INVESTIGATION OF FLEXURAL STRENGTHS OF CONCRETES CONTAINING RICE HUSK AND SAW DUST ASHES FROM DIFFERENT CALCINATION METHODS

BY

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CERTIFICATION

I certify that this work **"Investigation Of Flexural Strengths Of Concretes Containing Rice Husk And Saw Dust Ashes From Different Calcination Methods"** was carried out by **Stanley Ogadinma Amadi (20134872718)** in partial fulfilment of the requirements for the award of Master of Engineering (M.Eng) degree in Structural Engineering in the Department of Civil Engineering of the Federal University of Technology, Owerri.

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DEDICATION

To Almighty God, for His unquantifiable love, grace, and mercy that have seen me through all my endeavours thus far.

And

To the memory of my late father, Mr Edmund Ohaeri Amadi, for the legacies he bequeathed to me.

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CONTENTS

LIST OF TABLES

LIST OF FIGURES/CHARTS

LIST OF PLATES

ABSTRACT

This work investigated the flexural strengths of concretes containing rice husk ash (RHA) and saw dust ash (SDA) from different calcination methods. RHA and SDA were produced using three different calcination methods namely, Open-Air (OA), Furnace (F), and Stove (S). Each of RHA and SDA was used as partial replacement of Ordinary Portland Cement (OPC) at 5%, 10% and 15%. 171 concrete beams of dimensions 150 x 150 x 600mm were produced using 1:2:4 cement–sand–sandstone mix ratio and 0.6 water/cement ratio. This comprised 81 OPC–RHA, 81 OPC–SDA and 9 OPC (control) concrete beams. The beams were cured by immersion and tested for flexural strengths at 28, 90 and 150 days. The results showed that the flexural strength of the blended cement concrete increased with curing age and decreased with increased amount of RHA and SDA. The 150^{th} day flexural strengths for 5% RHA were 5.35 Nmm^{-2} for OA, 6.74 Nmm^{-2} for F and 5.20 Nmm^{-2} for S. Corresponding strengths at 5% SDA were 4.48 Nmm^{-2} for OA, 5.85 Nmm^{-2} for F, and 4.28 Nmm^{-2} for S, while the Control value was 6.41 Nmm^{-2}. Thus, for 5% replacement of OPC with RHA, OA calcination had 83.5% the strength of the control; F calcination had 105.1% the strength of the control, while S calcination had 81.1% the strength of the control. Similarly, for 5% replacement of OPC with SDA, OA calcination had 69.7% the strength of the control; F calcination had 91.3% the strength of the control, while S calcination had 66.7% the strength of the control. Therefore, although furnace calcination gave higher strength than open-air and stove calcinations, the latter two could still be used for structural members with lower flexural strength.

Key words: Concrete, Blended cement, Pozzolan, Rice husk ash, Sawdust ash, Flexural strength, Sandstone Aggregate, Mineral admixture, Workability, Calcination.

CHAPTER ONE
INTRODUCTION

1.1 Background Information

One of the basic needs of man is housing. In any developing country like Nigeria, there is problem of accommodation and inadequate housing. A recent investigation showed that more than seven million Nigerians have no accommodation (Punch, 2012 in Onwuka, et al., 2013). It is important to note that majority of housing units in Nigeria are constructed using concrete.

Concrete has been observed as the most widely used construction material, as it is used for different purposes in the construction industry. It is estimated that the present consumption of concrete in the world is of the order of 11 billion metric tonnes every year (Mehta and Monteiro, 2006). In many countries the ratio of concrete consumption to steel consumption exceeds ten to one. Man consumes no other material except water in such tremendous quantities (Brunauer and Copeland, 1964).

To meet this great need of concrete for various construction purposes, there has been corresponding increase in the demand for cement which is one of the conventional materials for making concrete. Therefore, there is the need to use alterative cementitious materials that are locally available and cheap. This has challenged many researchers and engineers to seek and develop new materials relying on renewable resources. These include the use of by-products and waste materials in building construction (Adewuyi and Adegoke, 2008). In the last decade, the use of supplementary cementing materials has become an integral part of high strength and high performance concrete mix design. This replacement is made possible as a result of the potential ability of these supplementary cementitious materials (SCMs) to improve the properties and performance of concrete through their filler effect, different chemical and

mineralogical compositions, and Pozzolanic reaction. These supplementary cementitious materials can be natural materials, by-products or industrial wastes, or the ones requiring less energy and time to produce.

Many researches are being done on the possible use of locally available pozzolanic materials to partially replace cement in concrete as cement is widely noted to be most expensive constituent of concrete. Some of the commonly used supplementary cementing materials are fly ash, Silica Fume (SF), Ground Granulated Blast Furnace Slag (GGBFS), Rice Husk Ash (RHA), etc. The use of RHA and SDA and other pozzolans are receiving attention since they result in enhanced properties of the blended cement concrete, save the cost of construction materials and reduce the negative environmental influences.

