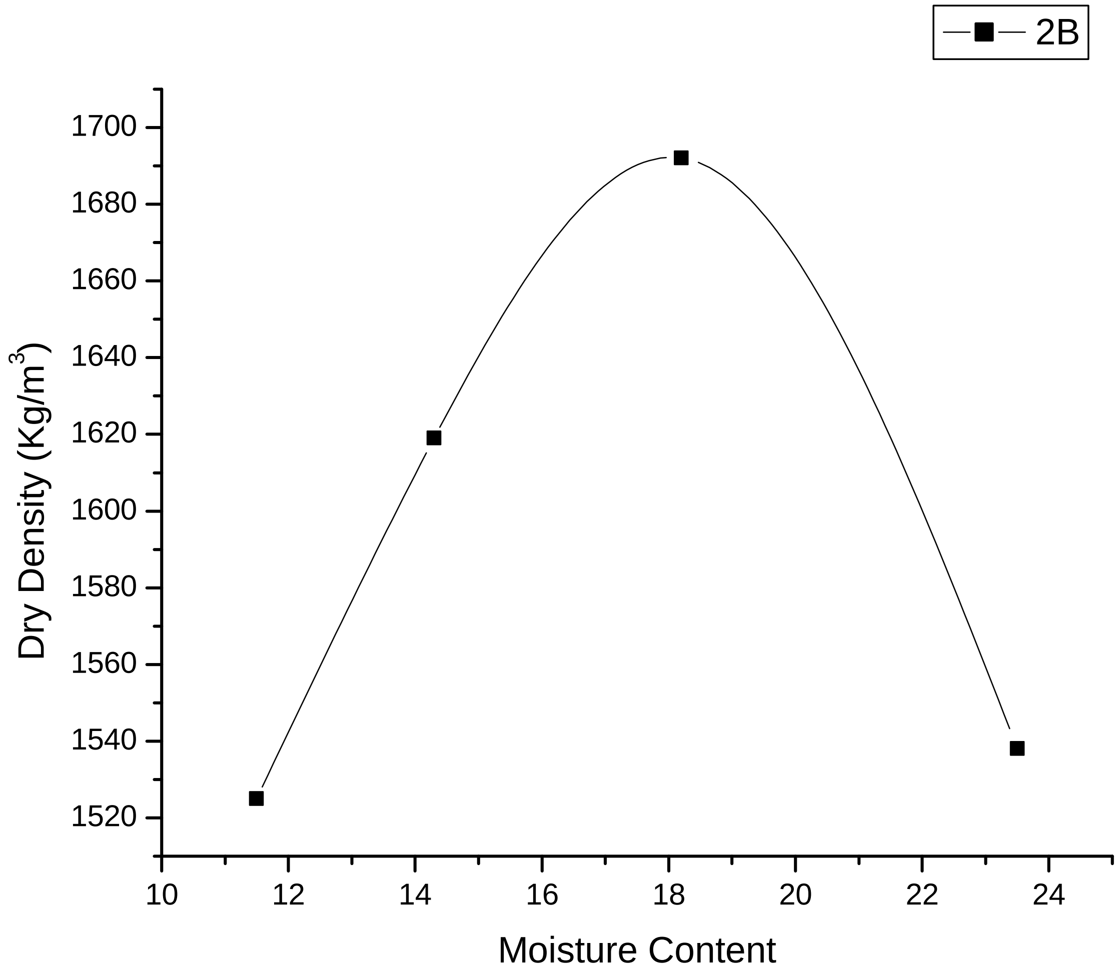
1
2
3
4
5
6
7
8
Penetration (mm)


OS 6
Unsoaked0
Soaked0
Unsoaked5
Soaked5
Unsoaked10
Soaked10
Load (kN)
Penetration (mm)
14
12
10
8
6
4
2
0
-1
0
1
2
3
4
5
6
7
8

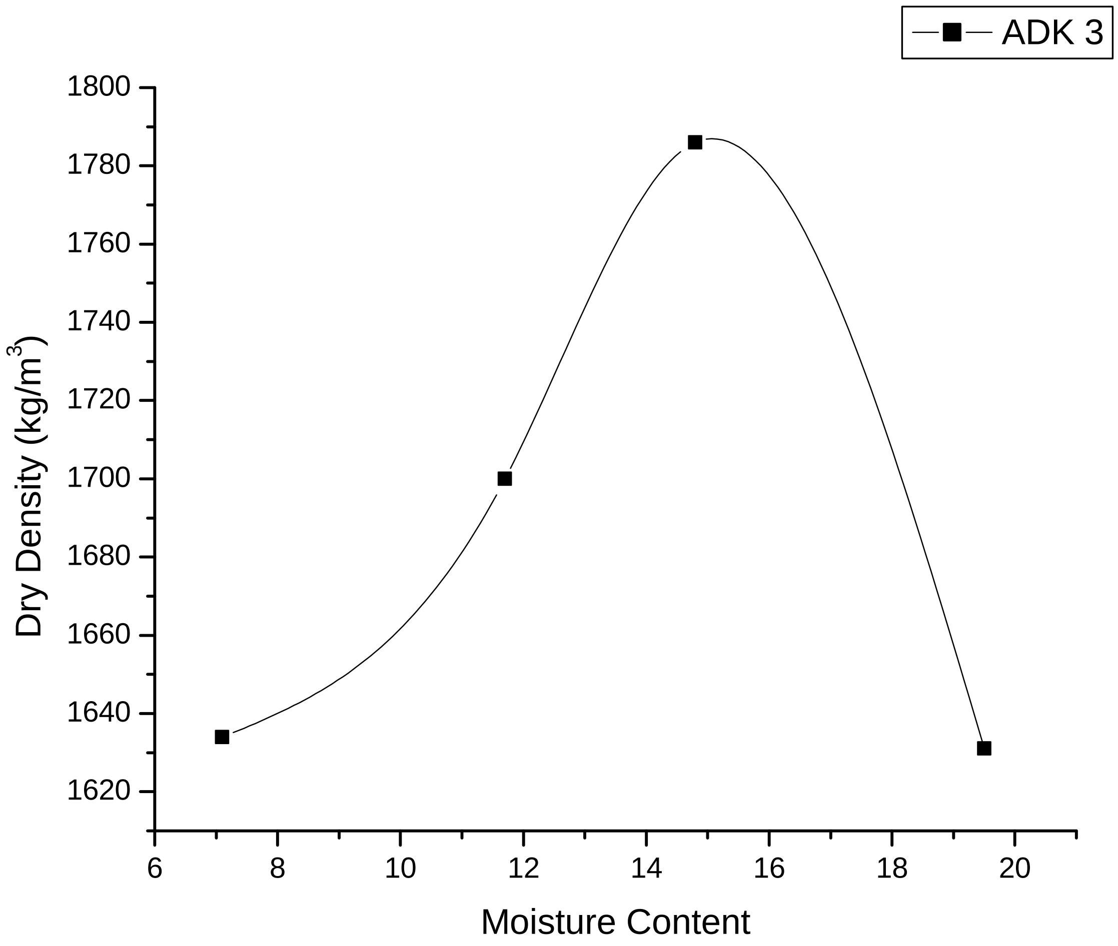

OS 7
Unsoaked0
Soaked0
Unsoaked5
Soaked5
Unsoaked10
Soaked10
Load (kN)
8.0
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
-1
0
1
2
3
4
5
6
7
8
Penetration (mm)

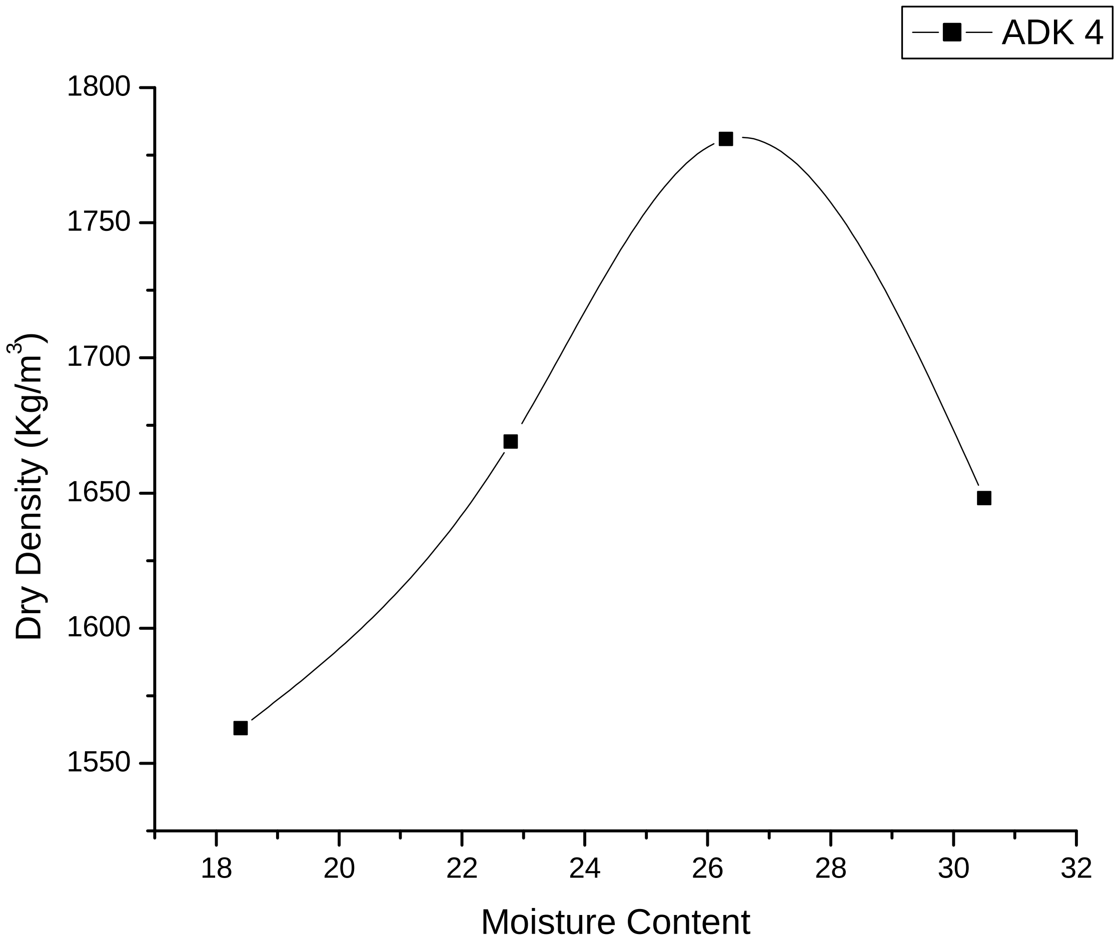
B-2 Compaction curves

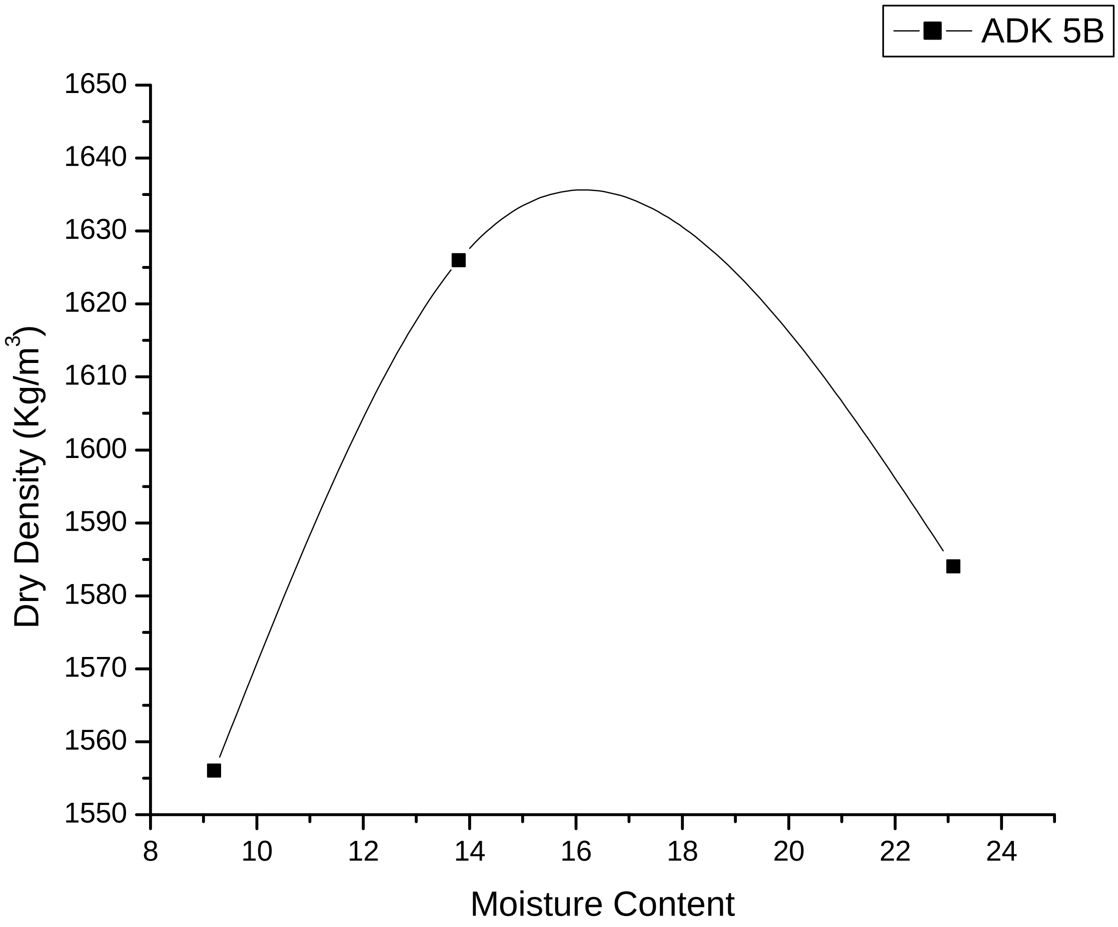


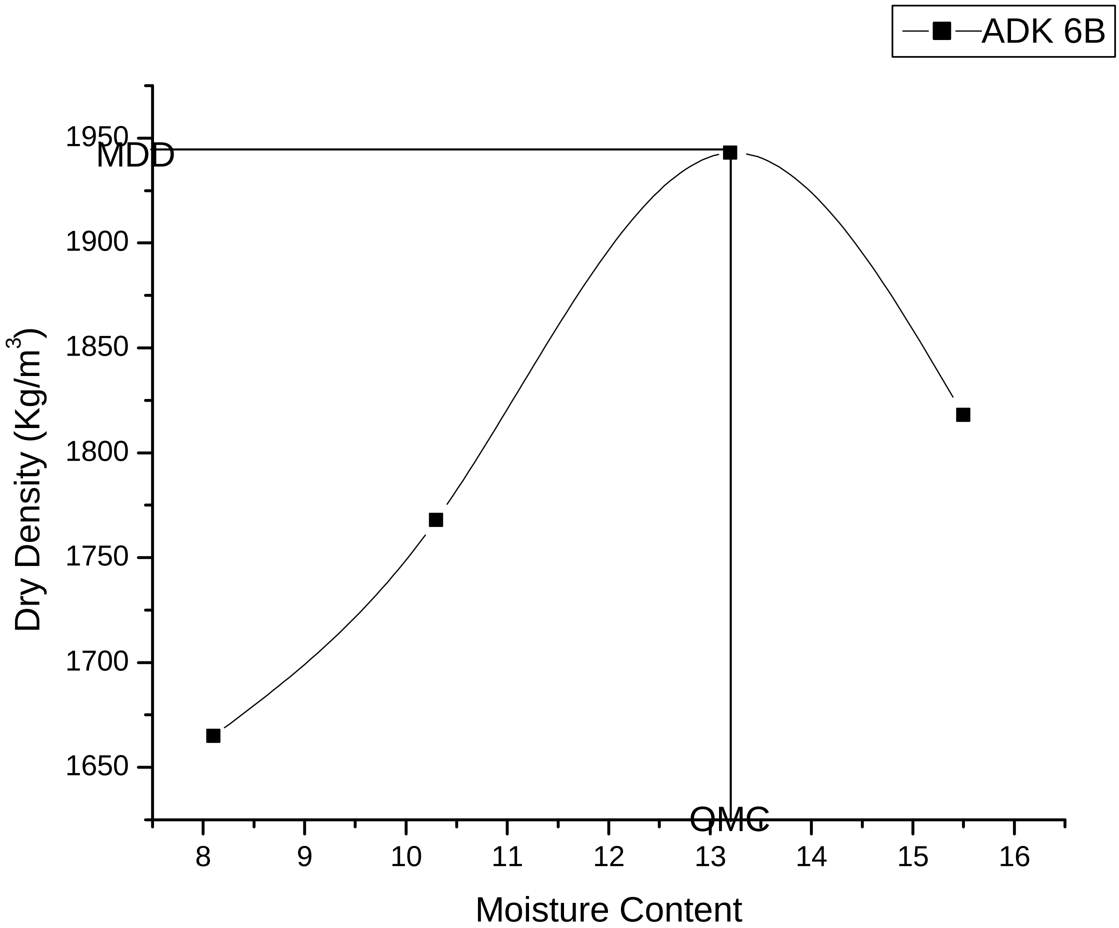


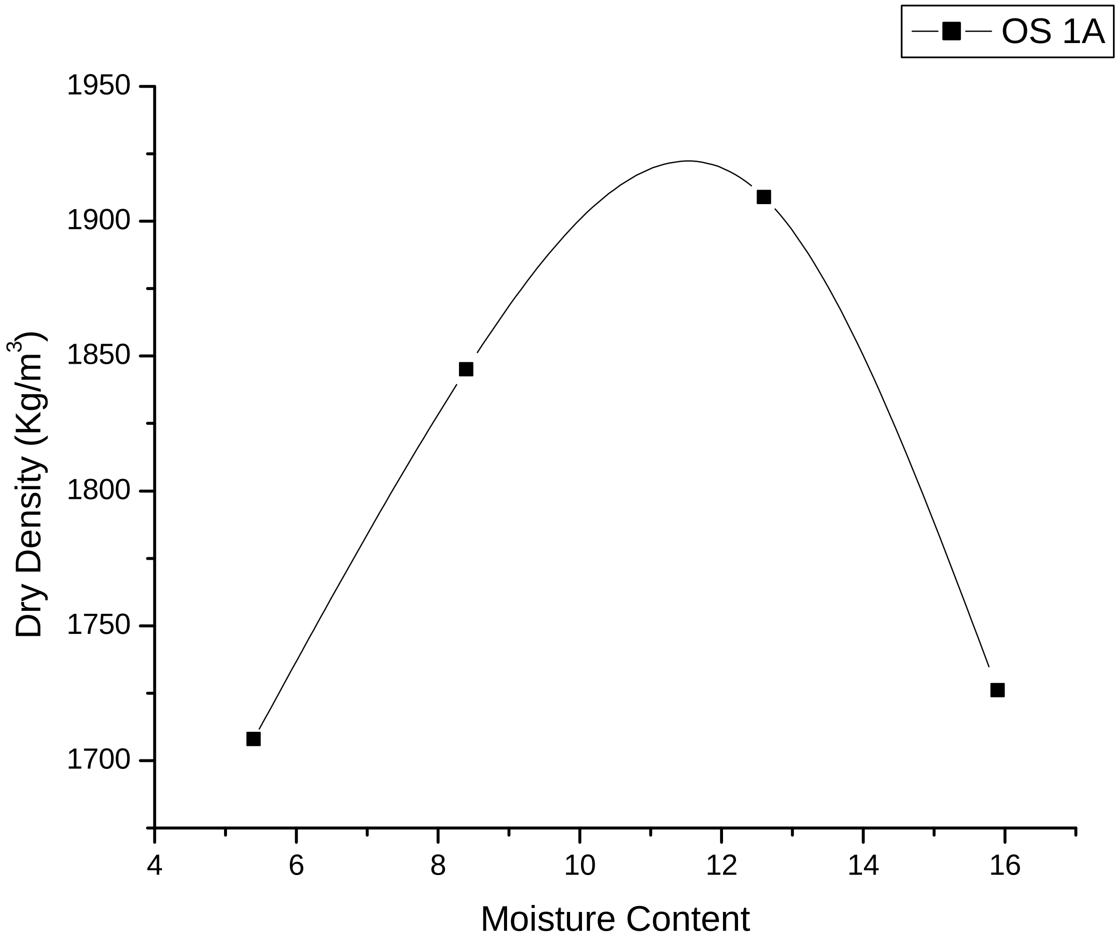
2B
Dry Density (Kg/m3)
1700
1680
1660
1640
1620
1600
1580
1560
1540
1520
10
12
14
16
18
20
22
24
Moisture Content

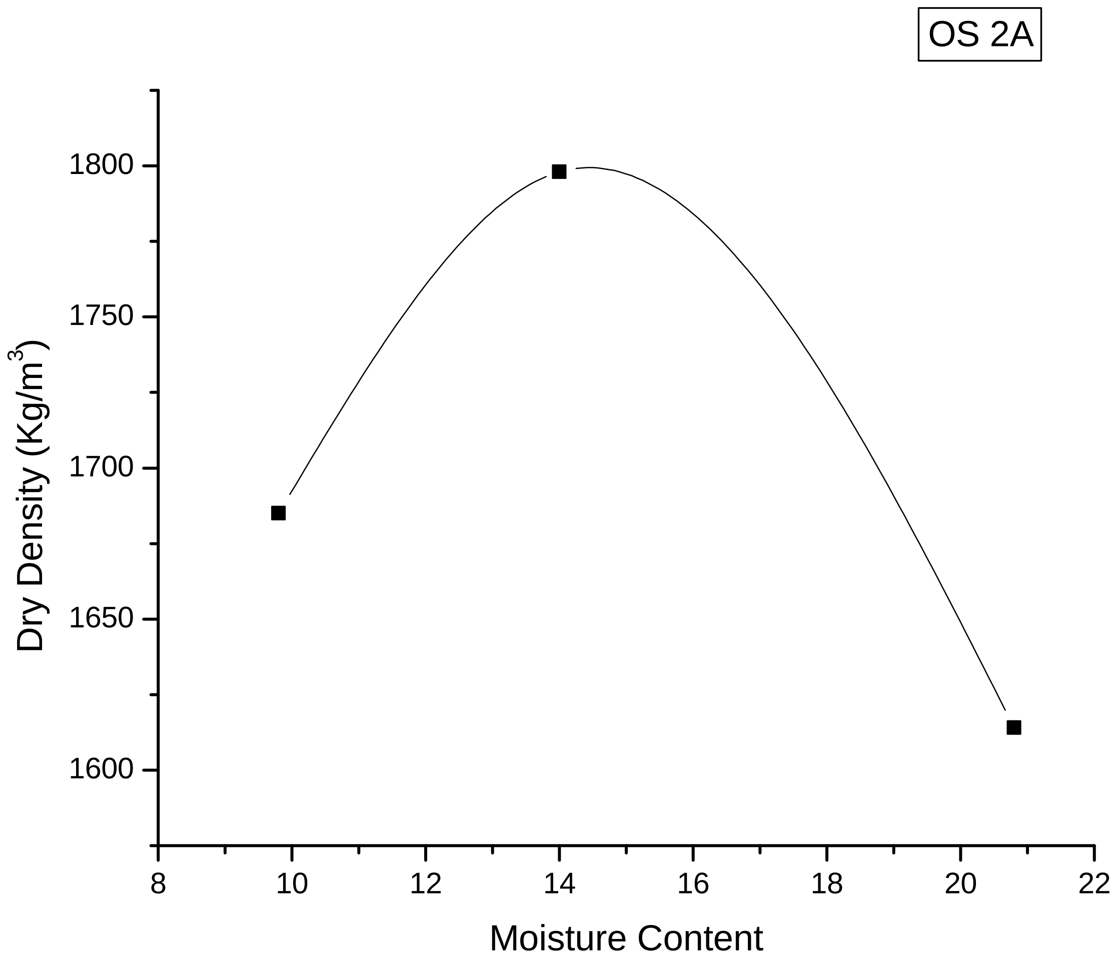

ADK 3
Dry Density (kg/m3)
1800
1780
1760
1740
1720
1700
1680
1660
1640
1620
6
8
10
12
14
16
18
20
Moisture Content