Rice husk is the outer covering of the grain of rice plant with a high concentration of silica, generally more than 80-85% (Siddique, 2008). It is responsible for approximately 30% of the gross weight of a rice kernel and normally contains 80% of organic and 20% of inorganic substances. Rice husk is produced in millions of tons per year as a waste material in agricultural and industrial processes. It can contribute about 20% of its weight to Rice Husk Ash (RHA) after incineration (Anwar et al., 2001).

Khedr, et al. (1994) studied the characteristics of silica fume Concrete. Rao, et al. (1989) studied the nature and reactivity of silica available in Rice Husk Ash (RHA). The effect of use of RHA on the strength of concrete was studied by Deshmukh, et al. (2012). Apoorv, Patel, and Khalid (2014) studied the compressive and flexural strength of concrete containing Silica Fume, Rice Husk Ash and Iron slag as partial replacement of cement. Givi, et al (2010) studied the Contribution of rice husk ash to the properties of mortar and concrete.

Sawdust is an industrial waste in the timber industry. It is obtained as loose particles or wood chippings from sawing of timber into standard useable sizes. It poses a nuisance to both the health and environment when not properly managed. It has pozzolanic properties and has been

shown to react chemically with the calcium hydroxide released from the hydration of Portland cement, to form cement compounds (Elinwa and Mahmood, 2002). Elinwa and Mahmood (2002) and Elinwa and Ejeh (2004) used SDA as partial replacement in mortar and concrete works. It has also been used as a powder material in the production of self-compacting concrete (SCC) and in combination with metakaolin as a ternary blend with 3 % added to act as an admixture (Elinwa and Mamuda, 2014). Marthong (2012) investigated the compressive strength of mortar and concrete using SDA as partial cement replacement with three grades of ordinary Portland cement (OPC) namely: 33, 43 and 53 as classified by Bureau of Indian Standard (BIS). Sumaila and Job (1999) investigated the properties of SDA-OPC concrete, while Udoeyo and Dashibil (2002) investigated the use of saw dust ash as concrete material. Cheah and Ramli (2011) investigated the implementation of wood waste ash as a partial cement replacement material in the production of structural grade concrete and mortar. Variation of Strength of OPC-Saw Dust Ash Cement Composites with Water-Cement Ratio conducted by Ettu, et al (2013) has confirmed the suitability of Nigerian SDA as a pozzolanic material for producing concrete, sandcrete, or soilcrete, either in binary combination with OPC or in ternary combination with OPC and one other agricultural by-product pozzolan such as RHA.

Chemical compositions of Pozzolanic ashes are affected due to burning process and temperature. Silica content in the ash increases with higher burning temperatures. Studies have shown that RHA resulting from the burning of rice husks at controlled temperatures have physical and chemical properties that meet ASTM C 618-94a (2015). Abalaka (2012) studied the effects of methods of incineration on rice husk ash blended concrete on the split tensile strength of concrete mixes, using charcoal fired incinerator. Chinda, et al (2007) studied compressive strength using RHA with LOI of 11.2% sourced from power plants. The study of Nehdi, et al (2003) shows that at 12.5% replacement, RHA produced using a Torbed reactor operated at 750^0 produced higher compressive strength at 28 days than RHA produced using

fluidized bed technology. Nurdeen, et al (2011) studied the Influence of Calcination Temperature on Characteristics and Pozzolanic Activity of Palm Oil Waste Ash using an aired electric oven at different temperatures ranging from 500 to 1000 ºC for 1-6 hrs. Chandrasekhar, et al (2006) studied the effect of calcination temperature and heating rate on the optical properties and reactivity of rice husk ash. Embong, et al. (2016) investigated silica extraction and incineration process of sugarcane bagasse ash as pozzolanic materials.

1.2 Problem Statement

Studies are being conducted on the use of pozzolanic materials in blended cement concrete to obtain improved properties such as strength, durability, workability, density, etc. Industrial pozzolans such as fly ash, silica fume, granulated blast furnace slag, etc., are mostly used as pozzolans. Attempts are being made to produce and use agricultural by-product ashes such as rice husk ash (RHA) and saw dust ash (SDA) as supplementary cementitious materials.

Many researches have already been published on the properties of blended RHA concrete such as strength and durability. Few researches have also been published on the use of SDA in blended cement concrete. However, most of the reviewed literatures were on the study of the compressive strength of concrete and mortar, and not on the study of the flexural strength of concrete.

Most of the works in the literatures used industrial method of calcination such as furnace calcination which is not readily available in most communities. Could the ashes obtained from other calcination methods that can be easily used in rural communities still be suitable as pozzolans? This, in a nutshell, is the problem investigated in this work.

1.3 Objectives

The main objective of this study is to investigate the flexural strengths of concretes containing Rice Husk and Saw Dust Ashes from different calcination methods. The specific objectives are:

i. To determine the chemical compositions of RHA and SDA.

ii. To determine the effect of partially replacing OPC with RHA and SDA on the flexural strength of concrete.

iii. To determine the effect of RHA and SDA calcination methods on the flexural strength of concrete.

iv. To prepare empirical models for predicting flexural strengths of RHA and SDA concretes.