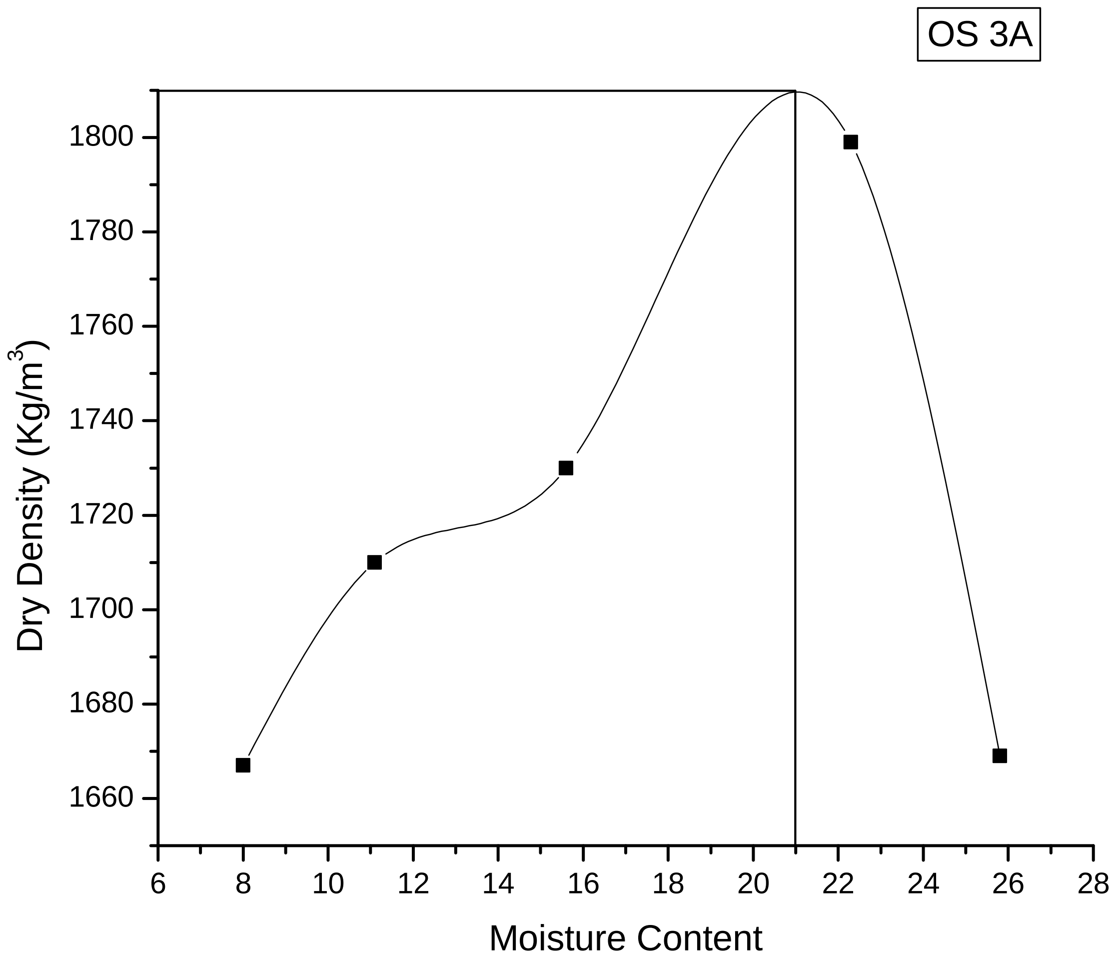

ADK 4
1800
1750
1700
1650
1600
1550
Dry Density (Kg/m3)
18
20
22
24
26
28
30
32
Moisture Content

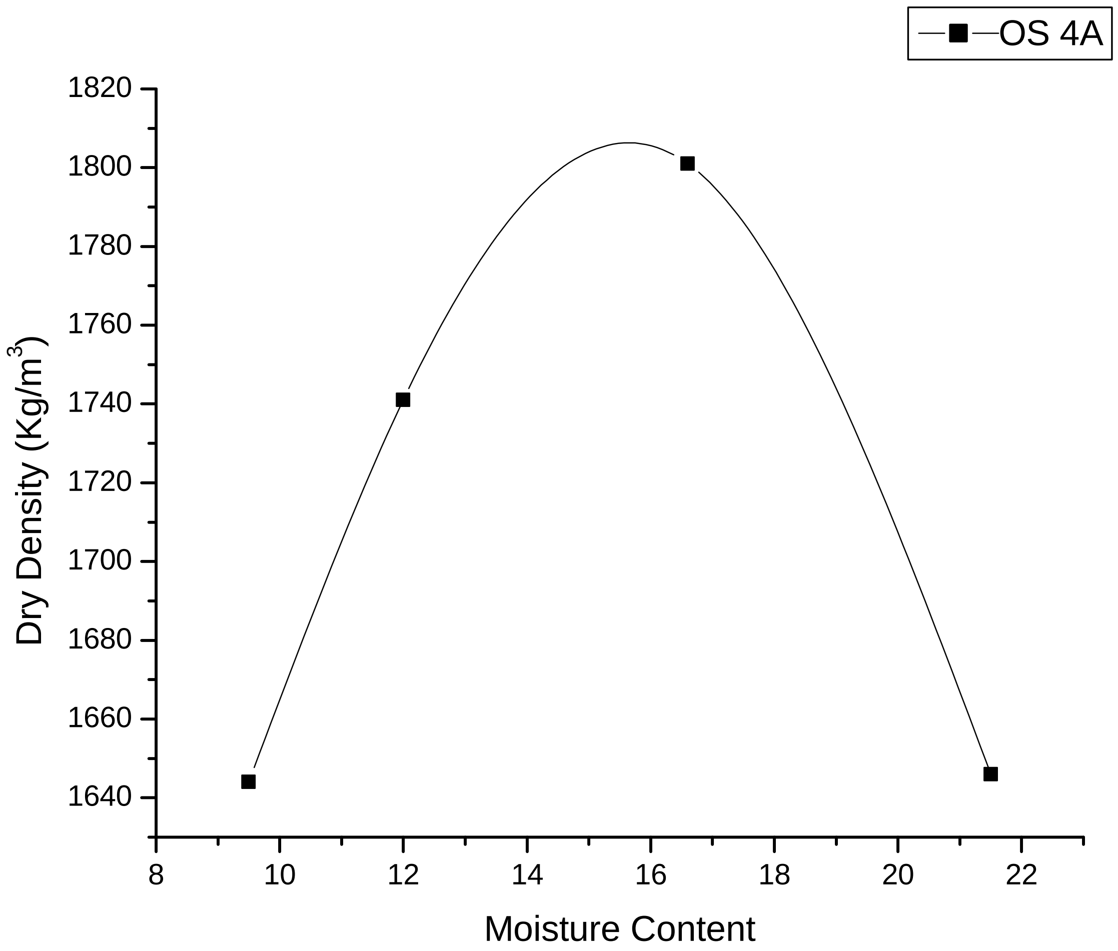

ADK 5B
1650
1640
1630
1620
1610
1600
1590
1580
1570
1560
1550
Dry Density (Kg/m3)
8
10
12
14
16
18
20
22
24
Moisture Content

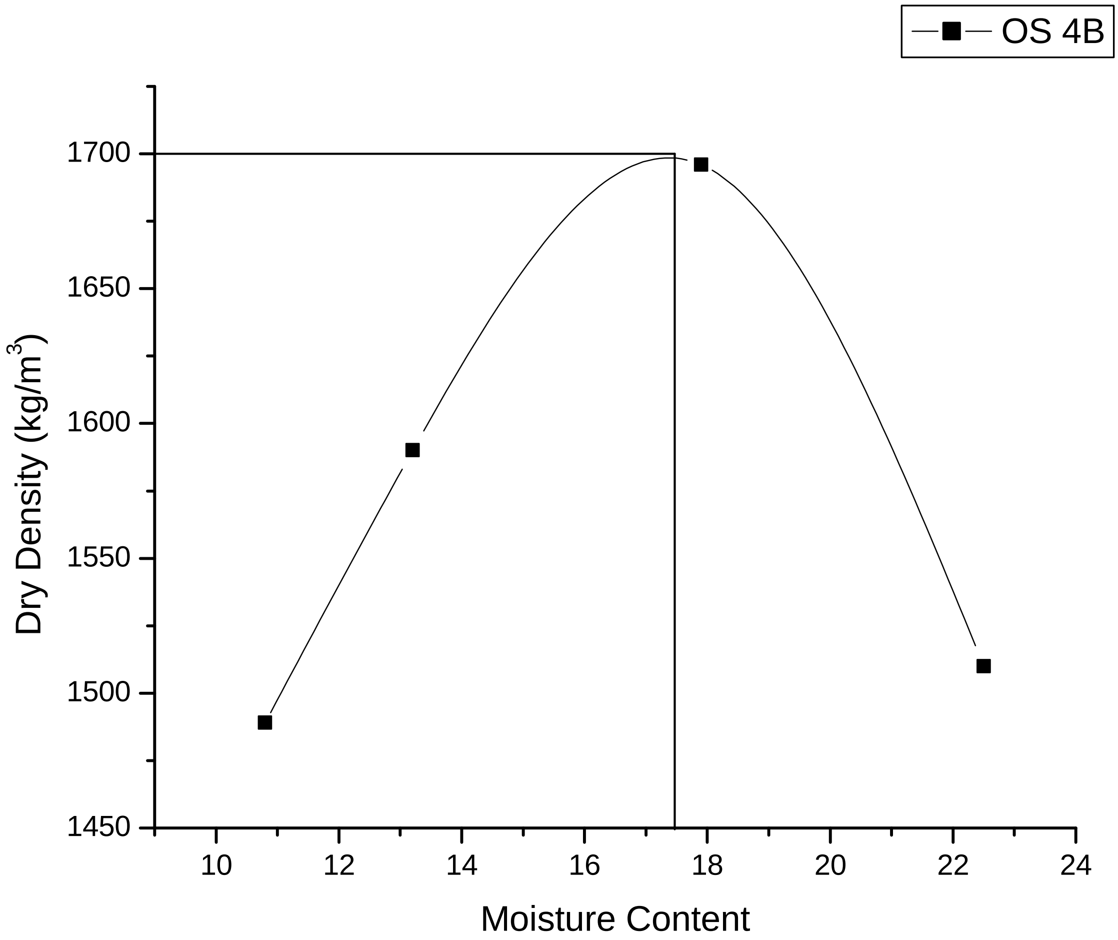

ADK 6B
1950
MDD
1900
1850
1800
1750
1700
1650
Dry Density (Kg/m3)
OMC
8
9
10
11
12
13
14
15
16
Moisture Content

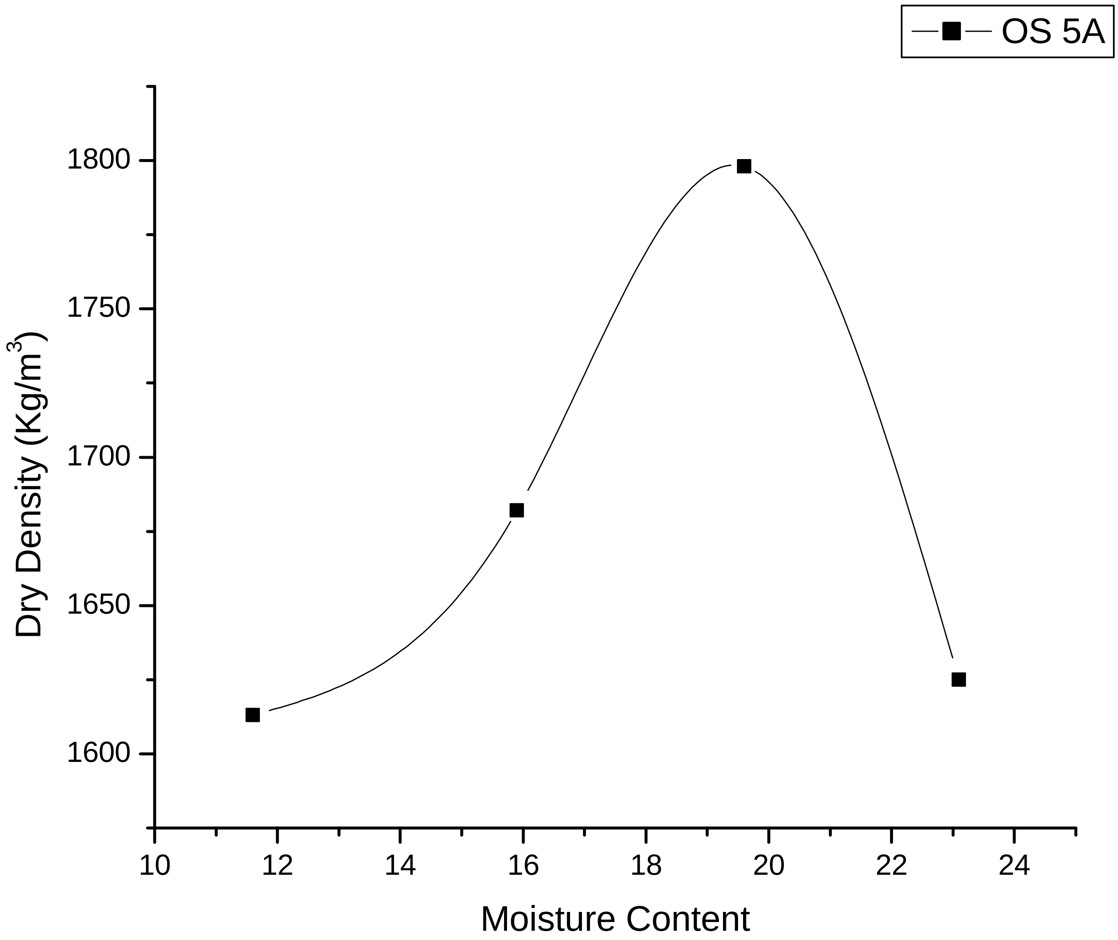

OS 1A
1950
1900
1850
1800
1750
1700
Dry Density (Kg/m3)
4
6
8
10
12
14
16
Moisture Content

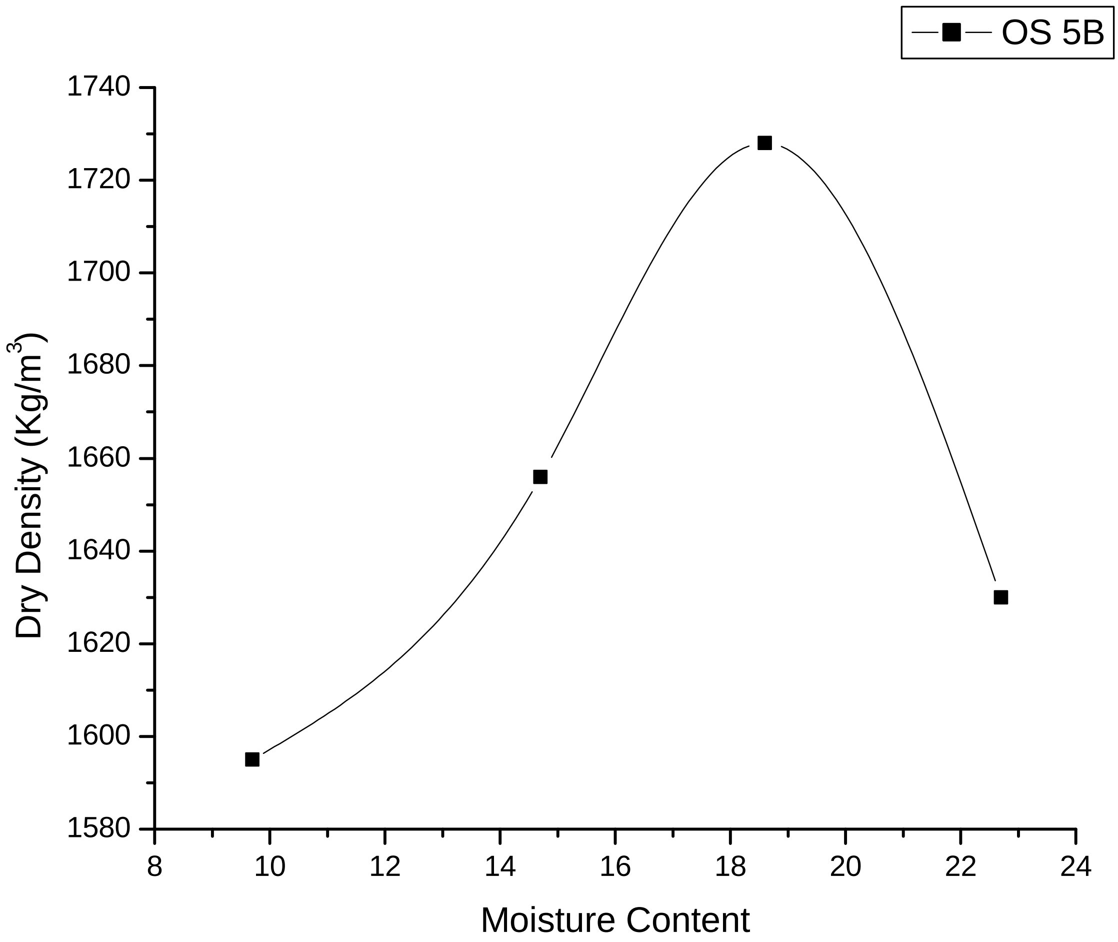

OS 2A
Dry Density (Kg/m3)
1800
1750
1700
1650
1600
8
10
12
14
16
18
20
22
Moisture Content

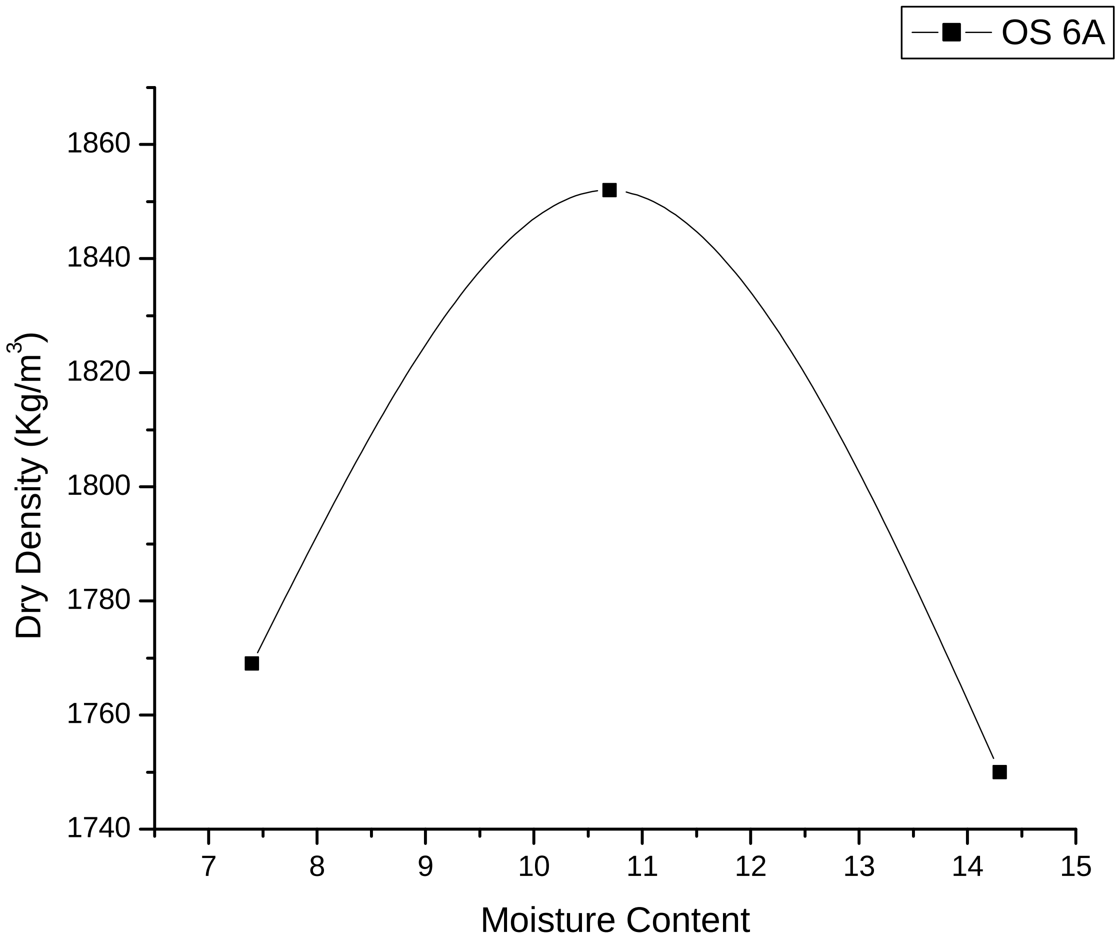

OS 3A
Dry Density (Kg/m3)
1660
1680
1700
1720
1740
1760
1780
1800
6
8
10
12
14
16
18
20
22
24
26
28
Moisture Content