1.4 Significance of Study

i. This research work will contribute to the attempts being made to produce and use pozzolanic agricultural by-product ashes such as rice husk ash (RHA) and saw dust ash (SDA) as supplementary cementitious materials.

ii. It seeks to explore the suitability of agricultural pozzolans, especially RHA and SDA which are available in abundance than the industrial pozzolans, for use as partial cement replacement in construction works.

iii. It will help reduce the high cost of civil engineering construction works which has been reported to be due to the high cost of cement.

iv. The use of RHA and SDA as supplementary cement materials will go a long in reducing the enormous environmental pollution occasioned by large quantities of these agricultural wastes produced annually as well as reduce the amount of cement used in

construction works which has been reported to be responsible for pollution of the environment.

v. This study will help recommend suitable, readily available, and cheap method(s) of producing these pozzolanic ashes.

1.5 Scope of Study

The work investigated the flexural strengths of concrete containing RHA and SDA. The production of Rice husk and Saw dust ashes for this work was limited to three calcination methods, namely Open-air, Furnace and Stove.

CHAPTER TWO

LITERATURE REVIEW

2.1 General

Concrete has been observed as the most widely used construction material, as it is been used for varieties of purposes in the construction industry. The most widely used construction material is concrete, commonly made by mixing Portland cement with sand, crushed rock, and water… 63 million tons of Portland cement were converted into 500 million tons of concrete, five times the consumption by weight of steel. In many countries the ratio of concrete consumption to steel consumption exceeds ten to one. Man consumes no other material except water in such tremendous quantities (Brunauer and Copeland, 1964). It is estimated that the present consumption of concrete in the world is of the order of 11 billion metric tonnes every year (Mehta and Monteiro, 2006).

Three reasons can be adduced for this widespread use of concrete. First, concrete possesses excellent resistance to water without serious deformation and this makes it a suitable material for building structures to control, store, and transport water. Second, structural concrete elements can be easily formed into variety of shapes and sizes, because of the plastic consistency of fresh concrete. Third, concrete is usually the cheapest and most readily available material for construction jobs (Mehta and Monteiro, 2006).

Concrete is a composite construction material composed of cement (commonly Portland cement) and other cementitious materials such as fly ash and slag cement, aggregates (generally a coarse aggregate plus a fine aggregate), water and chemical admixtures.

Concrete is a composite material that consists essentially of a binding medium within which are embedded particles of fragment of aggregate (ASTM C125, 2014). In hydraulic cement concrete, the binder is formed from a mixture of hydraulic cement and water. Concrete

solidifies and hardens after mixing with water and placement due to chemical reaction of process known as hydration.

Concrete as is well known is a heterogeneous mix of cement, water and aggregates. In its simplest form, concrete is a mixture of paste and aggregates. Admixtures such as fly ash, rice husk, etc. may be added in concrete in order to enhance some of the desired properties. The character of the concrete is determined by quality of the paste. The key to achieving a strong, durable concrete rests in the careful proportioning, mixing and compacting of the ingredients. Based on the compressive strength, concrete can be categorised into three namely: low-strength concrete (less than 20 MPa), moderate-strength concrete (20 to 40 MPa) and high-strength concrete (more than 40 MPa). Moderate-strength or ordinary or normal concrete is used for most structural work, while high-strength concrete is used for special applications (Mehta and Monteiro, 2006).

2.2 Components of Concrete

The following are the components of concrete.

2.2.1 Cement

Cement is an adhesive and cohesive material that binds two or more non-cohesive surfaces together to form a single cohesive mass. When mixed with water it forms a paste which sets and hardens due to chemical reactions with water (Dvorkin, and Dvorkin, 2006).

Cement has been observed to contain calcium oxide and silicate as the main mixtures, with oxides of aluminium, iron, magnesium, sulphur etc. present in small quantities. Hydraulic cements (such as Portland cement) are made of a mixture of silicates and oxides, the four main compounds being: Belite ($2CaO.SiO_2$), Alite ($3CaO.SiO_2$), Celite ($3CaO.Al_2O_3$) and Brownmillerite ($4CaO.Al_2O_3.Fe_2O_3$). The silicates are responsible for the mechanical properties of the cement.

The approximate oxide composition limits of ordinary Portland cement according to Shetty (2006) are as follows: CaO (60-67%), SiO_2 (17-25%), Al_2O_3 (3.0-8.0%), Fe_2O_3 (0.5-6.0%), MgO (0.1-4.0%), alkalis (K_2O, Na_2O) (0.4-1.3%), SO_3 (1.3-3.0%). These oxides are responsible for the various properties of cement.

i. Calcium Silicates

These consist of the dicalcium silicate ($2CaO.SiO_2$) usually abbreviated as C_2S and the tricalcium silicate ($32CaO.SiO_2$) abbreviated as C_3S. They are the most stable compounds in cement and they are essential because they contribute most of the strength to the cement paste and contribute to about 70 to 80% of the constituent of cement. When cement comes in contact with water, C_3S hydrates rapidly, thereby contributing significantly to the development of early strength and heat of hydration; while C_2S hydrates slowly thereby generating less heat and is responsible for the later strength of cement paste and concrete.

ii. Calcium Aluminate and Ferroaluminate

Tricalcium aluminate ($3CaO.Al_2O_3$ or C_3A) is the least stable of the four major compounds in cement. It is very exothermic and reacts readily to form tetracalcium aluminoferrite (C_4AF). Unless the rapid hydration of C_3A is slowed down by the use of gypsum, Portland cement cannot be used for most construction purposes.