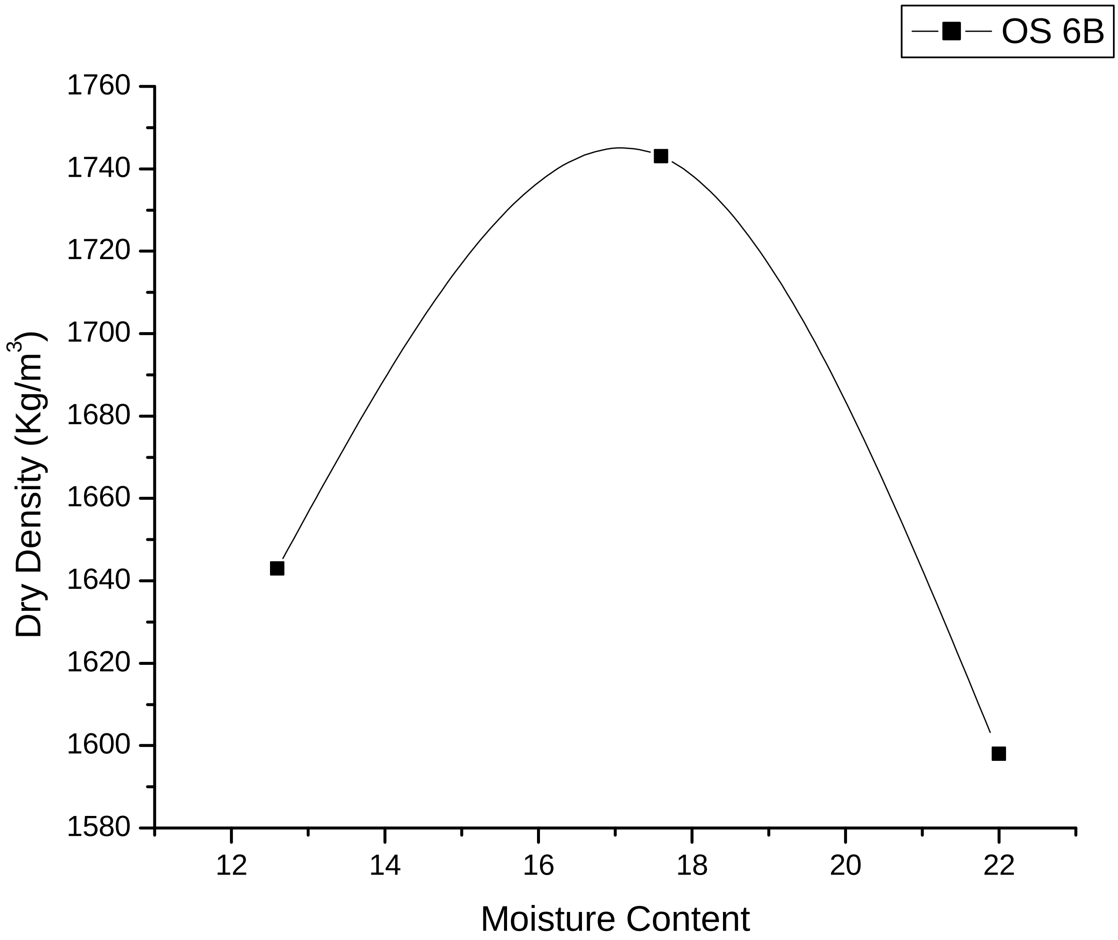

OS 4A
1820
1800
1780
1760
1740
1720
1700
1680
1660
1640
Dry Density (Kg/m3)
8
10
12
14
16
18
20
22
Moisture Content

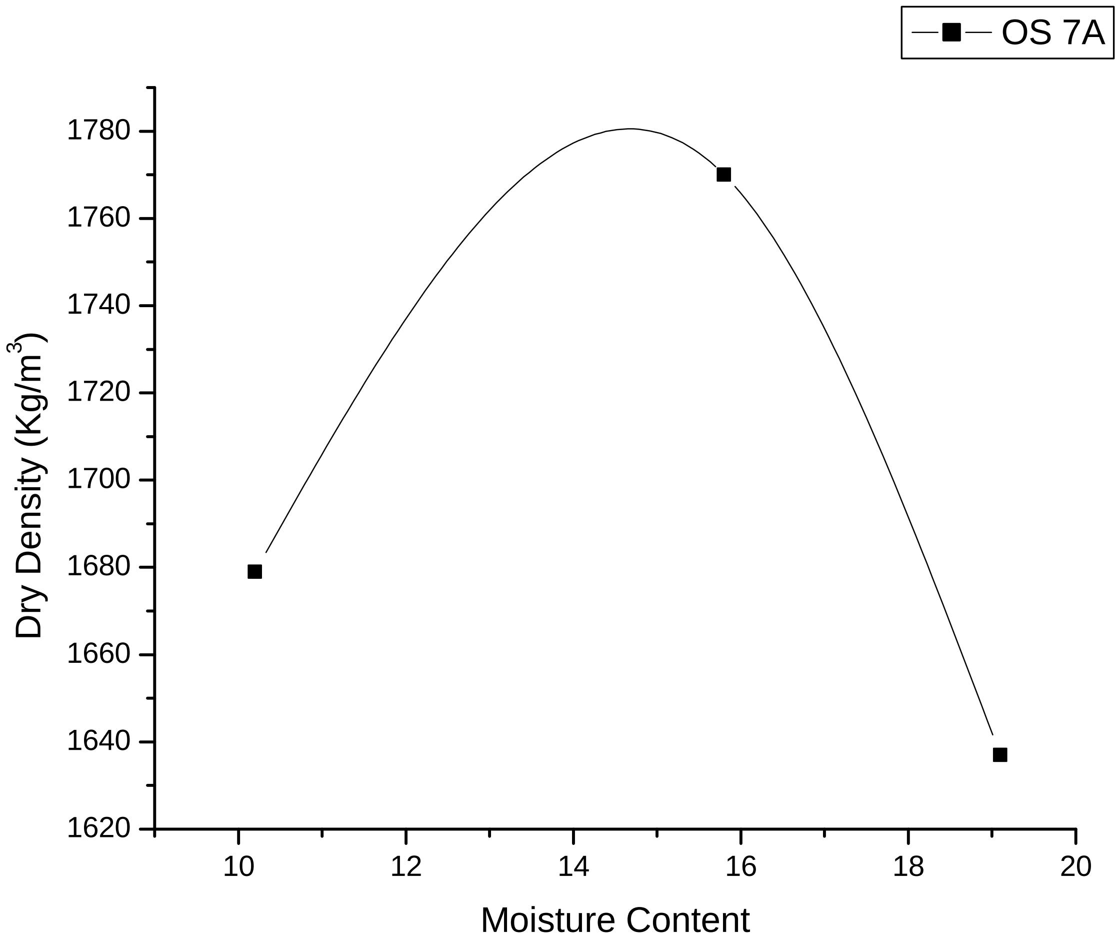

OS 4B
Dry Density (kg/m3)
1700
1650
1600
1550
1500
1450
10
12
14
16
18
20
22
24
Moisture Content

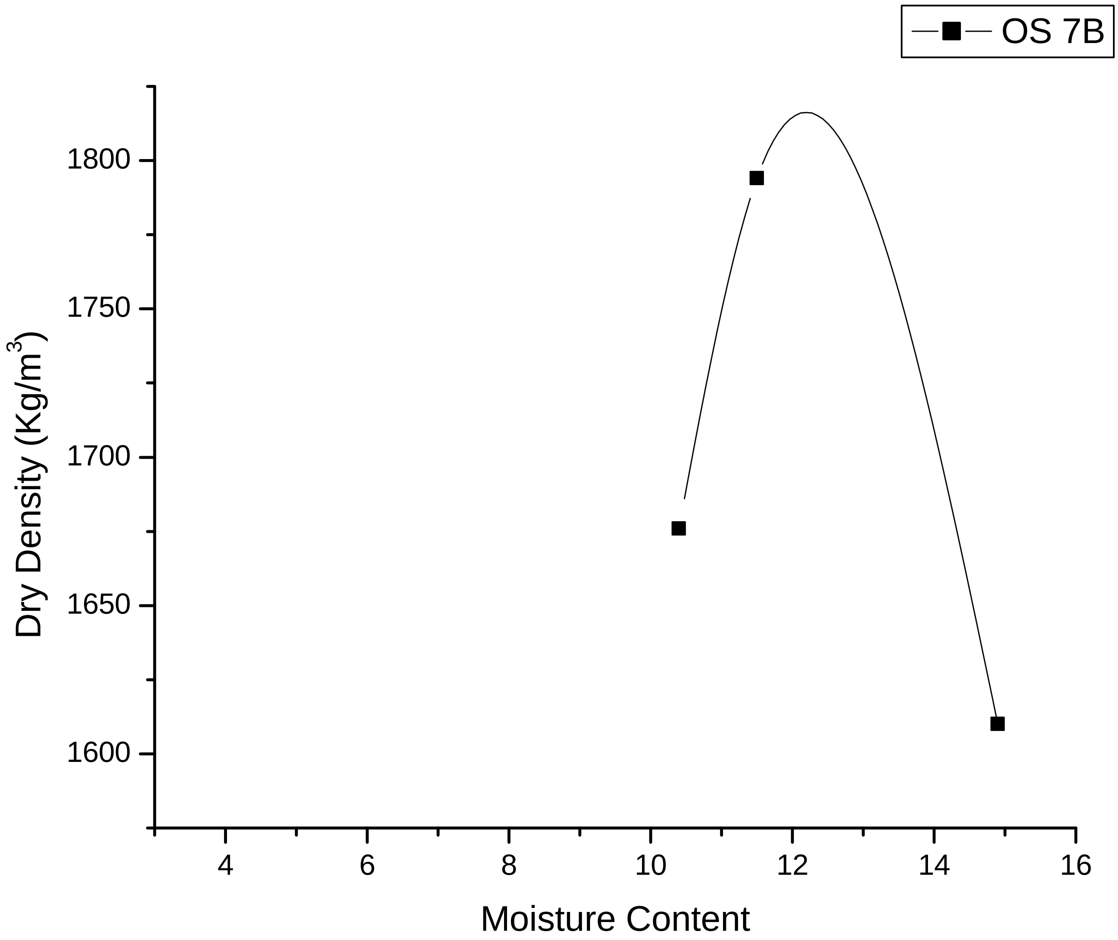

OS 5A
Dry Density (Kg/m3)
1600
1650
1700
1750
1800
10
12
14
16
18
20
22
24
Moisture Content


OS 5B
1740
1720
1700
1680
1660
1640
1620
1600
1580
Dry Density (Kg/m3)
8
10
12
14
16
18
20
22
24
Moisture Content

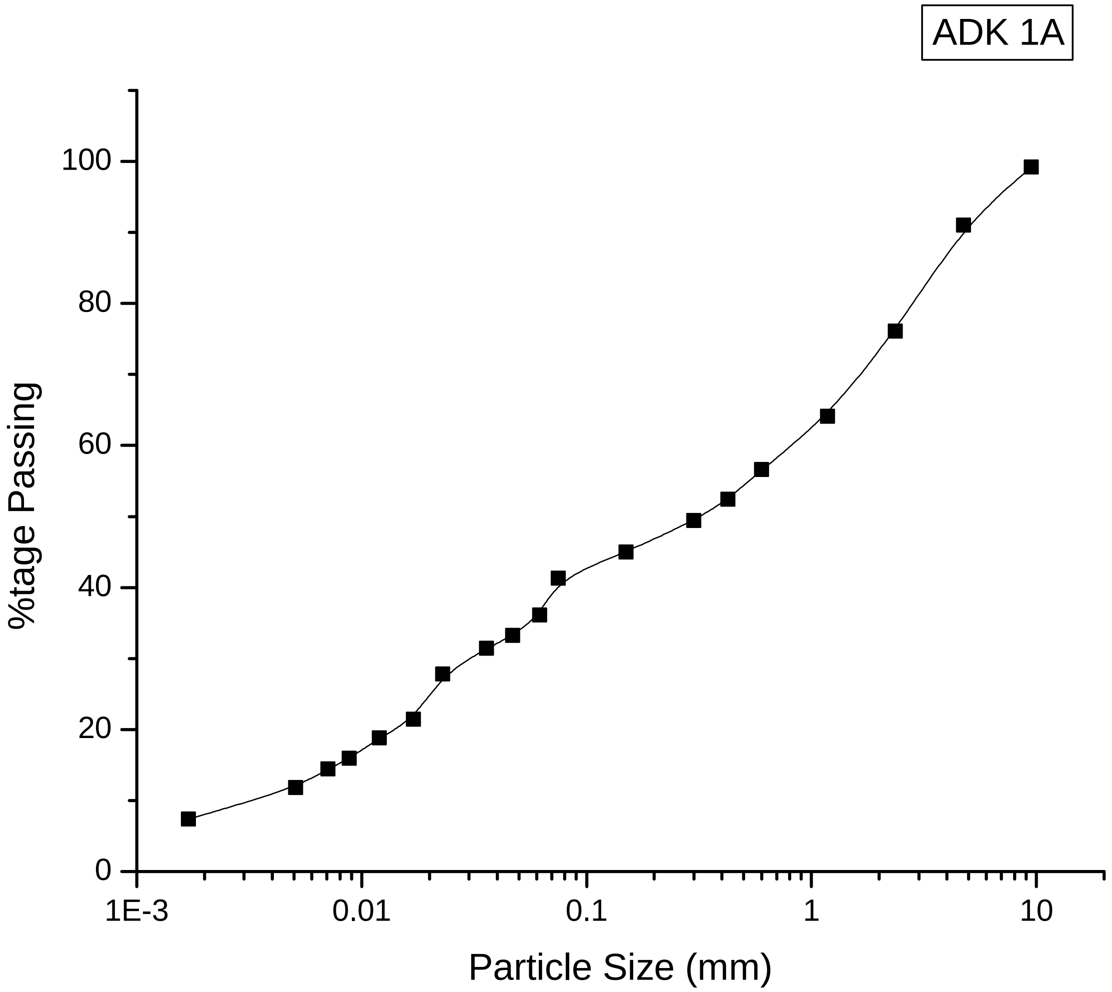

OS 6A
Dry Density (Kg/m3)
1860
1840
1820
1800
1780
1760
1740
7
8
9
10
11
12
13
14
15
Moisture Content

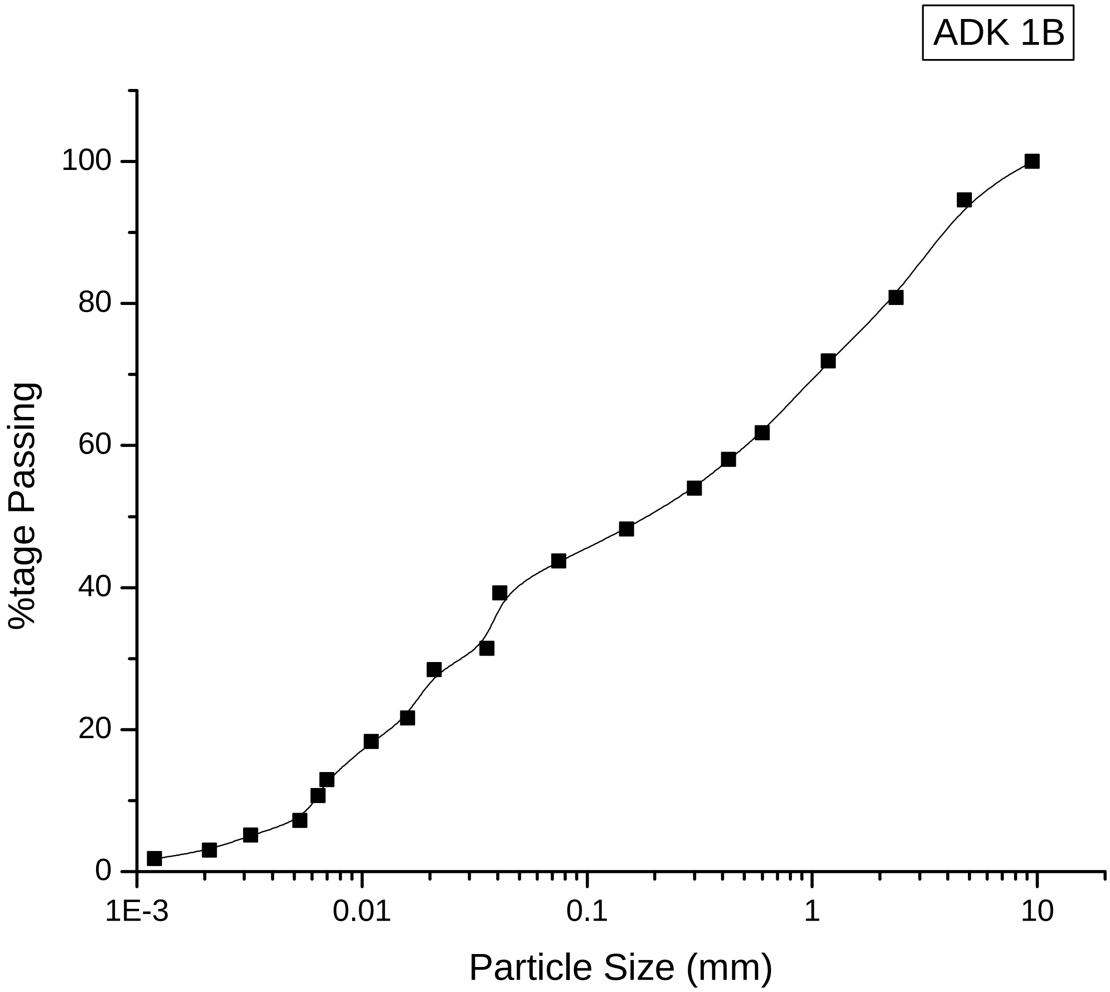

OS 6B
1760
1740
1720
1700
1680
1660
1640
1620
1600
1580
Dry Density (Kg/m3)
12
14
16
18
20
22
Moisture Content

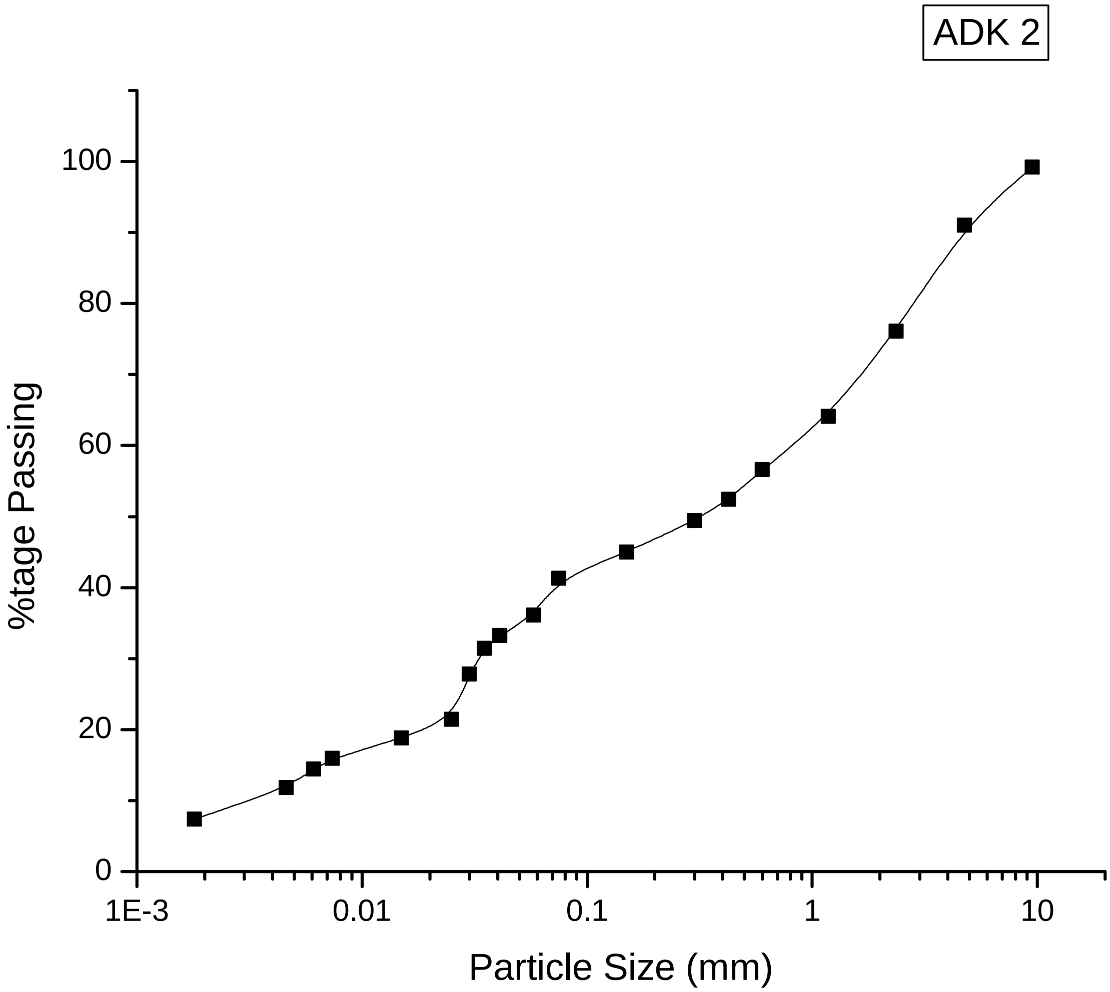

OS 7A
Dry Density (Kg/m³)
1780
1760
1740
1720
1700
1680
1660
1640
1620
10
12
14
16
18
20
Moisture Content

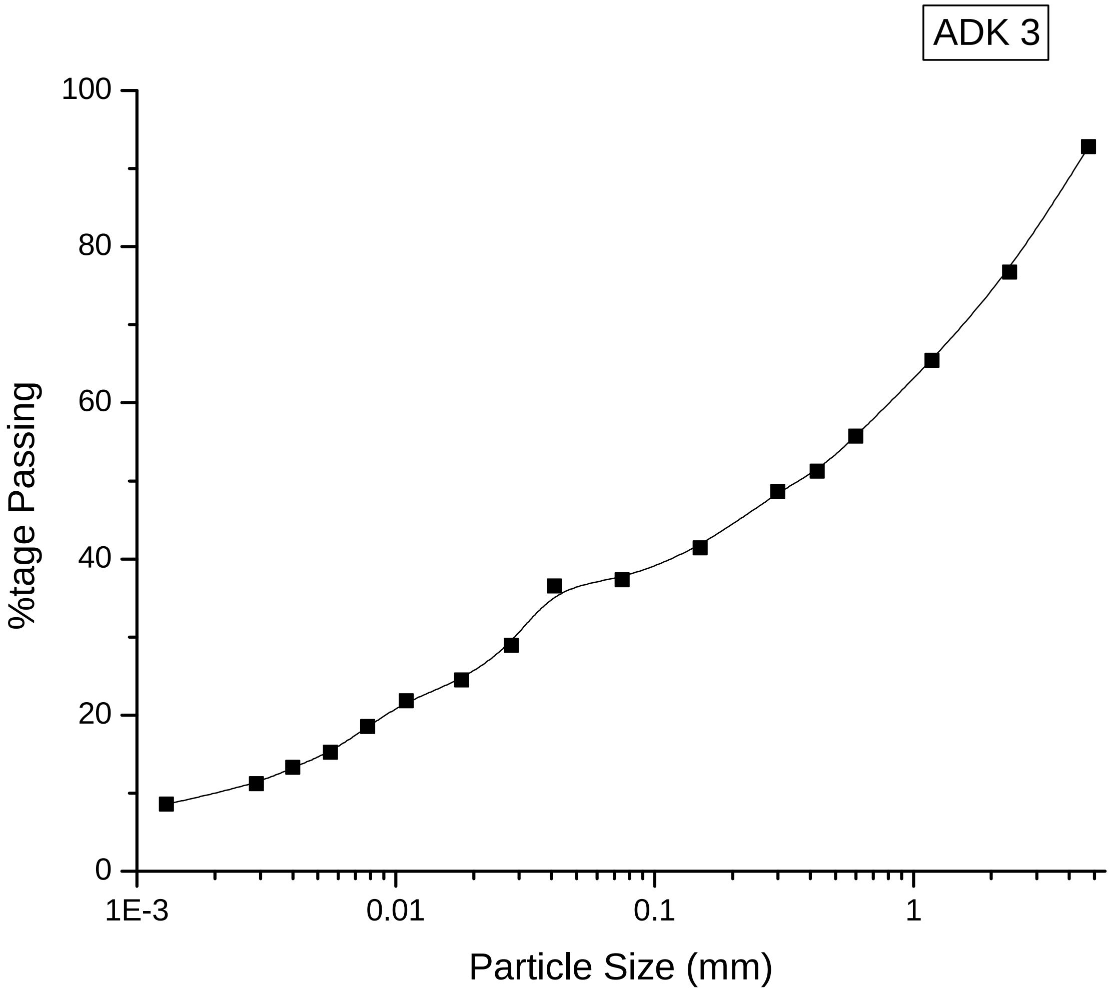

OS 7B
1800
1750
1700
1650
1600
Dry Density (Kg/m3)
4
6
8
10
12
14
16
Moisture Content