Tetra calcium aluminoferrite ($4CaO.Al_2O_3Fe_2O_3$ or C_4AF) is the least important of all the compounds in terms of strength. It also produces the least heat of hydration.

iii. Magnesium oxide and Calcium oxide

The hydration of magnesium oxide (MgO) and calcium oxide (CaO) is slow and expansive reaction that, under certain conditions, can cause unsoundness (i.e cracking and pop-outs in cement-based products).

iv. Alkali and Sulphate Compounds

The presence of alkalis (K_2O, Na_2O) has significant influence on the early hydration reactions of the cement.

The source of most sulphate (SO_3) is calcium sulphate in one of its several forms, added to the clinker during grinding to retard the quick-setting tendency of the clinker, attributed to the highly reactive C_3A phase.

2.2.1.1 Fineness of Cement

In addition to the compound composition, the fineness of cement also affects its reactivity with water. The fineness of cement has a direct effect on the hydration rate as increased fineness implies increased rate of hydration. Generally, researches have shown that the finer the particles of cement, the more rapidly the cement will react with water. Higher fineness (exceeding 400kg^2/m, Blaine) tends to increase early strength development and reduce bleeding. Therefore, the rate of reactivity and hence the strength development of cement paste can be enhanced by finer grinding of cement (Mehta and Monteiro, 2006). However, the cost of grinding and the heat evolved on hydration of the cement paste set some limits on the fineness. Coarse–ground cement falls within 250 to 280 m^2/kg, Blaine. Finer cements cause a more rapid generation of heat and greater strength gain particularly during hydration. Concrete with higher fineness ground cement will generally have a higher water demand for the required consistency and will bleed at a slower rate.

The fineness of cement is easily determined as the residue on 75 µm and 45 µm standard sieves. Due to the difficulty and the expensive equipment required, it is a common practice to obtain a relative measure of the particle size distribution from surface area analysis of the cement by the Blaine Air Permeability Method. Generally, cement particles larger than 45 µm hydrate slowly while those larger than 75 µm may never hydrate completely.

The fineness of cement is measured as specific surface. Specific surface is expressed as the total surface area in square metres of all the cement particles in one kilogram of cement. The higher the specific surface is, the finer the cement will be.

2.2.2 Aggregate

Aggregates are granular materials which can be mixed with binding materials such as cement or lime in the preparation of mortar or concrete based on sizes. Aggregates are very important constituent of concrete as they occupy about 70-80% of the volume and 70 to 85 per cent of the weight of concrete (Neville, 1996). Its use considerably improves thermal and elastic properties and volume stability and durability and reduces shrinkage of the resulting concrete as well as affect economy. The high variation in strength between concrete and mortar of the same cement/aggregate proportion, suggests the quintessence of coarse aggregates in the development of strength in concretes. For this reason, the quality of the coarse aggregates is essential when considering the quality of the concrete itself. The properties of coarse aggregates do grossly affect the durability and structural performance of concrete (Aginam, et al, 2014).

Earlier, aggregates were considered as chemically inert materials but now, it has been recognized that some aggregates are chemically active and also certain aggregates exhibit chemical bond at the interface of aggregate and paste (Shetty, 2006). Thus, its physical characteristics and in some cases its chemical composition affect to a varying degree the properties of concrete in both plastic and hardened states.

The choice of aggregates is important, their quality plays a great role, they can not only limit the strength of concrete but owing to their characteristics, they affect the durability and performance of concrete. It is important that the aggregates have good strength, durability, weather resistance and free from impurities that may weaken the bond with cement paste.

2.2.2.1 Classification of Aggregates

Aggregates can be classified according to their particle sizes, bulk density or weight, or source.

Aggregates are classified on the basis of particle sizes into:

i. Fine Aggregates

Fine aggregate is used for particles smaller than 4.75 mm. Typically, fine aggregates contain particles in the size range 75μm to 4.75 mm.

ii. Coarse Aggregates

The term coarse aggregate is used to describe particles in the size range from 4.75 mm to 50 mm. (Shetty, 2006).

Aggregates are classified on the basis of bulk density or weight into:

i. Normal Weight Aggregates

These have bulk density of 1520 to 1680 kg/m^3 and produce normal-weight concrete with approximately 2400 kg/m^3 unit weight (Shetty, 2006, Mehta and Monteiro, 2006). They include natural aggregates such as sand, gravel, granite, quartz, basalt, sand stone; and artificial aggregates such as broken bricks, air-cooled slag, sintered fly ash, bloated clay.

ii. Light Weight Aggregates

These are used in the production of light weight concrete so as to achieve a reduction in self-weight of concrete and to increase the efficiency of concrete as a structural material. They have bulk densities less than 1120 kg/m^3 and can be used to produce light weight concrete with bulk unit weight ranging from 300 to 1850 kg/m^3. Examples include: pumice, clays, shale, slate, etc.

iii. Heavy Weight Aggregates

These aggregates have bulk density ranging from 2900 to 6100 kg/m^3 (Mehta and Monteiro, 2006). Heavy weight aggregates are used in the production of heavy weight concrete with bulk unit weight ranging from 4000 to 5000 kg/m3, depending on the aggregate type, dimensions and degree of compaction. The most widely used natural heavy weight aggregates are barite and iron ores such as hematite and magnetite, whereas the artificial ones are based on iron shot or lead shot.

Aggregates can also be classified on the basis of source into:

iv. Natural Mineral Aggregates

These are derived from rocks of several types such as igneous, sedimentary and metamorphic rocks. These comprise of sand, gravel, and crushed rocks such as carbonate rocks, sandstone, granite, diorite, gabbro, and basalt. In general, aggregates from igneous rock origin have been observed to be very much stronger than those from sedimentary and metamorphic rock origin.

v. Synthetic Aggregates

These include thermally processed materials such as expanded clay and shale, which are used for making lightweight concrete. Also, aggregates made from industrial by-products (e.g. blast-furnace slag and fly ash), municipal wastes and recycled concrete from demolished buildings and pavements are synthetic aggregates.

2.2.2.2 Properties of Aggregate

Good aggregates should produce the desired properties in both fresh and hardened concrete. The properties of the aggregate known to have significant effects on concrete are its strength, deformation, durability, toughness, volume change, porosity, specific gravity, shape and grading, chemical reactivity, etc. (Neville and Brook, 2010). These properties of aggregates are

discussed under physical, mechanical properties, and chemical reactivity (alkali-aggregate reaction).

i. Physical Properties of Aggregates

a. Size and Grading

Grading is the proportioning of particles of different sizes of the aggregates from large to small to obtain a mixture that will provide the required workability and strength in concrete (BS 882, 1973).

Aggregates are graded to determine the particle size distribution and this is achieved using the sieve analysis. The grading curve shows the particle size distribution of the aggregates. A distribution with virtually one particle size is termed uniformly graded, while that in which some of the particle sizes are missing is called gap graded. The distribution with wide range of sizes of particles is called well graded. Well graded aggregates produce concrete that is strong, dense and impervious to water. Also as sizes of coarse aggregate increases, concrete strength decreases whether the aggregate is natural or artificial.

b. Shape

The shape of aggregates is an important characteristic since it affects the workability of the concrete. Some examples of shapes of aggregates are: rounded (e.g. river or seashore gravels, seashore and wind-blown sands), irregular or partly rounded (e.g. pit sands and gravels), angular (e.g. crushed rocks of all types), flaky (e.g. laminated rocks).

From the standpoint of economy in cement requirement for a given water-cement ratio, rounded aggregates are preferable to angular aggregates. On the other hand, the additional cement required for angular aggregate is offset to some extent by the higher strength and sometimes by greater durability as a result of interlocking texture of the hardened concrete and higher bond characteristics between aggregate and cement paste.

Flat aggregates in concrete will have particularly objectionable influence on workability, cement requirement, strength and durability. In general, excessively flaky aggregates make very poor concrete (Shetty, 2006).

The Particle Size Distribution Curve

This is a curve plotted after the sieve analysis test has been performed. The specimen can be described based on the shape of the curve and how it fits on the gradation chart.

If the major part of the curve is steep or almost vertical, then the specimen has particle distribution extending over a limited range with most particles tending to be about the same size. The specimen is said to be closely graded or uniformly graded. If the shape of the curve is not steep but more or less flat extending over the full range of the specimen particle sizes axis, then the sample has particle sizes distributed evenly over a wide range of the particle sizes within the specimen with no deficiency or excess of any particle. Such specimen is said to be well graded. A specimen is said to be gap graded if it has a large widely varying percentages of intermediate particles.

Effective Size (D_{10})

The effective size is defined as the largest size of the smallest 10 percent. Other particle sizes such as D_{30} and D_{60} are defined in similar manner.

Uniformity Coefficient, Cu

This is the ratio of the maximum particle size of the smallest 60 percent to the effective size, and is denoted by:

$$Cu = \frac{D_{60}}{D_{10}} \quad (2.1)$$

If Cu < 4, the specimen is uniformly graded. If Cu > 4, the specimen is either well graded or gap graded.