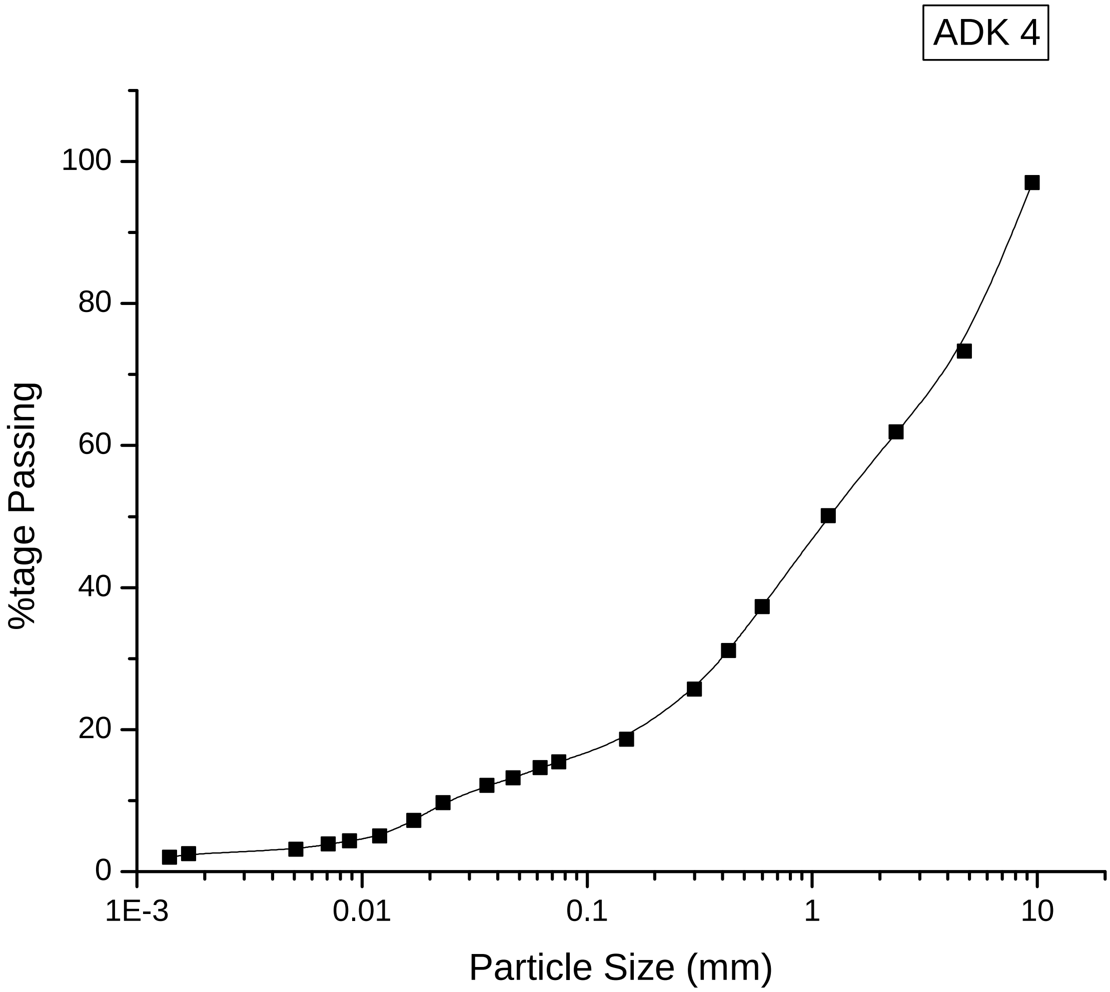
B-3 Grain Size Distribution Curves

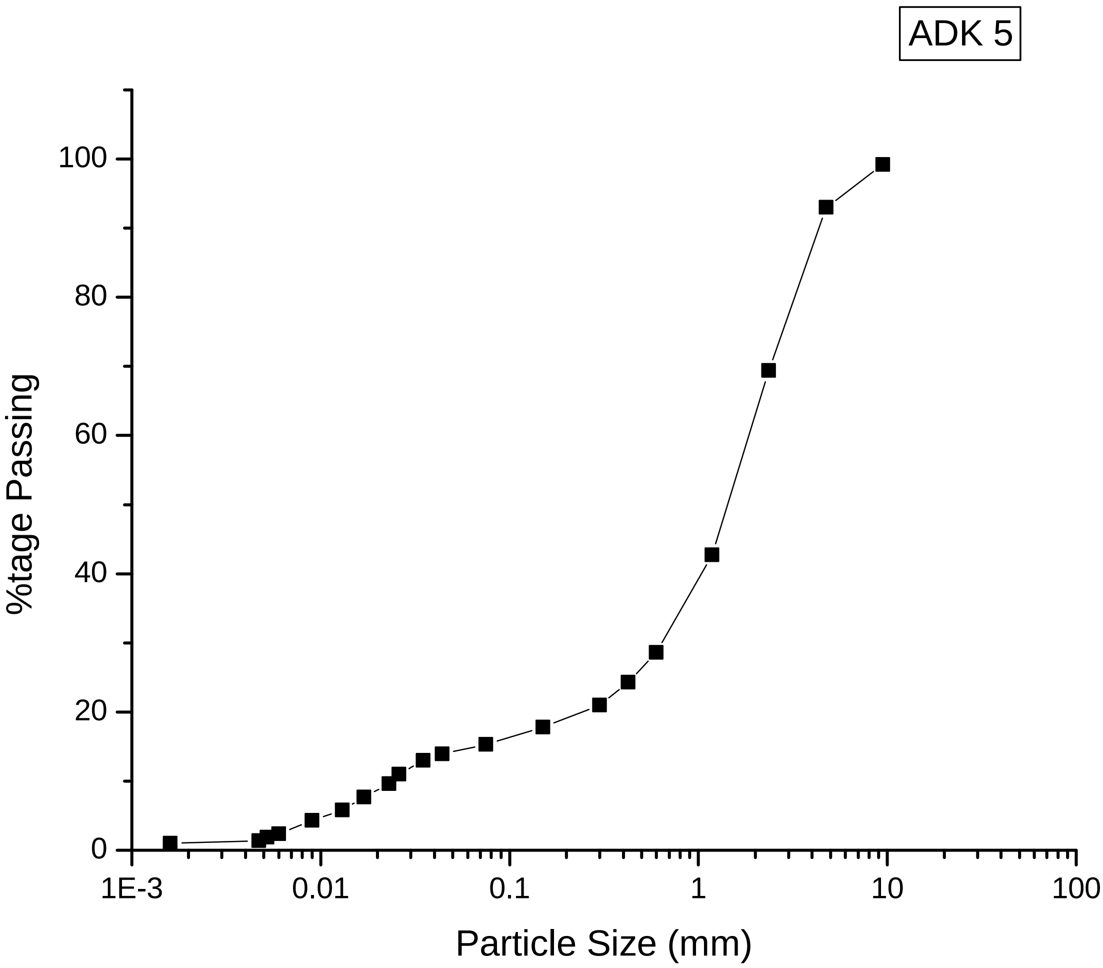


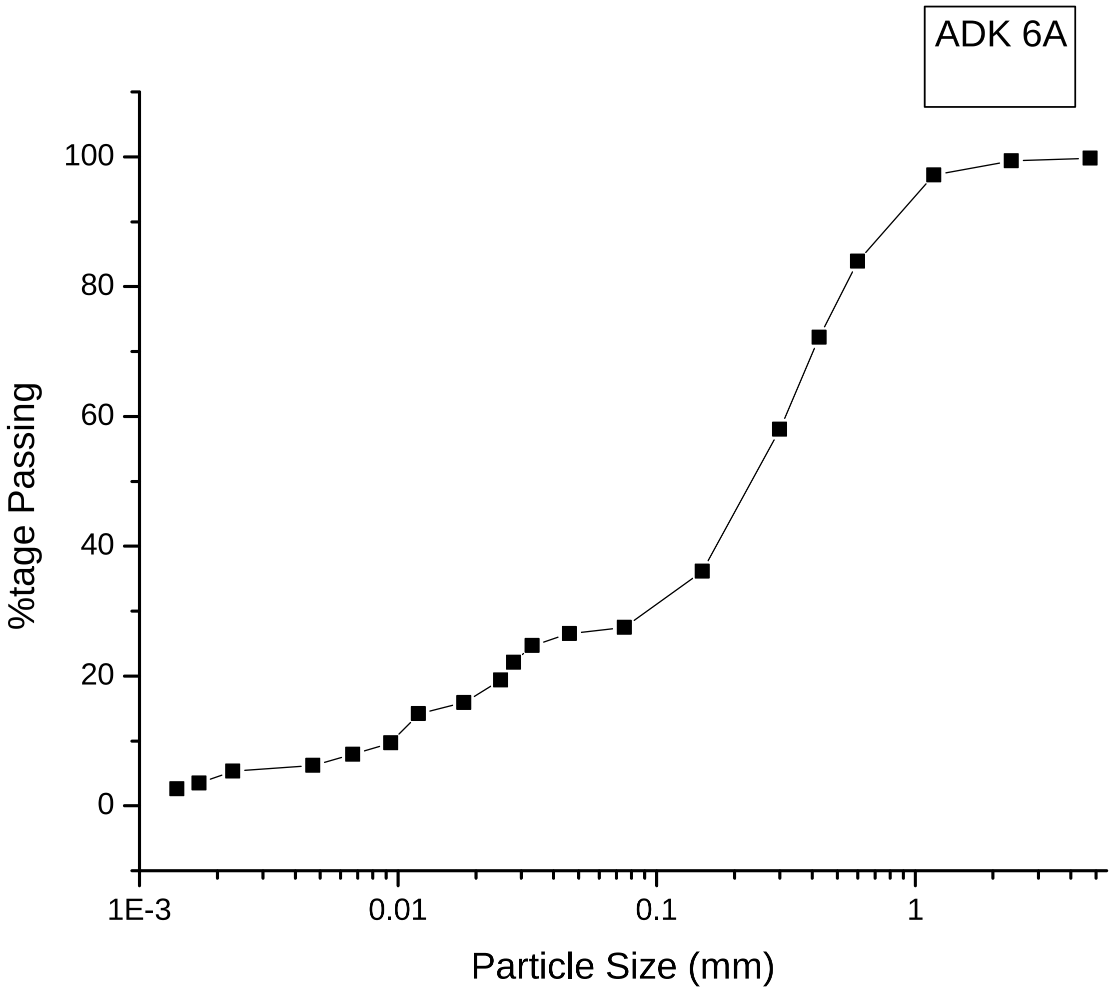

ADK 1B
%tage Passing
0
20
40
60
80
100
1E-3
0.01
0.1
1
10
Particle Size (mm)

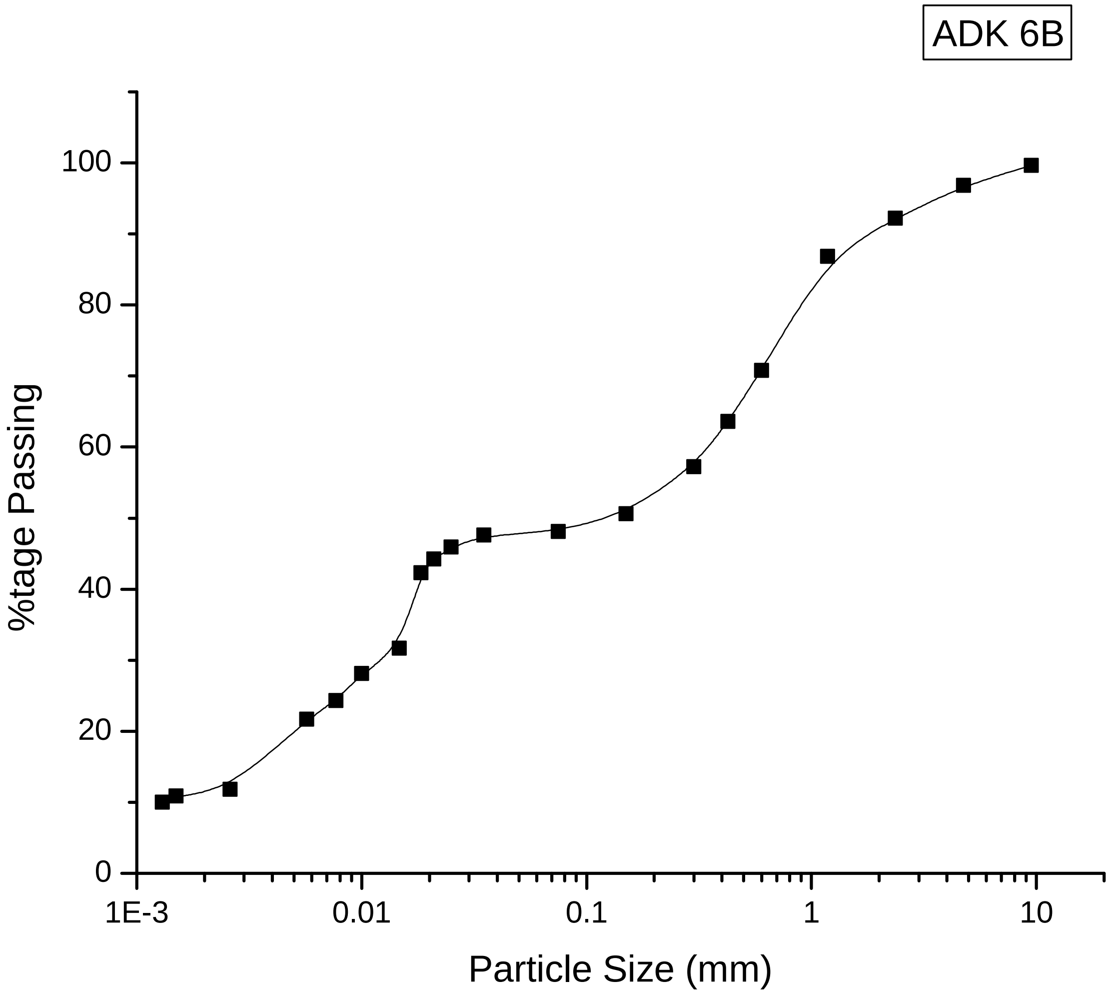

ADK 2
%tage Passing
0
20
40
60
80
100
1E-3
0.01
0.1
1
10
Particle Size (mm)

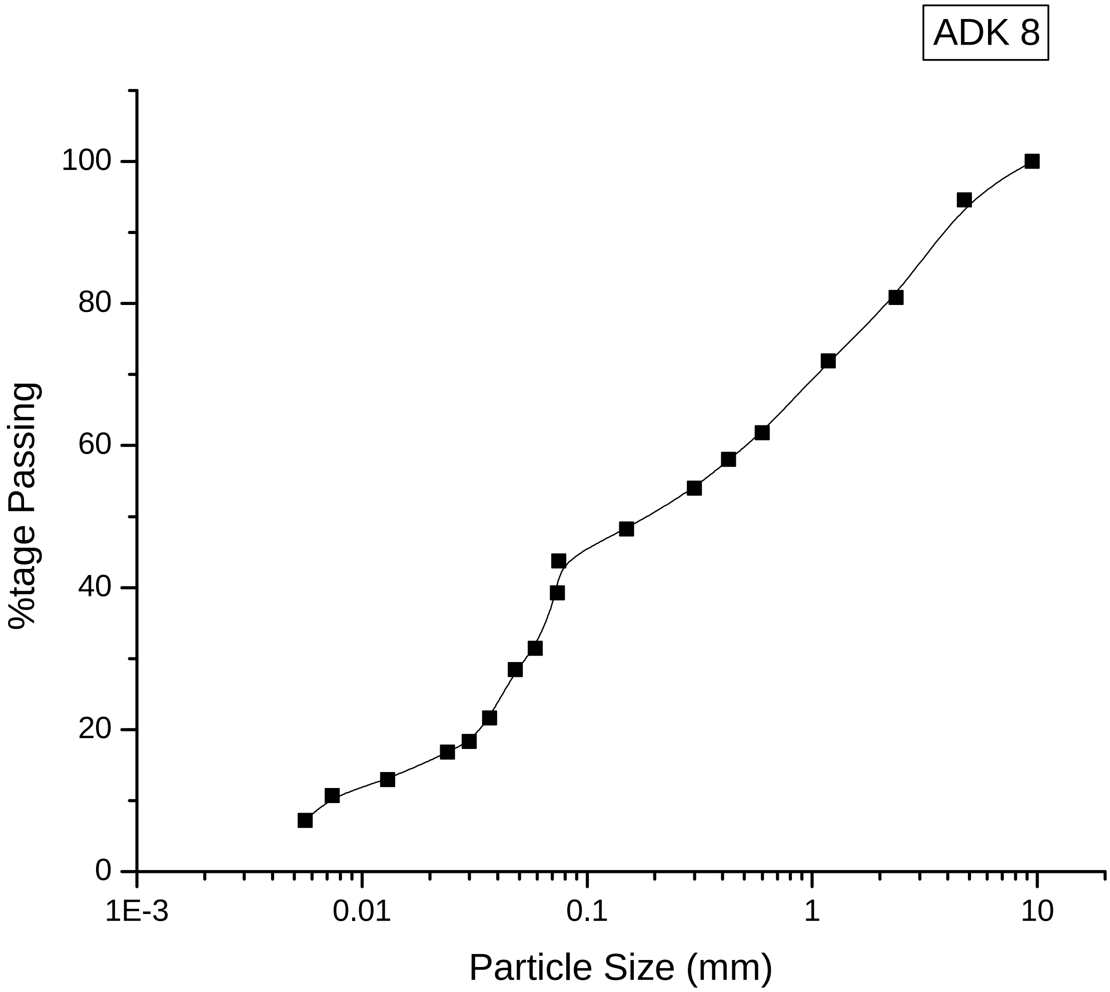

ADK 3
100
80
60
40
20
0
%tage Passing
1E-3
0.01
0.1
1
Particle Size (mm)

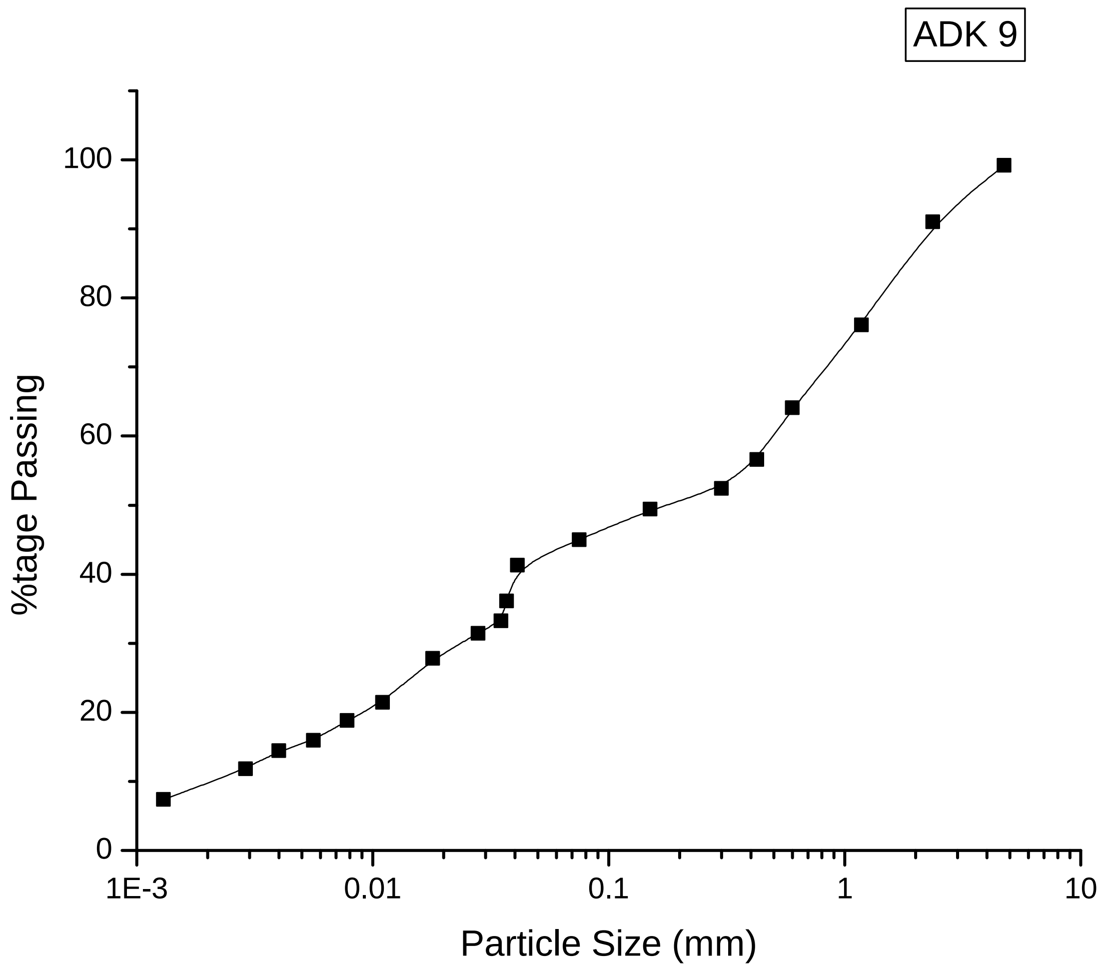

ADK 4
%tage Passing
0
20
40
60
80
100
1E-3
0.01
0.1
1
10
Particle Size (mm)