Coefficient of Gradation, Cg

This is calculated from the equation:

$$Cg = \frac{(D_{30})^2}{D_{60} \times D_{10}} \quad (2.2)$$

Fineness Modulus, F

This is the index of coarseness or fineness of a specimen. It is an empirical factor obtained by adding the cumulative percentage of aggregate retained on each of the standard sieves and dividing this sum by an arbitrary number, 100. The larger the value of the fineness modulus of a specimen; the more coarse the specimen will be. Nawy (2001) recommends a fineness modulus in the range of 2.5 to 3.2.

c. Surface Texture

This is the property, the measure of which depends upon the relative degree to which particle surfaces are polished or dull, smooth or rough. Surface texture of aggregates depends on hardness, grain size, pore structure, structure of the rock, and the degree to which forces acting on the particles have smoothened or roughened.

As surface smoothness increases, contact area decreases, hence, a highly polished particle will have less bonding area with the matrix than a rough particle of the same volume. A smooth particle, however, will require a thinner layer of paste to lubricate its movements with respect to other aggregate particles (Mehta and Monteiro, 2006). It will therefore, permit denser packing for equal workability and hence, will require lower paste content than rough particles. Researches have shown that rough textured aggregates develop higher bond strength in tension than smooth textured aggregates.

d. Water Absorption

This is an indirect measure of the permeability of an aggregate and as such in turn relates to other physical properties such as strength, shrinkage, soundness, and its general durability

potential. The absorption of water reduces workability of concrete. So an aggregate with high water absorption produces a less workable concrete (Shetty, 2006). Aggregates can be classified based on the quality of water in theme as: bone dry, air dry, saturated and surface dry and moist.

e. Bulk Density

The bulk density of an aggregate or its unit weight reflects in its void content and therefore, an indirect measure of grading and shape characteristics. The bulk density is most commonly used to enable concrete mixes specified in volume to be converted into gravimetric proportions and thus, enable batch masses to be determined. There are two types of bulk density viz: uncompacted bulk density and compacted bulk density (BS 812, 1975).

f. Specific Gravity

Aggregate is the major constituent of concrete and as such its specific gravity is important factor affecting the density of the resulting concrete. Since aggregates incorporate pores, the value of specific gravity varies depending on the extent which the pores absorb water when the value is determined. For the purpose of mix design, the specific gravity of a "saturated and surface dry'' basin is used. The specific gravity of most natural aggregates falls within the range 2.5 to 3.0 (Shetty, 2006). The apparent specific gravity for many commonly used rocks ranges between 2.6 and 2.7; typical values for granite, sandstone, and dense limestone are 2.69, 2.65, and 2.60 (Mehta and Monteiro, 2006).

ii. **Mechanical Properties of Aggregates**

a. Aggregate Impact Value

This value gives the toughness of an aggregate which is its resistance to failure by impact. Toughness of aggregate is determined by the impact value test. The aggregate impact value gives a relative measure of the toughness or resistance of an aggregate to sudden shock or impact which in some aggregates differs from its resistance to slow compressive load. The aggregate impact value is restricted to 45 per cent for normal concrete work as shown in table 2.1. Aggregate impact values are used to classify stone aggregates with respect to toughness property. Table 2.1 shows the aggregate impact values as specified by BS 812 (1975).

Table 2.1: Aggregate Impact Values

Aggregate Impact Value (%)	Toughness Properties
< 10	Exceptionally tough/ Strong
10 – 20	Very tough/ Strong
20 – 30	Good for pavement surface course
> 35	Weak for pavement surface course

b. Abrasion and Attrition Value

This measures the aggregate hardness which is the resistance of an aggregate to wear. Abrasion and attrition values of aggregates are determined by carrying out abrasion and attrition test using the Los Angeles abrasion test machine (ASTM C131). The resultant particles smaller than 1.7 mm size from the test should not be more than 30% for aggregate use in road and floor works. For other concrete works up to 50% of particles smaller than 1.7 mm may be permitted. The more the amount of particles that is smaller than 1.7 mm, the lower the hardness, hence, the lower the resistance of the aggregate to wear. Table 2.3 shows Los Angeles Abrasion loss values as specified by AASHTO, and U.S FHWA for use in highway construction.

Table 2.2: Typical L.A. Abrasion Loss Values (AASHTO T 96, 2003 and ASTM C 131, 2003)

Rock Type	L.A. Abrasion Loss (by percent weight)
General Values	
Hard, igneous rocks	10
Soft limestone and sandstones	60

Ranges for specific rocks	
Basalt	10 – 17
Dolomite	18 – 30
Gneiss	33 – 57
Granite	27 – 49
Limestone	19 – 30
Quartzite	20 – 35

2.2.2.3 Chemical Reactivity (Alkali-Silica Reaction, ASR)

Alkali-silica reaction (ASR) occurs either in mortar or concrete. ASR is a deleterious chemical reaction between hydroxyl (OH^-) ions associated with alkalis (sodium and calcium) present in cement or other sources and certain reactive siliceous components that may be present in aggregates, producing gel. When this alkali-silica gel absorbs moisture, it expands, and eventually produces cracks in aggregate particles as well as in the cement paste in concrete (Prezzi e tal, 1997). It causes serviceability problem in concrete structures.

The following three conditions must be satisfied for expansive ASR to occur:

i. A reactive form of silica or silicate must be present in the aggregate.
ii. Sufficient alkali, sodium (Na) and or potassium (K) mainly from cement must be available.
iii. Sufficient moisture, i.e., not less than 80% relative humidity (RH) in the pore structure of the concrete or mortar, is required.

It is the form of silica that determines whether a siliceous aggregate is reactive or not. Many of the coarse aggregates used in construction are siliceous in composition (i.e. high in silica, SiO_2 content. However, they are not necessarily reactive. Certain reactive aggregates do not exhibit maximum expansion unless the aggregate is present in critical range. Fine aggregate is more susceptible to ASR because of it higher surface area (Paramsivan, 2006).

2.2.2.4 Sandstone Aggregate

The coarse aggregates are obtained naturally or artificially and occupies up to 60% by weight or volume of the concrete, depending on the mix proportion adopted which, in turn, depends on the expected compressive strength.

It is a common practice in Nigeria to use locally available aggregates for construction purposes. The integrity of these aggregates should be investigated to ascertain their performance in structural members.

The local stone aggregate otherwise known as Sandstone or Arenite is a sedimentary rock consisting of sand consolidated with some cementitious material. Sandstone is a sedimentary rock composed of sand-sized grains of minerals, rock or organic material. Mainly clay, quartz and feldspar form its major mineral constituents. Sandstone consisting of more than 25% feldspar clasts is termed arkose; while sandstone consisting of more than 90% quartz clasts is called quartzose.

It is composed of sand sized minerals or rock grains and may appear any colour depending on the geographical location from where it is obtained. The most common colours being tan brown, yellow, grey, pink, black and white. It also contains cementing materials that bind the sand grains together (the most cementing materials are silica and calcium carbonate) and may contain a matrix of silt- or clay-size particles that occupy the spaces between the sand grains. Sandstone is one of the most common types of sedimentary rock and it is found in sedimentary basins throughout the world. In the sub-surface, sandstone often serves as an aquifer for groundwater or as a reservoir for oil and natural gas. It is gritty to touch, like sandpaper. It is relatively soft, making it easy to carve. Sandstone is often mined for use as a construction material or as a raw material used in manufacturing.

Researches have shown that aggregates types and sizes affect the strength of concrete. Rough and highly angular aggregate particles increase internal friction in concrete, but lead to higher percentage of voids (Gupta and Gupta, 2004). The internal friction increase concrete strength while the voids lead to decrease in strength (Shetty, 2005). In contrast however, smooth gravel, which produces low voids give lower stresses than the rough and angular crushed rock (Neville, 1995). Smooth coarse aggregates lowered the strength of concrete by 10% than when the aggregates were roughened (Neville, 1981).

Young and Sam (2008) stated that smooth rounded aggregates was more workable but yielded a lesser compressive strength in the matrix than irregular aggregates with rough surface texture. They were also of the opinion that a fine coating of impurities such as silt on the aggregate surface could hinder the development of a good bond and thus affects the strength of concrete produced with the aggregates.

The test carried out by Soroka (1993), revealed the variations between the compressive strengths of concrete made with crushed stone and uncrushed stone. He achieved a better compressive strength with the crushed stone than the uncrushed stone. This strength performance was as a result of several factors like water/cement ratio, grading, surface texture, shape, strength, and stiffness of aggregates used.

Also, Bloem and Gaynor (1963) studied the effect of shape, surface texture, fine coatings, and maximum size of aggregates on the water requirement and strength of concrete. The study reported that at equal water/cement ratio, irregular shaped smaller sized aggregates without coatings achieved a better strength than smooth rounded large sized aggregates. They also opined that individual properties of aggregates and the magnitude of the size difference may lead to increase or decrease in concrete strength at a fixed cement content.

Furthermore, Chen and Liu (2004), as well as Rao and Prasad (2002), viewed aggregates as the skeleton of concrete and consequently persuaded that all forms of coatings should be avoided in order to achieve a good concrete. When a concrete mass is stressed, failure may originate within the aggregates, the matrix, or at the aggregate-matrix interface. The aggregate-matrix interface is an important factor determining the strength of concrete.

The study by Jimoh, and Awe (2007) reveals that for combination of artificial and natural aggregates, the concrete having sand and granite is stronger than the corresponding one with quarry dust and gravel. Also as sizes of coarse aggregate increases, concrete strength decreases whether the aggregate is natural or artificial. The rate of decrease in strength with change in aggregate size is highest in concrete with quarry dust and granite and lowest in concrete having combination of sand and gravel. This shows that the strength of concrete is more sensitive to difference in sizes when aggregates are granite and quarry dust than similar one with gravel and sand.

2.2.3 Water

Water is an important ingredient of concrete as it actively participates in the chemical reaction with cement. Since it helps to form the strength giving cement gel, the quality and quantity of water is required to be looked into very carefully. Water used in concrete in addition reacting with cement and thus, causing it to set and harden, also facilitates mixing, placing and compacting of fresh concrete. It is used for washing aggregates and for curing purposes.