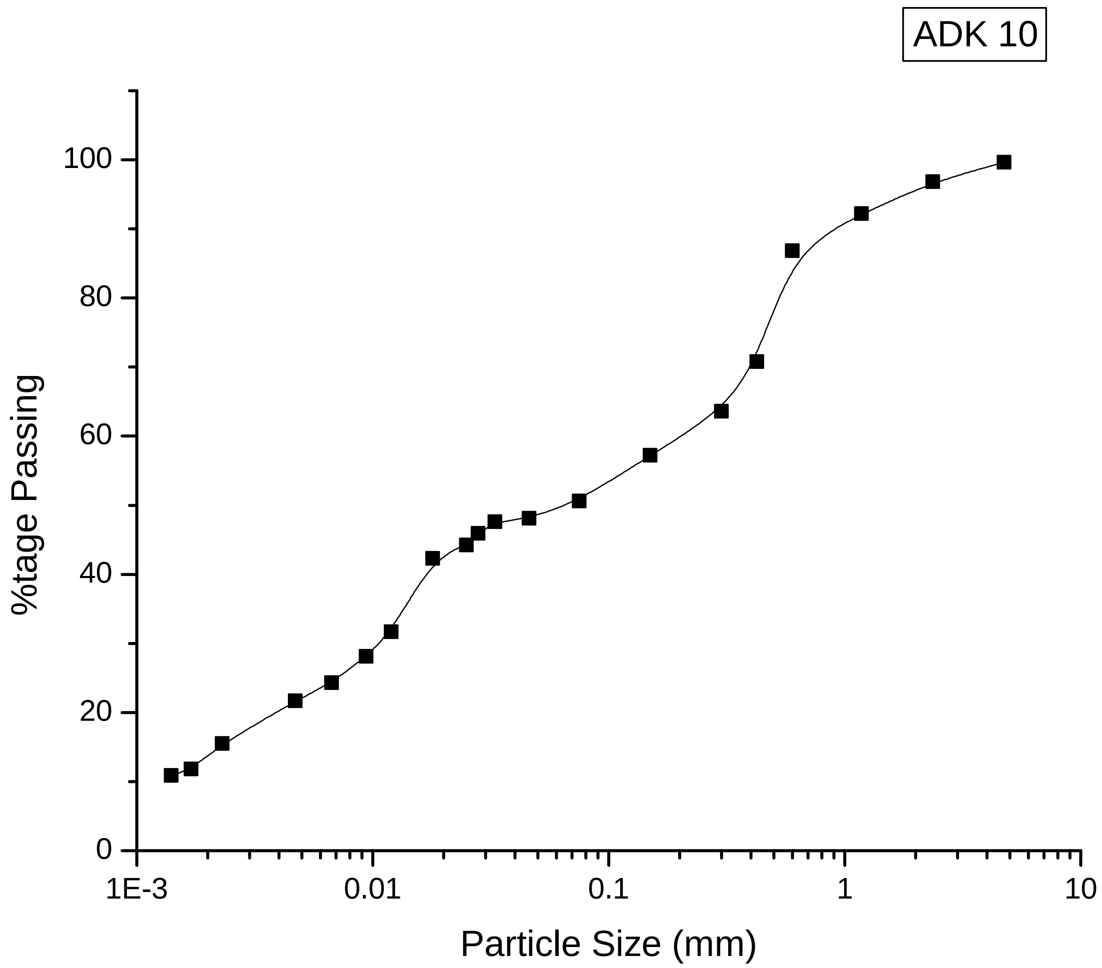

ADK 5
100
80
60
40
20
0
%tage Passing
1E-3
0.01
0.1
1
10
100
Particle Size (mm)

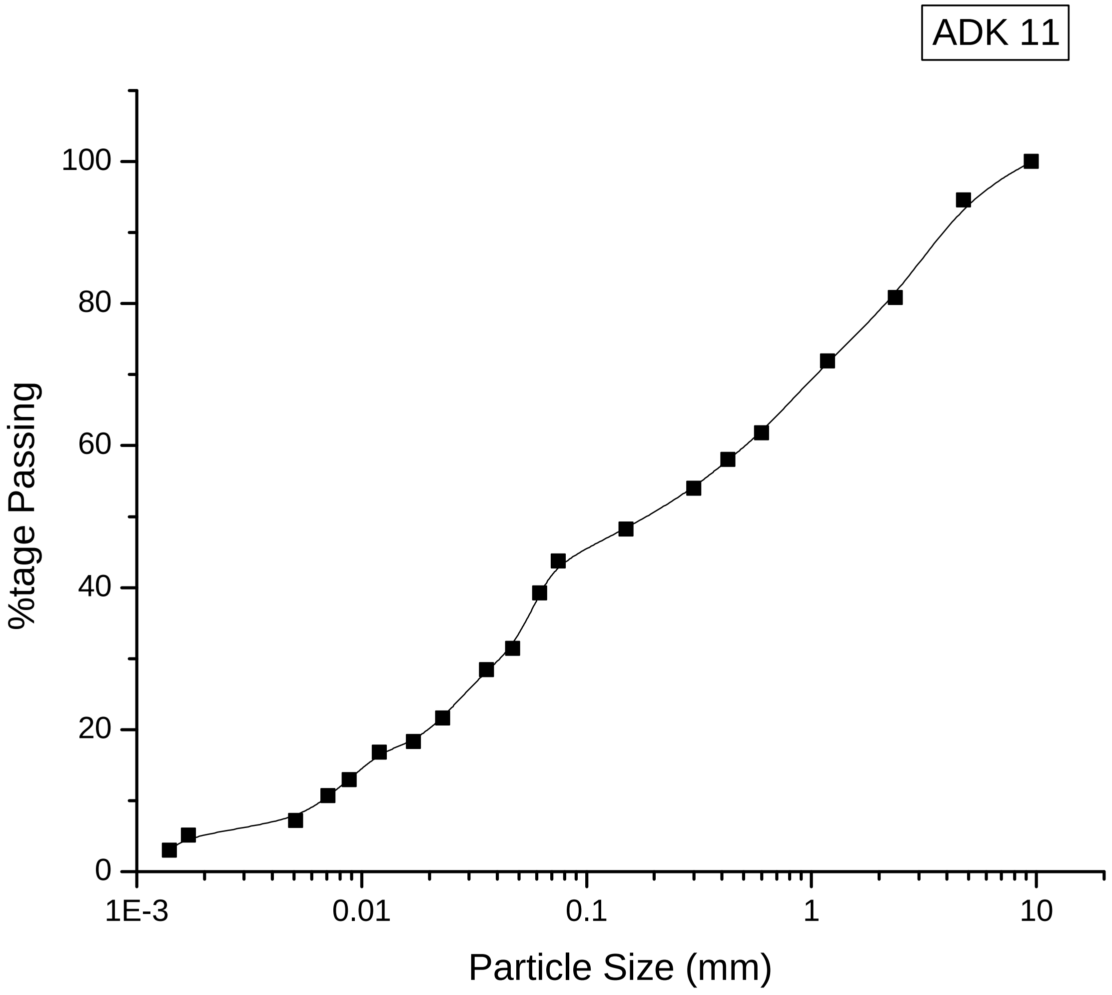

ADK 6A
%tage Passing
100
80
60
40
20
0
1E-3
0.01
0.1
1
Particle Size (mm)

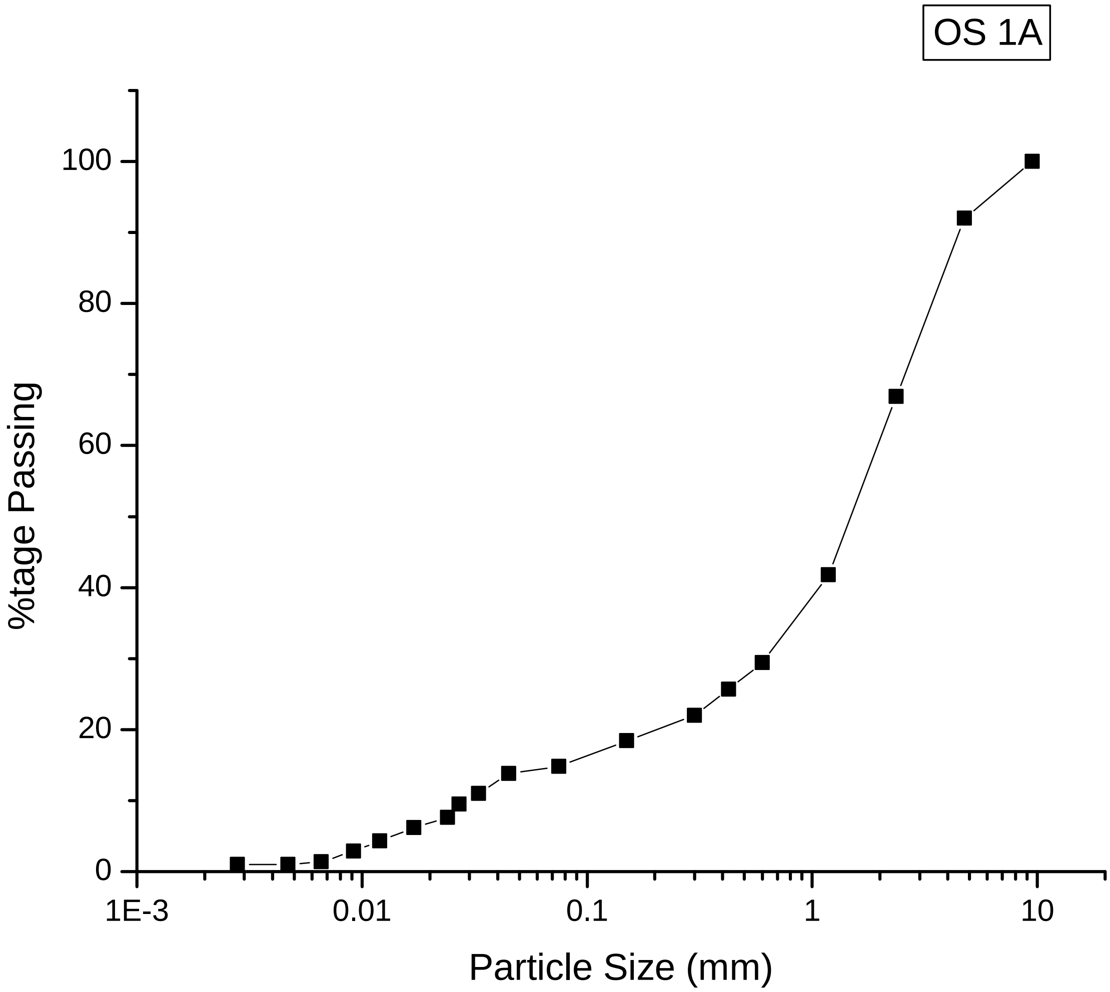

ADK 6B
%tage Passing
0
20
40
60
80
100
1E-3
0.01
0.1
1
10
Particle Size (mm)

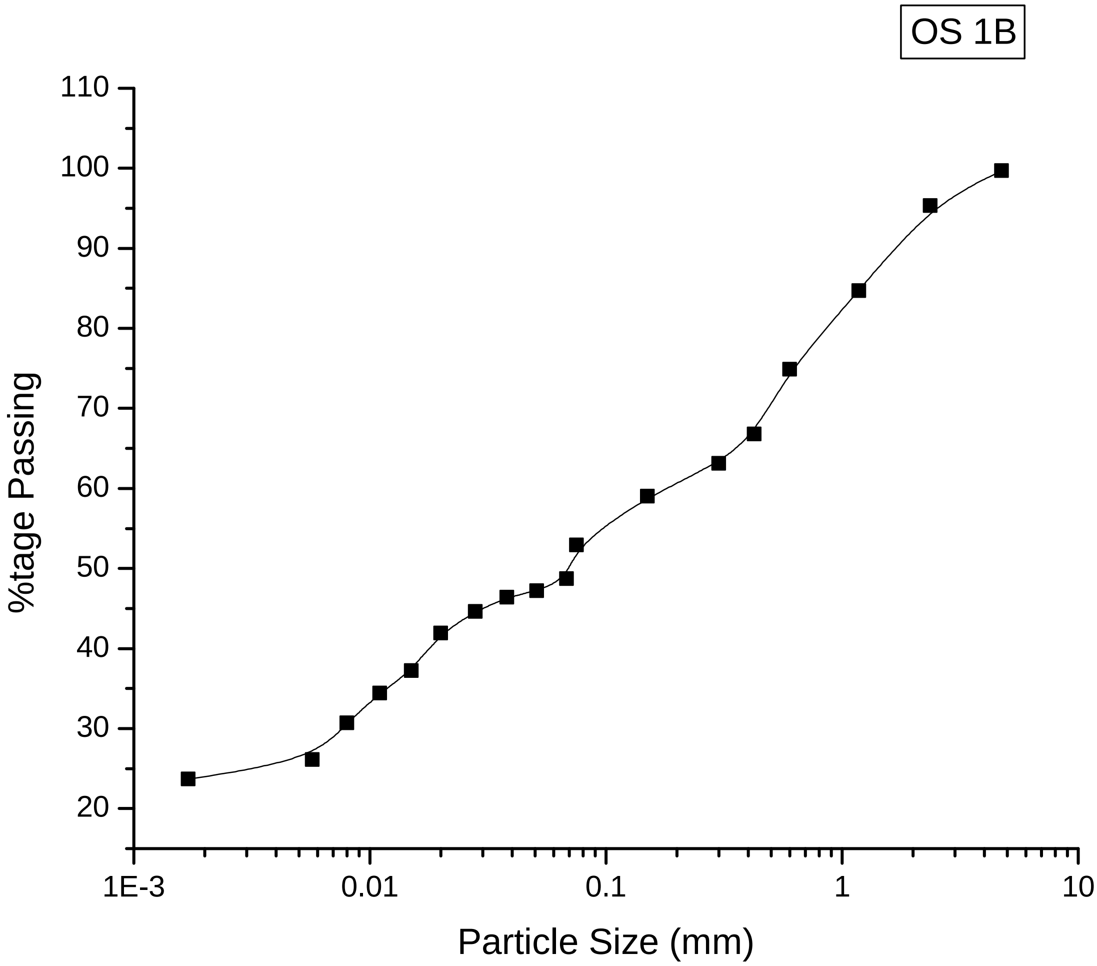

ADK 8
%tage Passing
100
80
60
40
20
0
1E-3
0.01
0.1
1
10
Particle Size (mm)

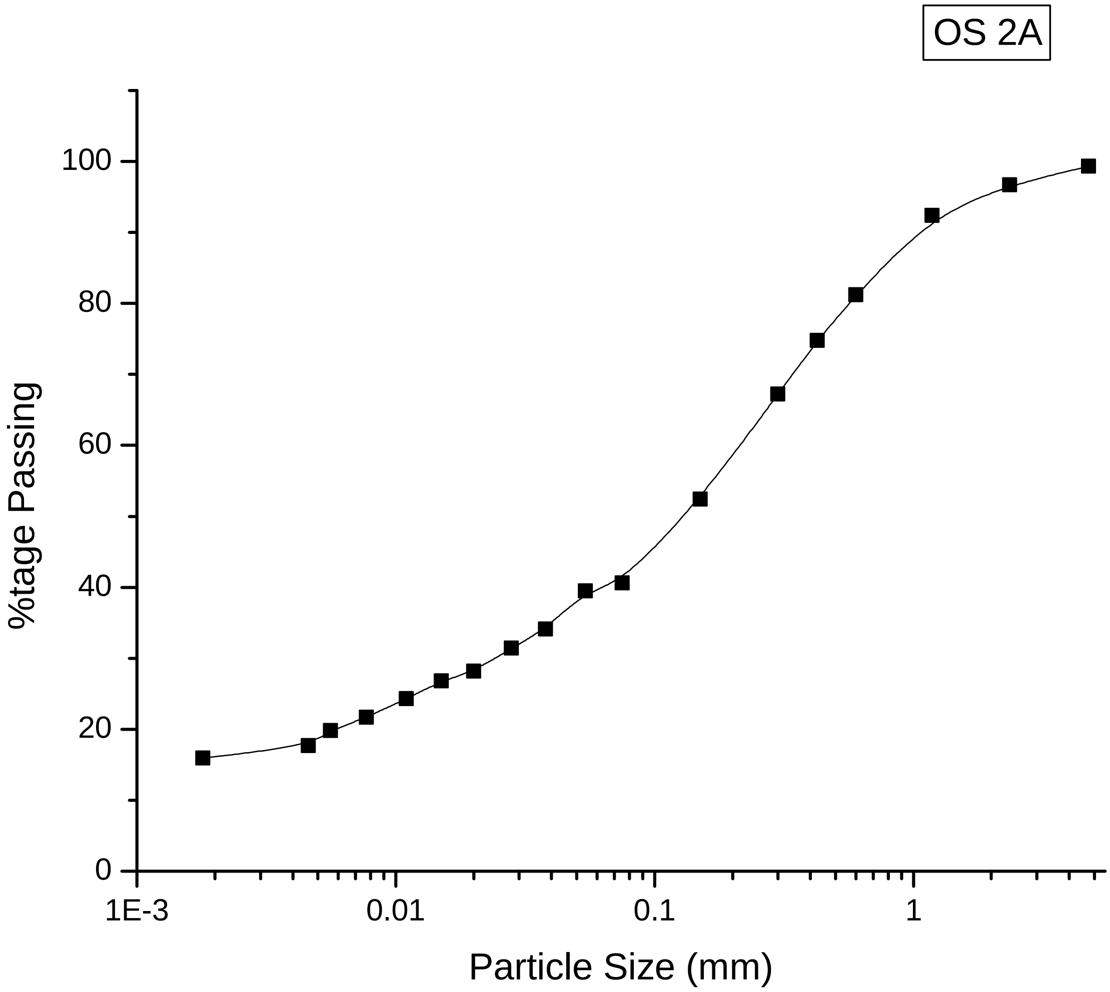

ADK 9
100
80
60
40
20
0
%tage Passing
1E-3
0.01
0.1
1
10
Particle Size (mm)

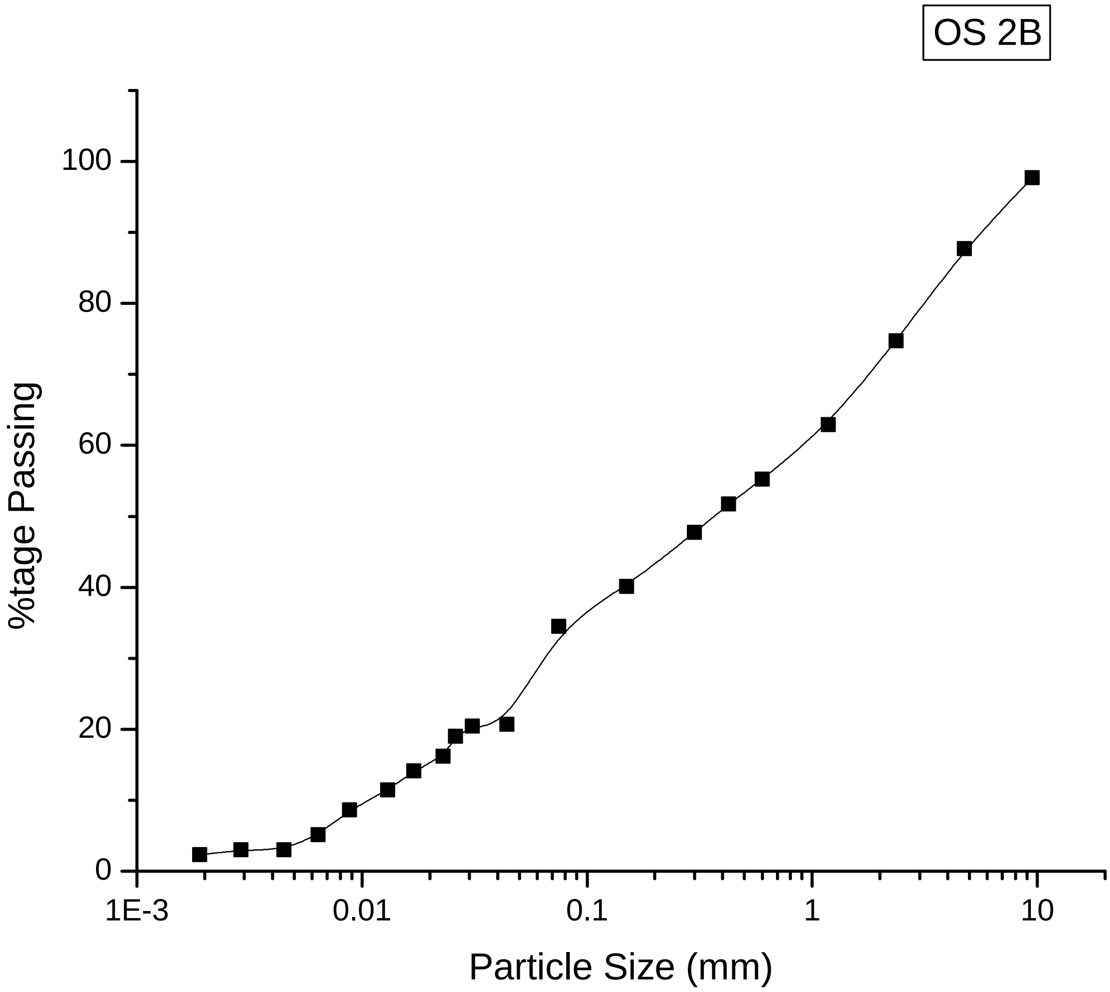

ADK 10
%tage Passing
0
20
40
60
80
100
1E-3
0.01
0.1
1
10
Particle Size (mm)

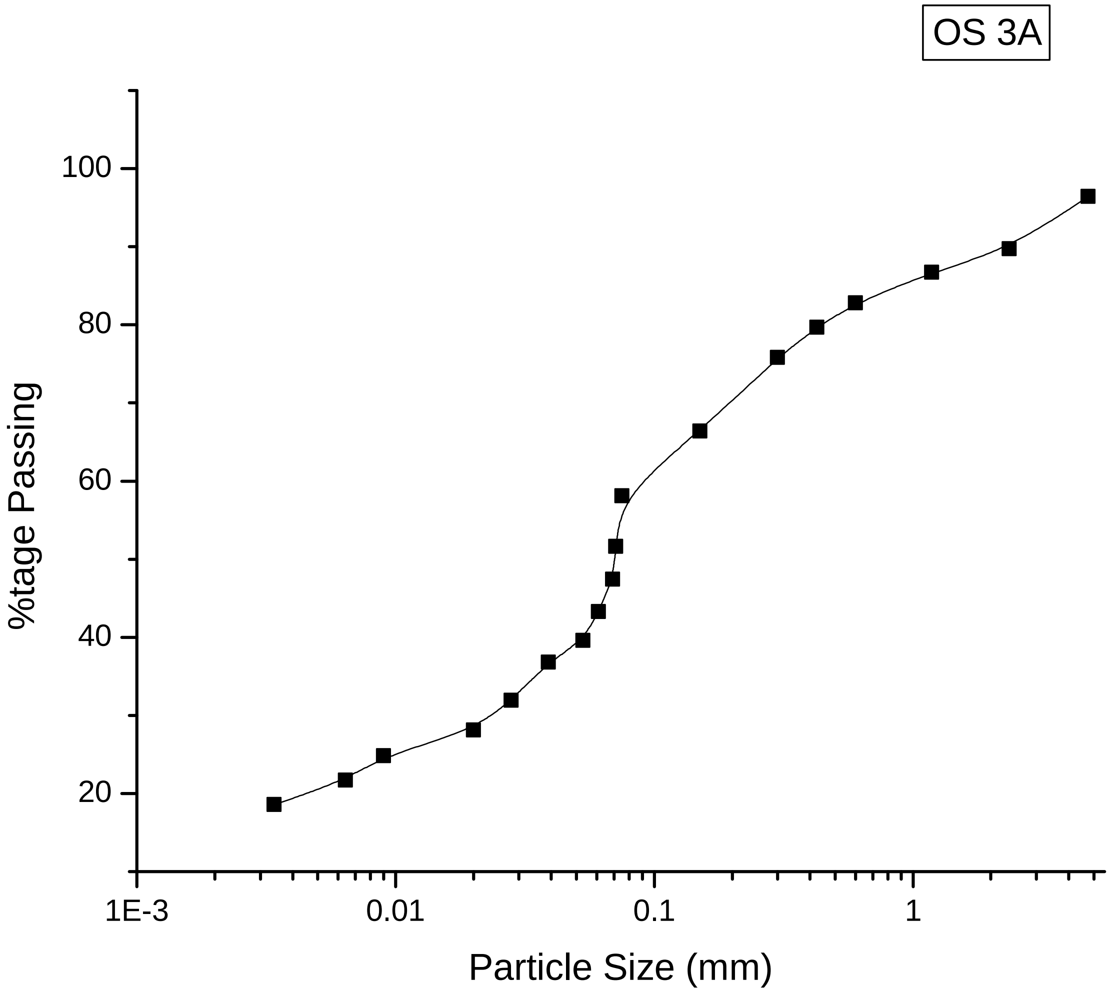

ADK 11
%tage Passing
100
80
60
40
20
0
1E-3
0.01
0.1
1
10
Particle Size (mm)

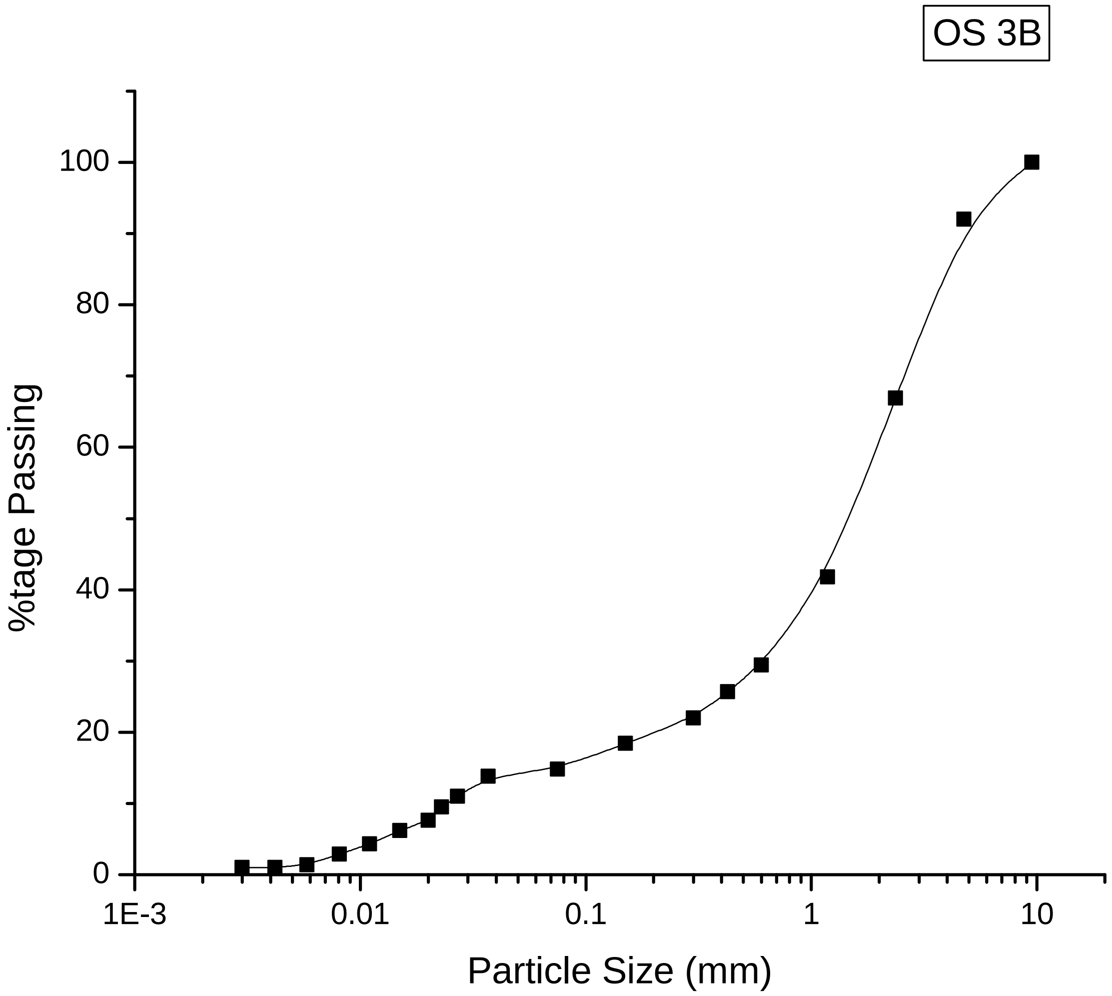

OS 1A
%tage Passing
0
20
40
60
80
100
1E-3
0.01
0.1
1
10
Particle Size (mm)

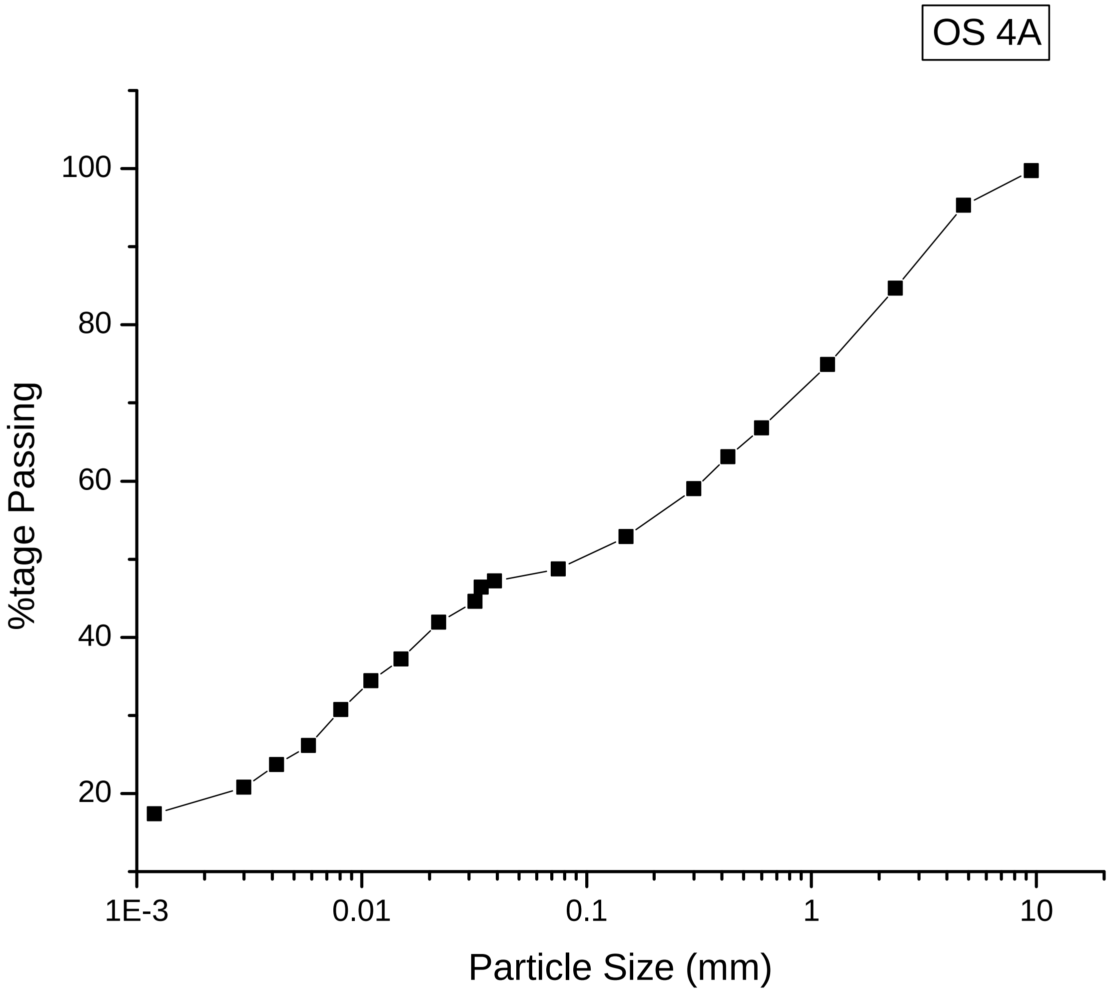

OS 1B
%tage Passing
110
100
90
80
70
60
50
40
30
20
1E-3
0.01
0.1
1
10
Particle Size (mm)

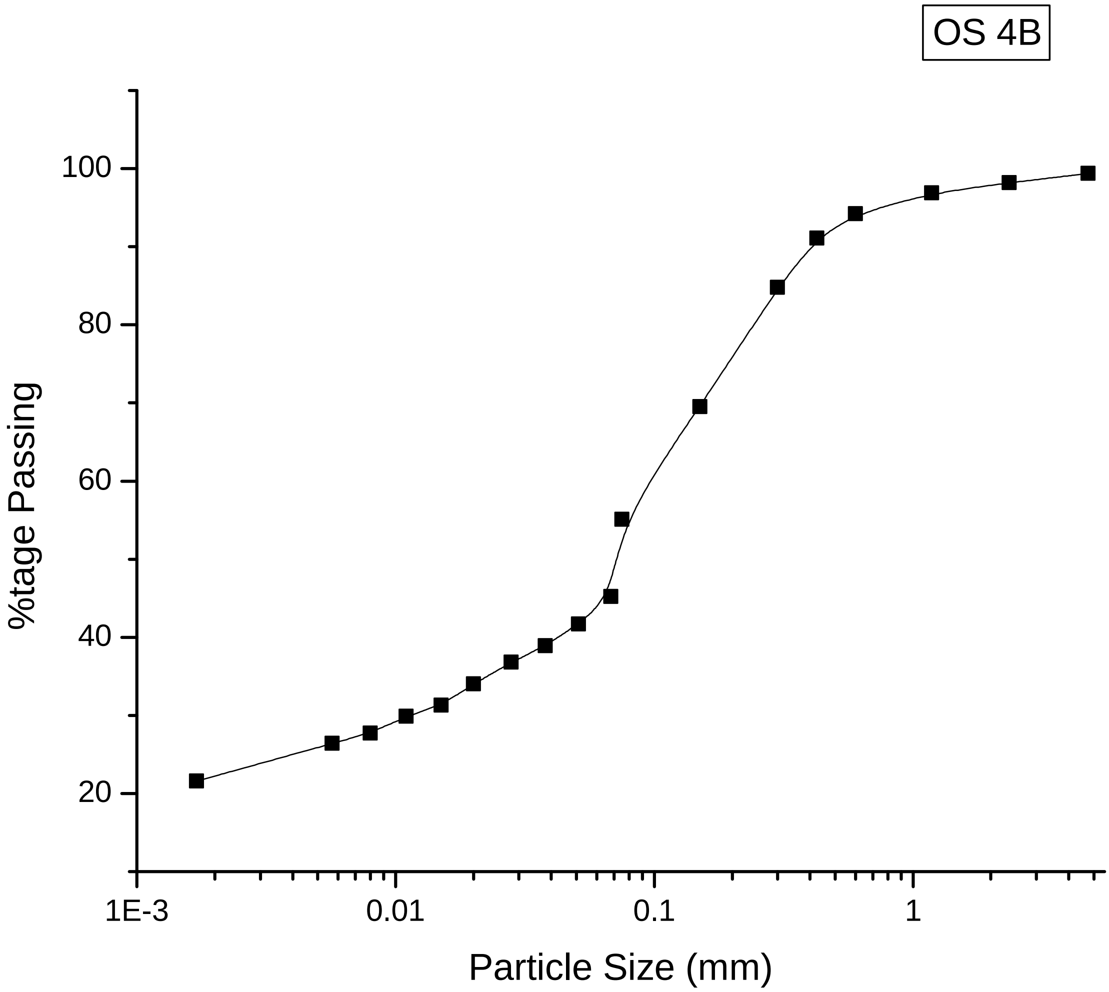

OS 2A
%tage Passing
0
20
40
60
80
100
1E-3
0.01
0.1
1
Particle Size (mm)

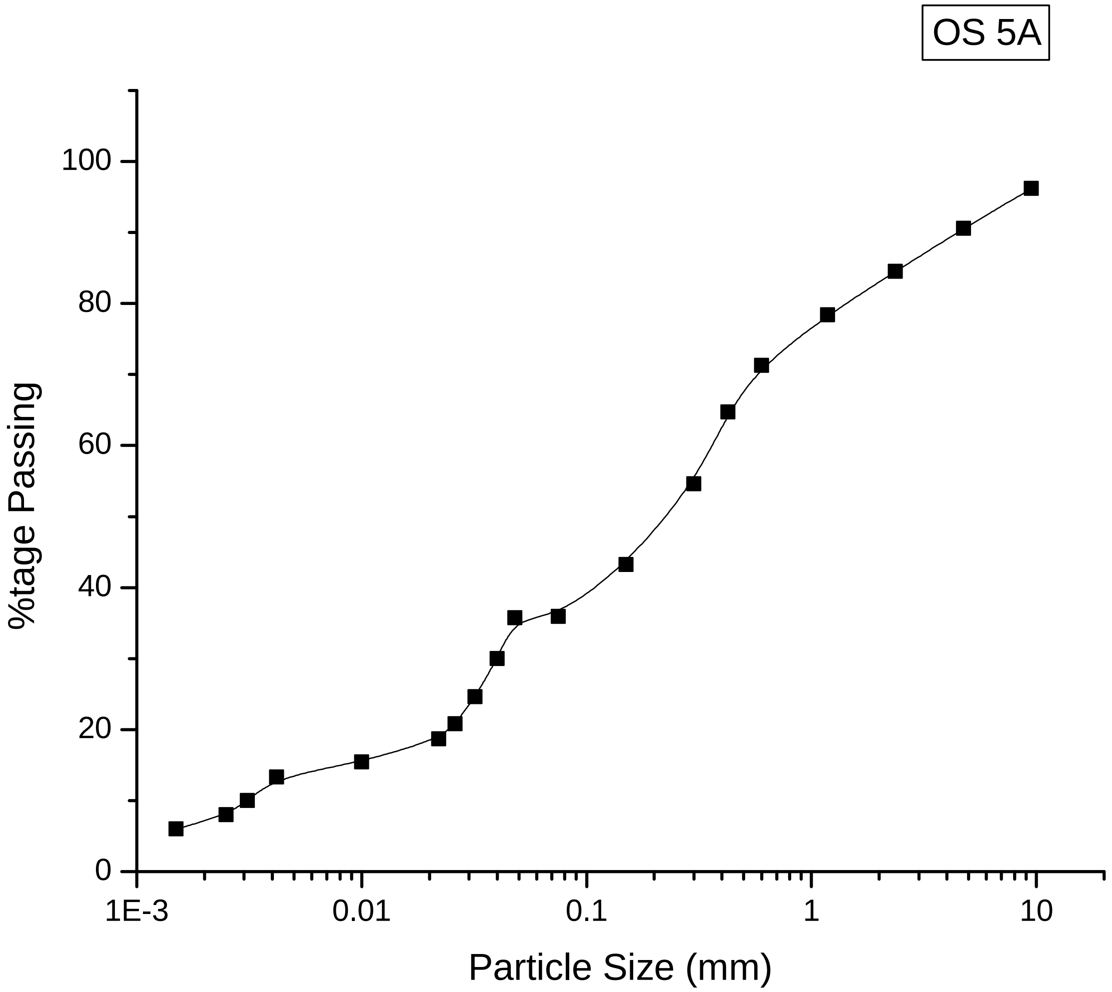

OS 2B
%tage Passing
100
80
60
40
20
0
1E-3
0.01
0.1
1
10
Particle Size (mm)

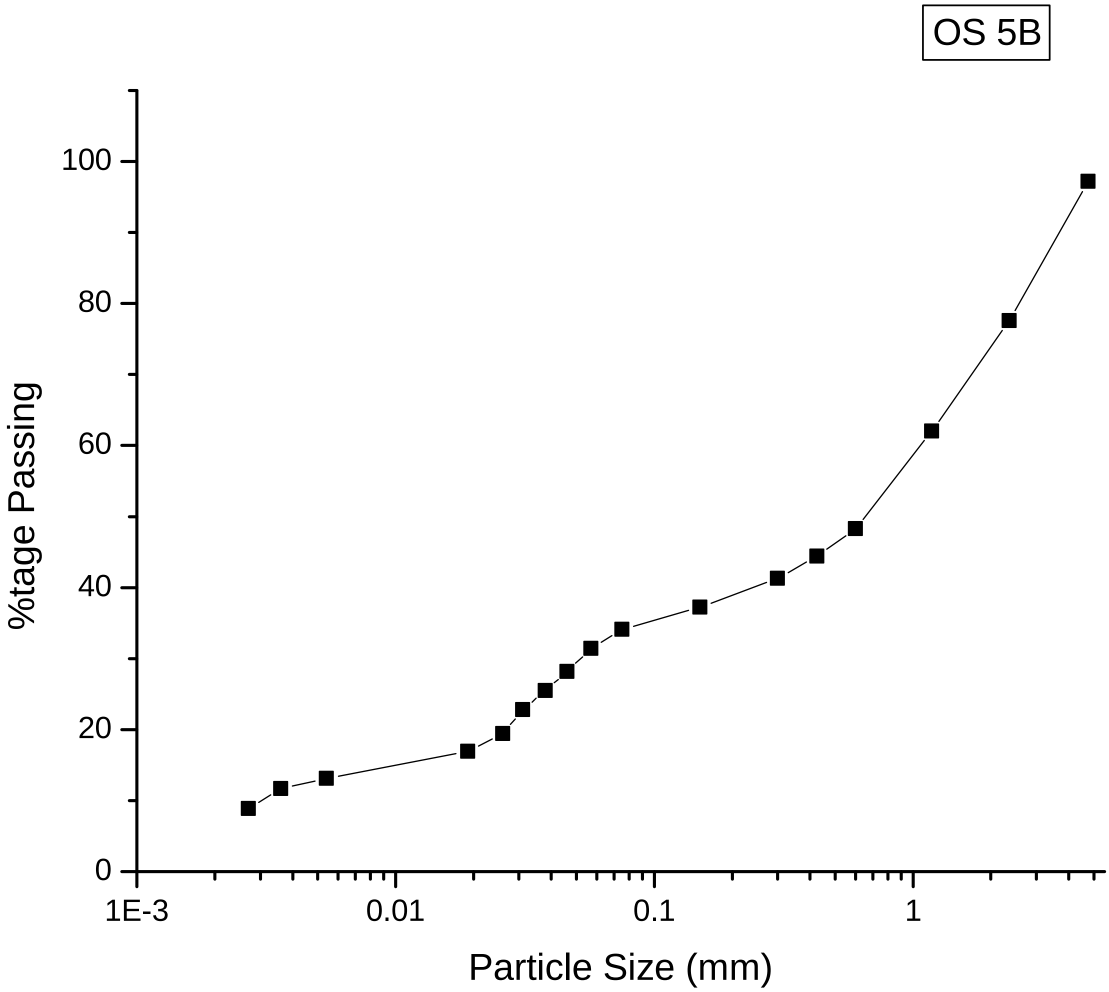

OS 3A
%tage Passing
100
80
60
40
20
1E-3
0.01
0.1
1
Particle Size (mm)

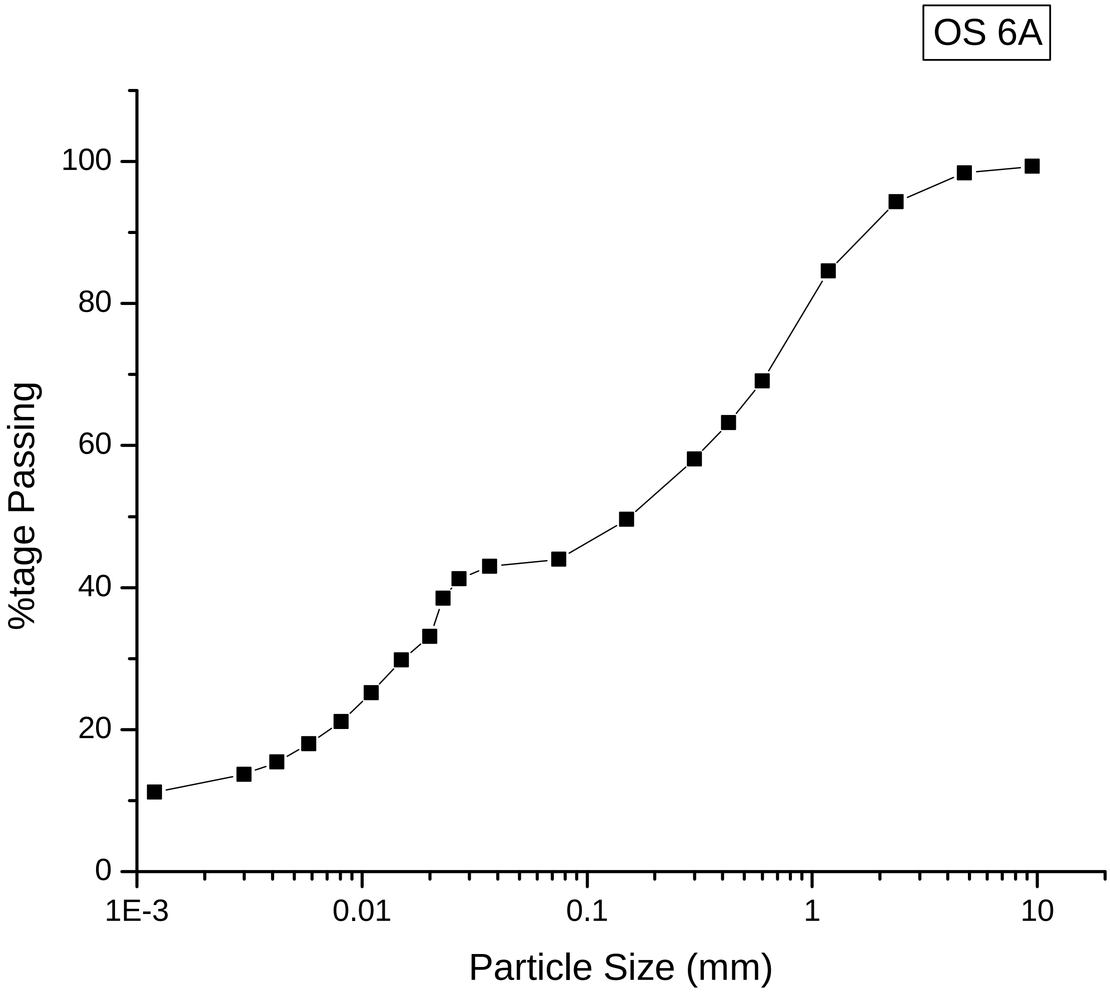

OS 3B
%tage Passing
0
20
40
60
80
100
1E-3
0.01
0.1
1
10
Particle Size (mm)

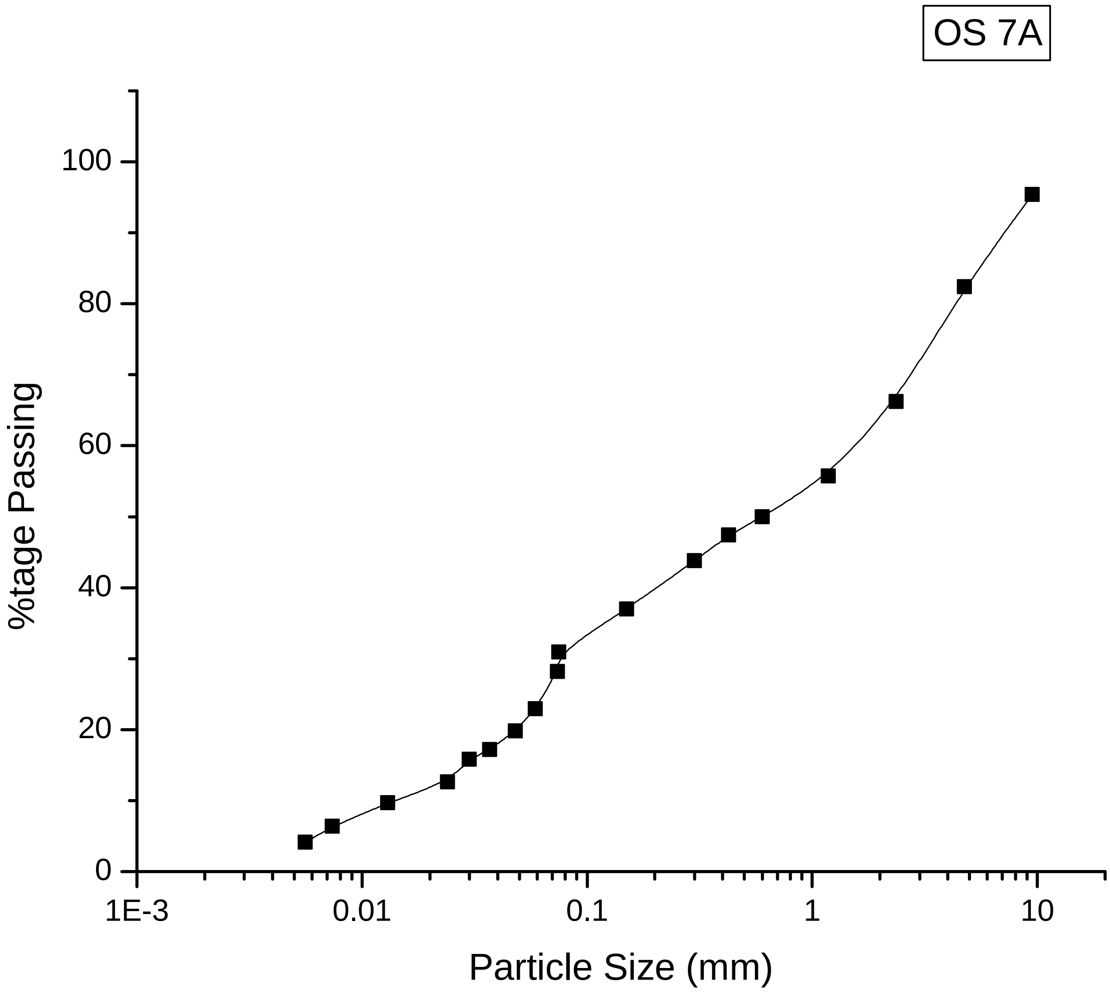

OS 4A
%tage Passing
100
80
60
40
20
1E-3
0.01
0.1
1
10
Particle Size (mm)


OS 4B
%tage Passing
100
80
60
40
20
1E-3
0.01
0.1
1
Particle Size (mm)

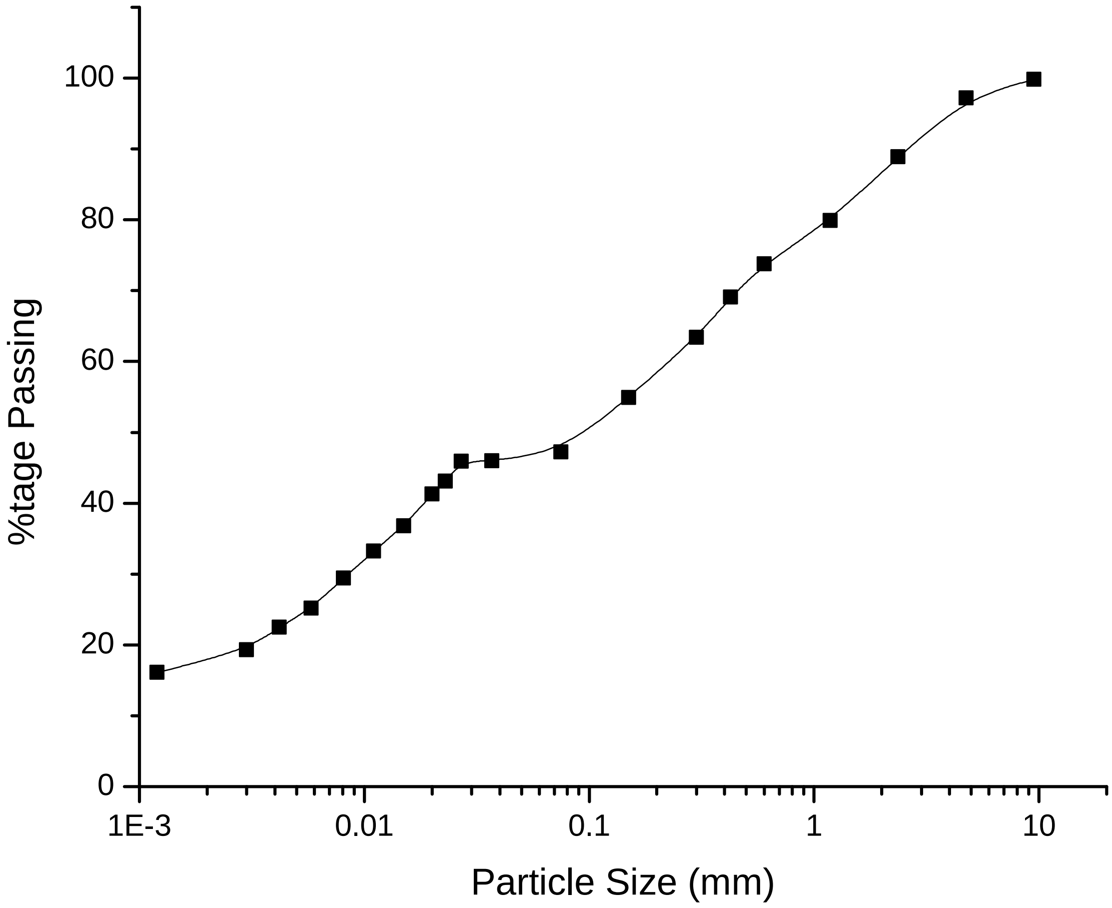

OS 5A
%tage Passing
0
20
40
60
80
100
1E-3
0.01
0.1
1
10
Particle Size (mm)


OS 5B
%tage Passing
0
20
40
60
80
100
1E-3
0.01
0.1
1
Particle Size (mm)


OS 6A
%tage Passing
0
20
40
60
80
100
1E-3
0.01
0.1
1
10
Particle Size (mm)

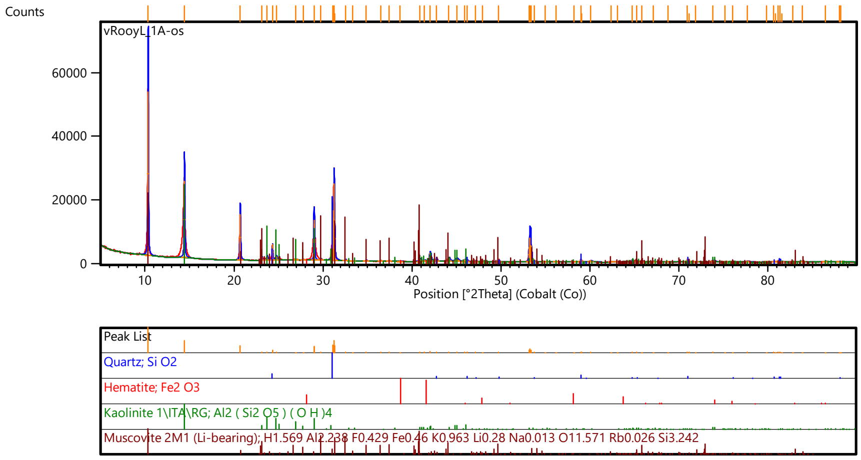

OS 7A
%tage Passing
0
20
40
60
80
100
1E-3
0.01
0.1
1
10
Particle Size (mm)

OS 7B

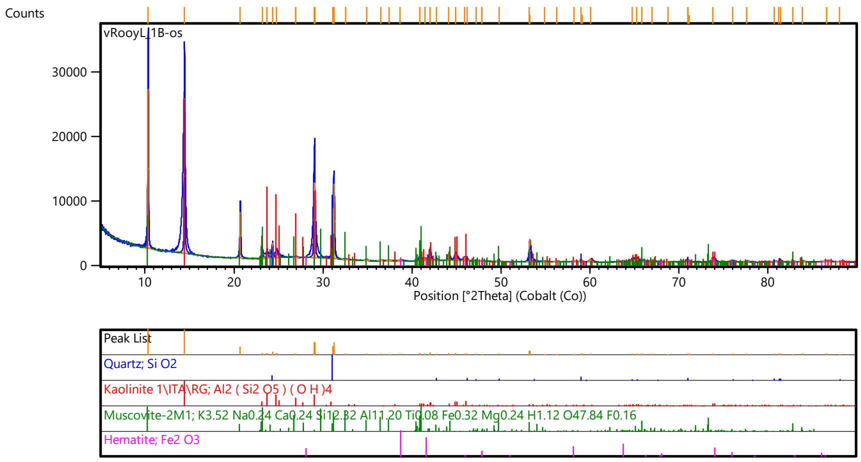


B-4 X-RAY DIFFRACTOGRAMS

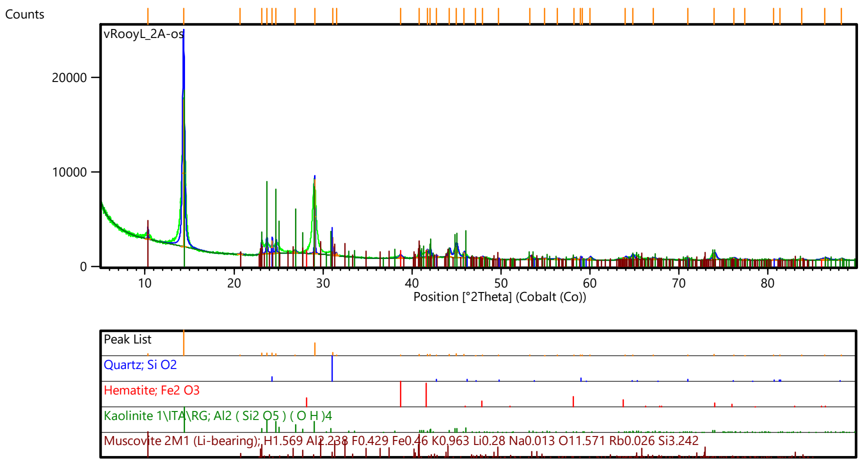
OS 1A



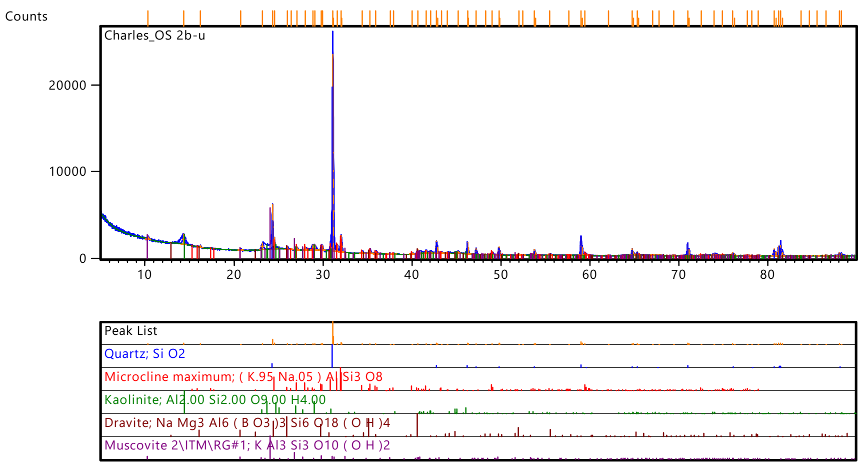
OS 1B



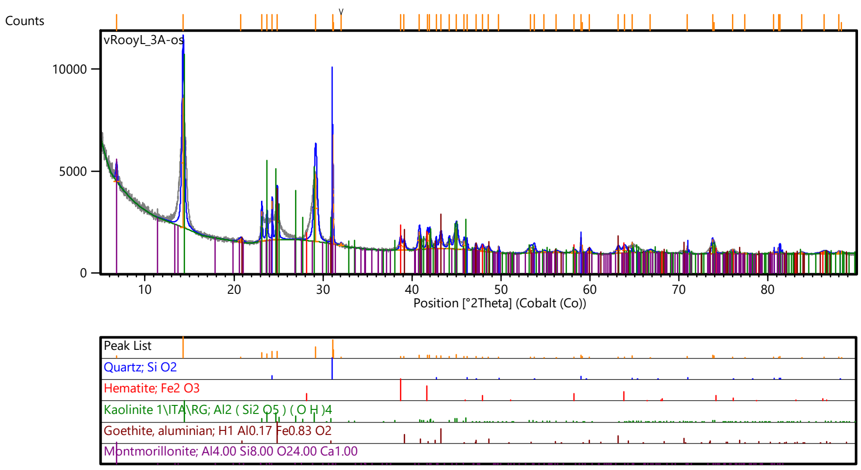
OS 2A



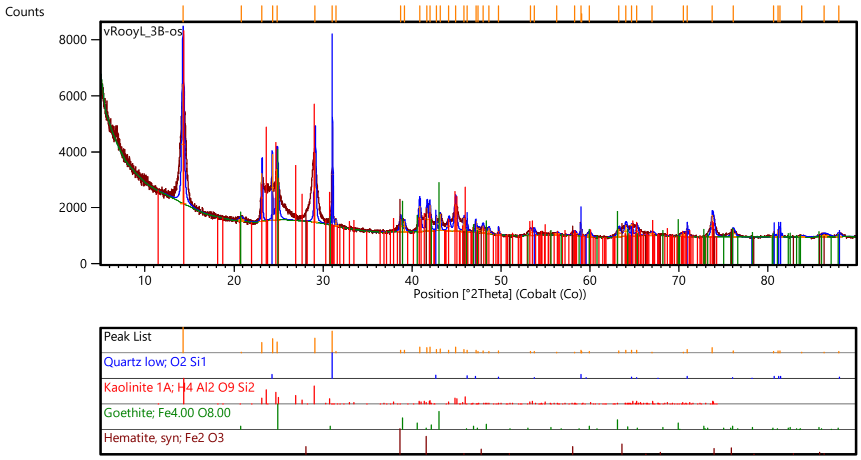
OS 2B



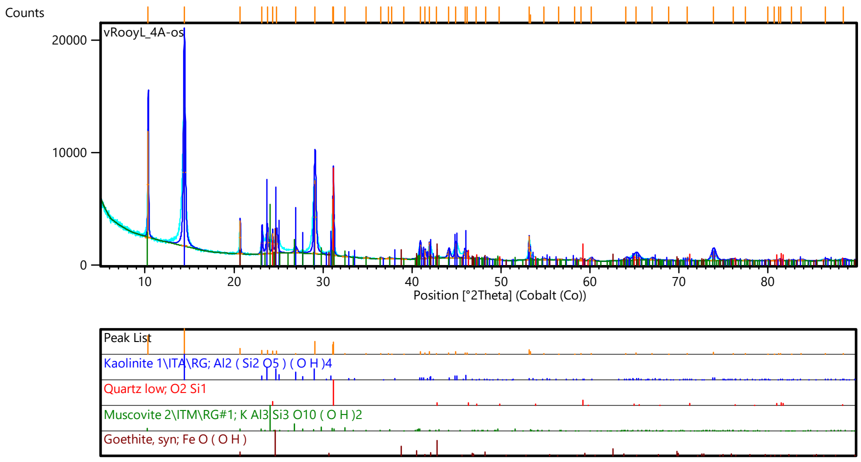
OS 3A



OS 3B


Counts
8000
6000
4000
2000
0
vRooyL_3B-os
10
20
30
40
50
60
70
80
Position [°2Theta] (Cobalt (Co))
Peak List
Quartz low; O2 Si1
Kaolinite 1A; H4 Al2 O9 Si2
Goethite; Fe4.00 O8.00
Hematite, syn; Fe2 O3

OS 4A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20000
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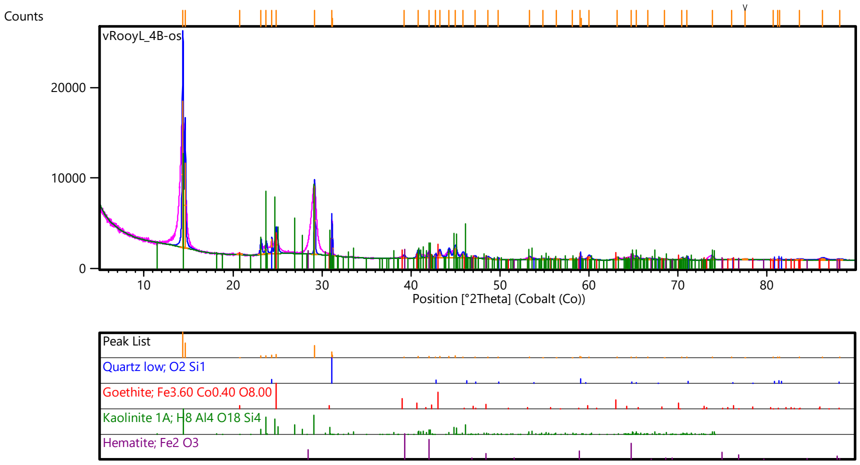
0
vRooyL_4A-os
10
20
30
40
50
60
70
80
Position [°2Theta] (Cobalt (Co))
Peak List
Kaolinite 1\ITA\RG; Al2 (Si2 O5) (O H)4
Quartz low; O2 Si1
Muscovite 2\ITM\RG#1; K Al3 Si3 O10 (O H)2
Goethite, syn; Fe O (O H)

OS 4B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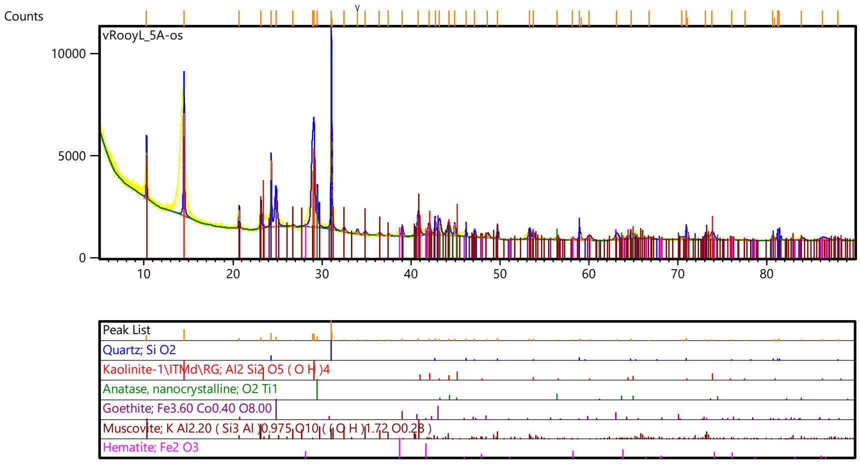
vRooyL_4B-os
20000
10000
0
10
20
30
40
50
60
70
80
Position [°2Theta] (Cobalt (Co))
Peak List
Quartz low; O2 Si1
Goethite; Fe3.60 Co0.40 O8.00
Kaolinite 1A; H8 Al4 O18 Si4
Hematite; Fe2 O3

OS 5A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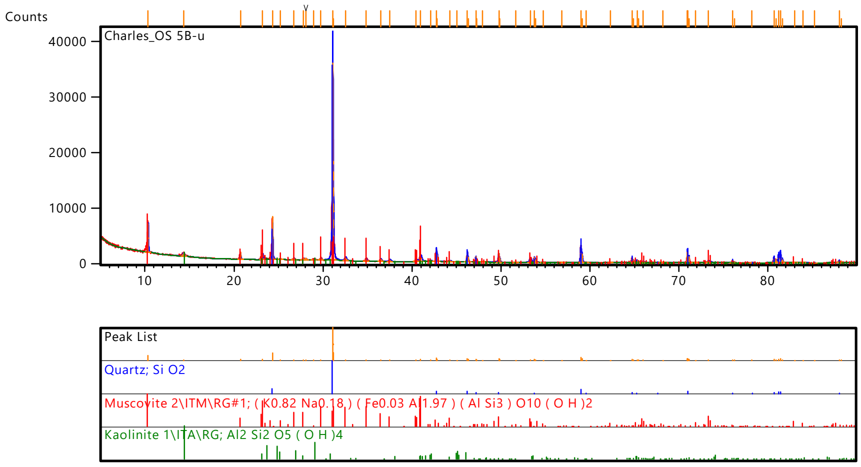
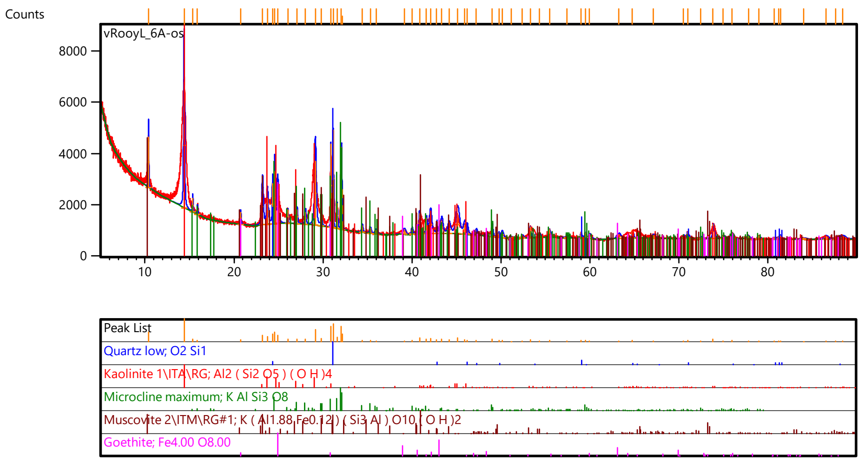
vRooyL_5A-os
10000
5000
0
10
20
30
40
50
60
70
80
Peak List
Quartz; Si O2
Kaolinite-1\ITMd\RG; Al2 Si2 O5 (O H)4
Anatase, nanocrystalline; O2 Ti1
Goethite; Fe3.60 Co0.40 O8.00
Muscovite; K Al2.20 (Si3 Al)0.975 O10 ((O H)1.72 O0.28)
Hematite; Fe2 O3

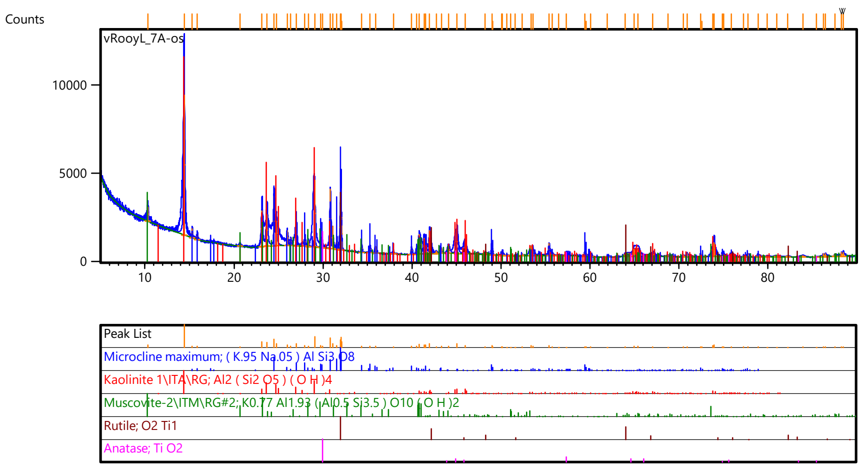
OS 5B



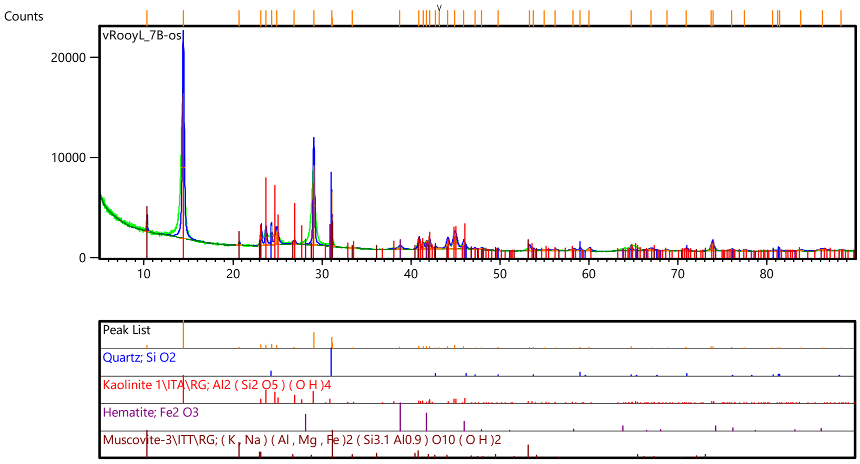
OS 6A



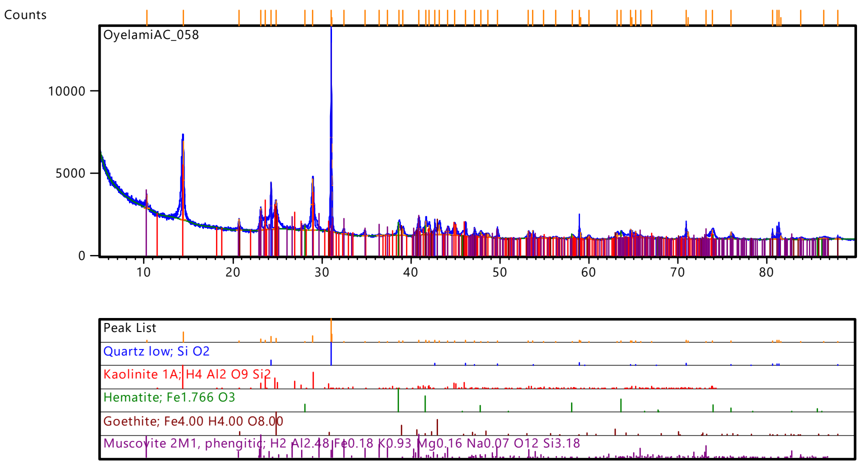
OS 7A



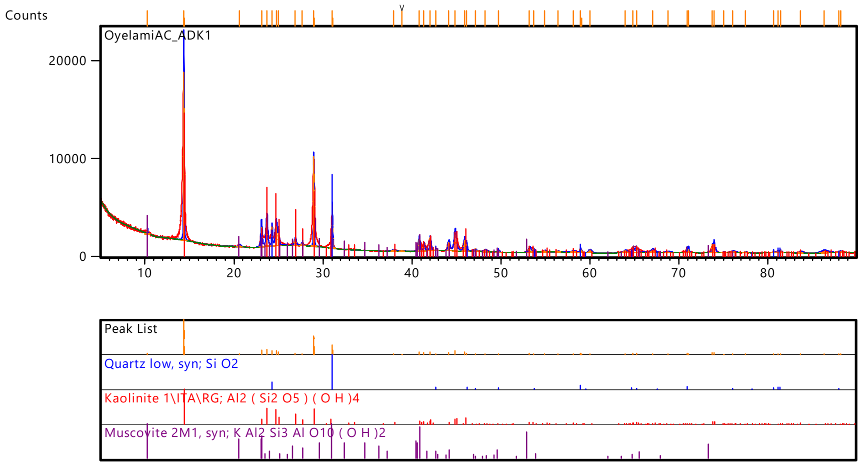
OS 7B


Counts
vRooyL_7B-os
20000
10000
0
10
20
30
40
50
60
70
80
Peak List
Quartz; Si O2
Kaolinite 1\ITA\RG; Al2 (Si2 O5) (O H)4
Hematite; Fe2 O3
Muscovite-3\ITT\RG; (K , Na) (Al , Mg , Fe)2 (Si3.1 Al0.9) O10 (O H)2

OS 8


Counts
OyelamiAC_058
10000
5000
0
10
20
30
40
50
60
70
80
Peak List
Quartz low; Si O2
Kaolinite 1A; H4 Al2 O9 Si2
Hematite; Fe1.766 O3
Goethite; Fe4.00 H4.00 O8.00
Muscovite 2M1, phengitic; H2 Al2.48 Fe0.18 K0.93 Mg0.16 Na0.07 O12 Si3.18

ADK 1A


Counts
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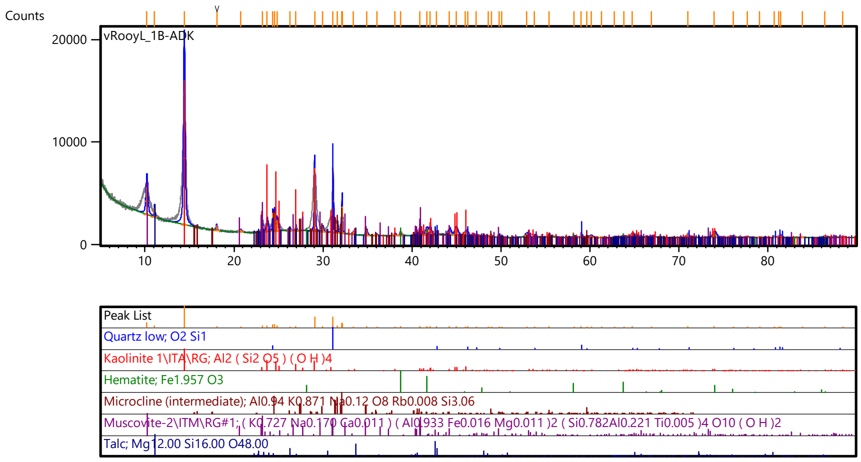
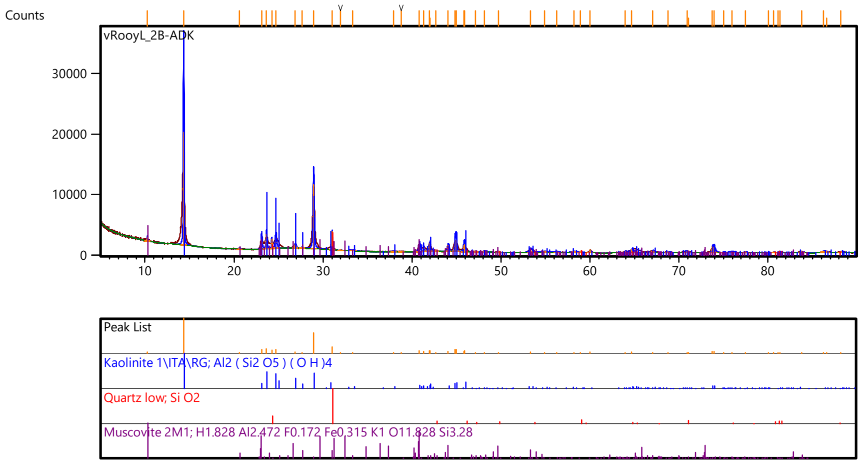
20000
10000
0
10
20
30
40
50
60
70
80
Peak List
Quartz low, syn; Si O2
Kaolinite 1\ITA\RG; Al2 (Si2 O5) (O H)4
Muscovite 2M1, syn; K Al2 Si3 Al O10 (O H)2

ADK 1B


Counts
vRooyL_1B-ADK
20000
10000
0
10
20
30
40
50
60
70
80
Peak List
Quartz low; O2 Si1
Kaolinite 1\ITA\RG; Al2 (Si2 O5) (O H)4
Hematite; Fe1.957 O3
Microcline (intermediate); Al0.94 K0.871 Na0.12 O8 Rb0.008 Si3.06
Muscovite-2\ITM\RG#1; (K0.727 Na0.170 Ca0.011) (Al1.933 Fe0.016 Mg0.011)2 (Si0.782Al0.221 Ti0.005)4 O10 (O H)2
Talc; Mg12.00 Si16.00 O48.00

ADK 2B


Counts
vRooyL_2B-ADK
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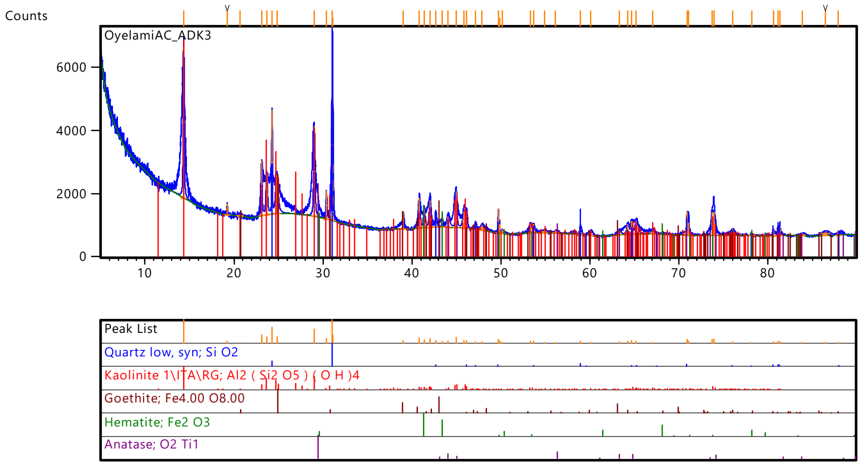
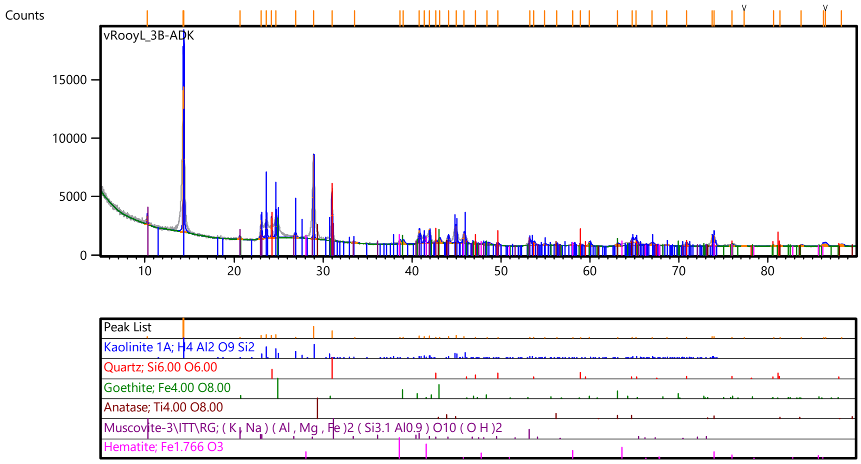
20000
10000
0
10
20
30
40
50
60
70
80
Peak List
Kaolinite 1\ITA\RG; Al2 (Si2 O5) (O H)4
Quartz low; Si O2
Muscovite 2M1; H1.828 Al2.472 F0.172 Fe0.315 K1 O11.828 Si3.28

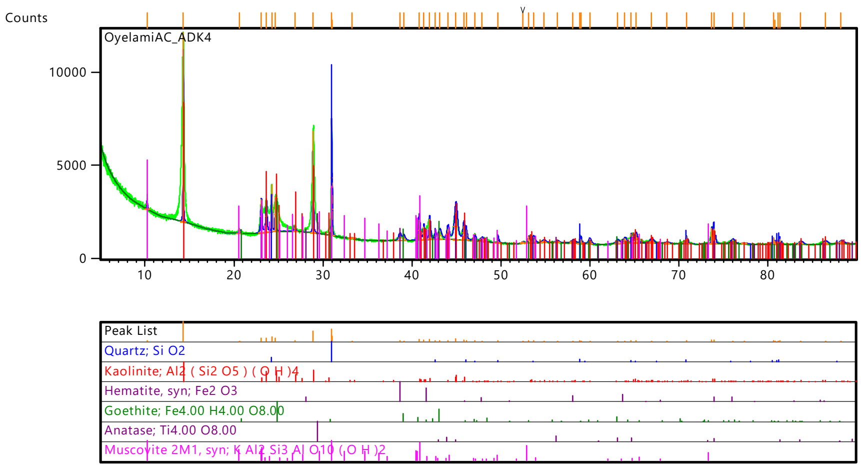
ADK 3A


Counts
OyelamiAC_ADK3
6000
4000
2000
0
10
20
30
40
50
60
70
80
Peak List
Quartz low, syn; Si O2
Kaolinite 1\ITA\RG; Al2 (Si2 O5) (O H)4
Goethite; Fe4.00 O8.00
Hematite; Fe2 O3
Anatase; O2 Ti1

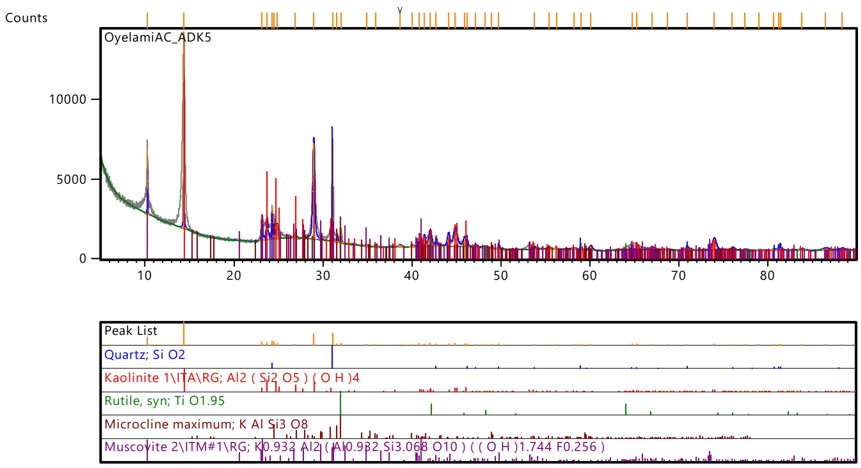
ADK 3B



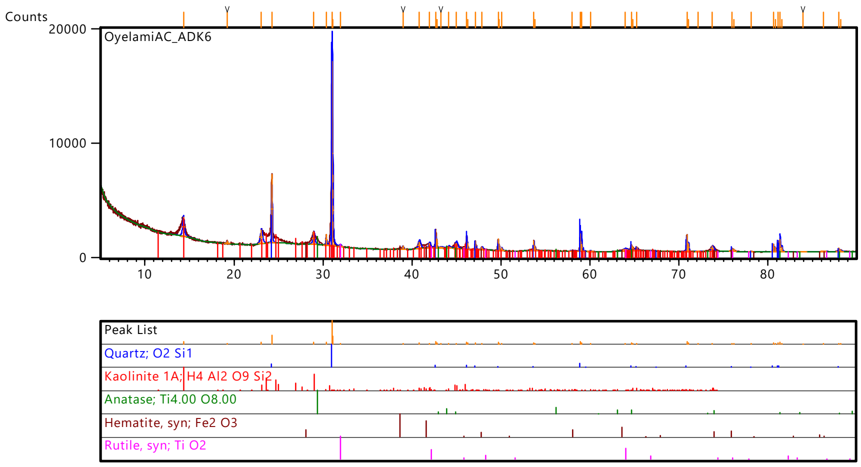
ADK 4



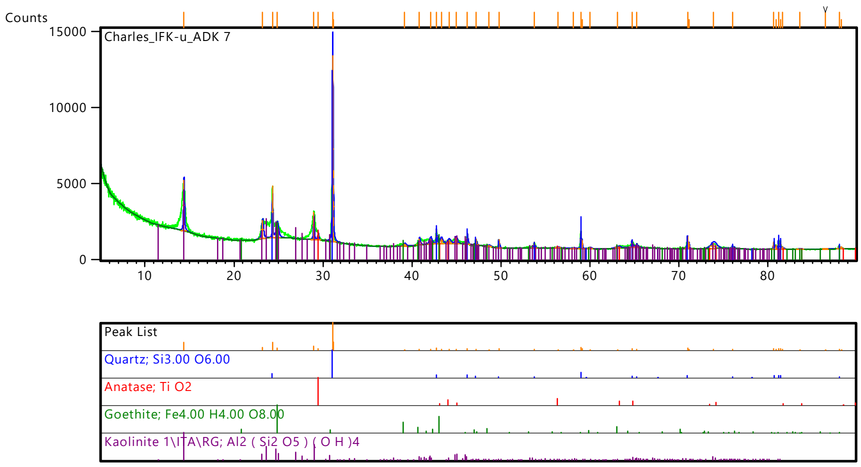
ADK 5



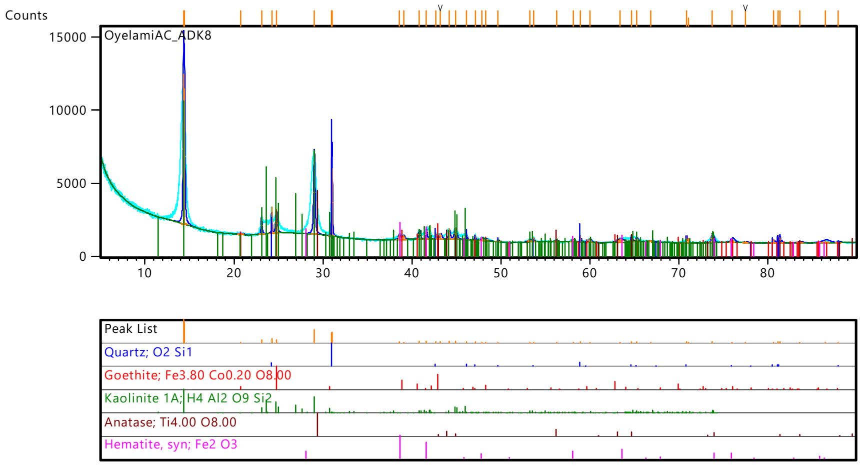
ADK 6


Counts
20000
10000
0
OyelamiAC_ADK6
10
20
30
40
50
60
70
80
Peak List
Quartz; O2 Si1
Kaolinite 1A; H4 Al2 O9 Si2
Anatase; Ti4.00 O8.00
Hematite, syn; Fe2 O3
Rutile, syn; Ti O2

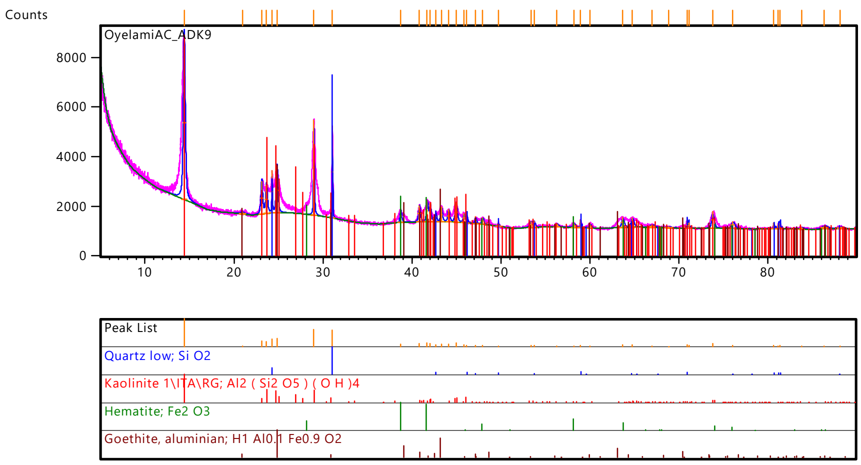
ADK 7


Counts
15000
10000
5000
0
Charles_IFK-u_ADK 7
10
20
30
40
50
60
70
80
Peak List
Quartz; Si3.00 O6.00
Anatase; Ti O2
Goethite; Fe4.00 H4.00 O8.00
Kaolinite 1\ITA\RG; Al2 (Si2 O5) (O H)4

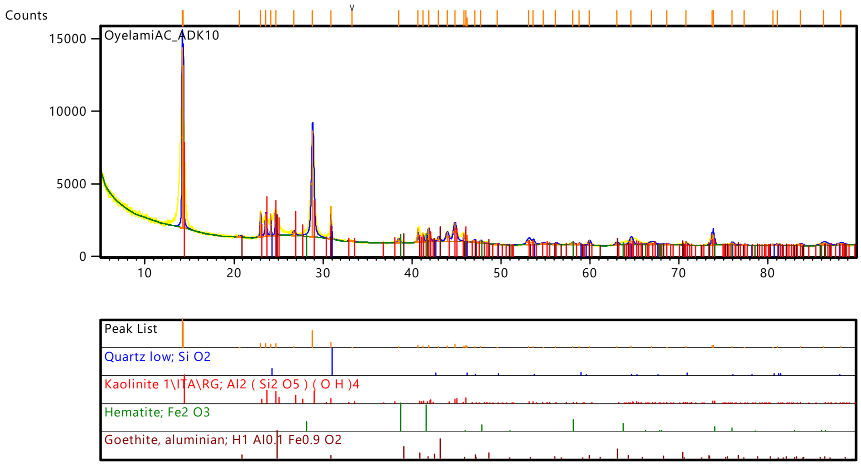
ADK 8



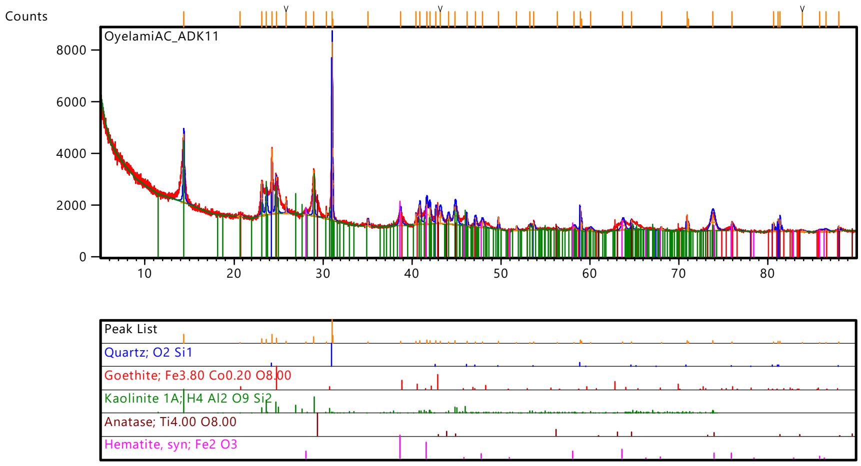
ADK 9



ADK 10


Counts
15000
10000
5000
0
OyelamiAC_ADK10
10
20
30
40
50
60
70
80
Peak List
Quartz low; Si O2
Kaolinite 1\ITA\RG; Al2 (Si2 O5) (O H)4
Hematite; Fe2 O3
Goethite, aluminian; H1 Al0.1 Fe0.9 O2

ADK 11


Counts
8000
6000
4000
2000
0
OyelamiAC_ADK11
10
20
30
40
50
60
70
80
Peak List
Quartz; O2 Si1
Goethite; Fe3.80 Co0.20 O8.00
Kaolinite 1A; H4 Al2 O9 Si2
Anatase; Ti4.00 O8.00
Hematite, syn; Fe2 O3