2.2.3.1 Quality of Water

In general, water suitable for drinking is specified by BS 3148 (1980) as good water for making concrete. However, this does not appear to be a true statement for all conditions.

The water should be free from excessive amounts of oils, silts, clay, acids, sugar, alkalis, salts, organic matter sewage, etc. Sea water is not known to have any adverse effect on strength and durability of Portland cement concrete but should not be added in concrete making as it can cause staining or damping of the concrete surface and corrosion of reinforcement for reinforced concrete. The best way to find out, whether a particular source of water is suitable for concrete making or not, is to make concrete with this water and compare its 7 and 28 days strength with companion cubes made with distilled water. If the compressive strength is up to 90%, the source of water may be accepted (Shetty, 2006).

However, it is logical to know what harm the impurities in water do to the concrete quality and what degree of impurity is permissible in mixing and curing concrete. Carbonates and bi-carbonates of sodium and potassium have been observed to affect the setting time of cement by either accelerating or retarding the setting. Higher concentrations of these salts will also affect the strength and durability of the concrete. Brackish water contains chlorides and sulphates. When chloride does not exceed 10,000 ppm and sulphate does not exceed 3,000 ppm the water is harmless. Salts of Manganese, Tin, Zinc, Copper and Lead cause a marked reduction in the strength of concrete. Silts and suspended particles are undesirable as they interfere with setting, hardening and bond characteristics of concrete. A turbidity of 2,000 ppm has been suggested (Shetty, 2006).

2.2.3.2 Quantity of Water

The total quantity of water required in concrete mix is the water absorbed to meet the saturated dry surface condition and the free water absorbed by the cement for full hydration, workability and strength. So it is important to know the moisture content of the aggregate before mixing of the concrete. The more the water is added to the mix, the more fluid and plastic, but the weaker the concrete will be (Neville, 1996 and Shetty, 2006). Much water will cause the aggregates to

segregate resulting in a concrete with non-uniform strength and workability as excess water will float the fine and light particles of cement to the top of the freshly placed concrete and some portion of the concrete will not have enough cement for proper hydration of the paste. This is called bleeding. So, it is important to regulate the quantity of water in a concrete mix. This is obtained from the water to cement ratio.

2.3 Mineral Admixtures (Pozzolans)

An admixture is a material other than water, aggregates, hydraulic cements, and fibre reinforcement that is use as an ingredient of concrete or mortar and added to the batch immediately before or during mixing (ASTM C 125, 2014).

Mineral admixtures are pozzolans. They are finely divided siliceous materials that are added to concrete in relatively large amounts, generally, in the range 20 to 60% by mass of the total cemtitious material (Mehta and Monteiro, 2006). A pozzolan is a siliceous or a siliceous and aluminous material which in itself possesses little or no cementitious property but will, in finely divided form and in the presence of moisture or water, chemically react with calcium hydroxide at ordinary temperature to form compounds possessing cementitious properties (ASTM C 618, 1978).

Mineral admixtures or pozzolans are commonly used as an addition or supplementary cementitious materials (SCM) (i.e. cement extender) to improve the properties of concrete. To act as a good SCM, the pozzolan must possess a pozzolanic property, i.e., providing excess reactive SiO_2 to react with $Ca(OH)_2$ released from cement hydration to yield calcium silicate hydrates (CSH). Good pozzolanic activity results from high specific surface area (100-200 m^2/g), small particle size (<10 μm), low carbon content (< 6-8% by weight), and most importantly, high amorphous SiO_2 content (80-90% by weight), among other factors (Wansom et al., 2009)

Researches have shown that the use of Pozzolanic materials exercise beneficial effects on the properties of concrete. Mineral admixtures have the following significance on the properties of concrete:

i. Used when special performance is needed:

Enhancement of the long-term strength and impermeability of concrete (due to pore refinement and strong interfacial transition zone) and workability of fresh concrete; improvement in resistance of concrete to thermal cracking (due to low heat of hydration), alkali-aggregate expansion, and sulphate attack; low heat of hydration; improved durability, correcting deficiency in aggregate gradation (as filler), etc.

ii. Results in cost and energy savings:

Replacement of cement leads to cost savings, energy required to process these materials is also much lower than cement.

iii. Environment damage and pollution is minimized

2.3.1 Classification of Pozzolans

There have been several classifications for pozzolans and natural pozzolans. Mineral admixtures are divided into two namely:

I. Natural materials

These are materials that have been processed for the sole purpose of producing pozzolans. These include natural pozzolans such as volcanic tuff or pumicite, volcanic glasses, calcined clays or shale, and opaline cherts and diatomaceous earths. Natural pozzolan is a raw or calcined natural material that shows pozzolanic properties. Natural pozzolans are those materials which do not require any further treatment apart from grinding to react with lime.

II. By-product materials

These are those materials that are not the primary products of the industry producing them. These include: industrial pozzolans (e.g. fly ash, silica fume, ground granulated blast furnace

slag, etc.) and agricultural by-product pozzolans (e.g. rice husk ash, wood ash, etc.). These artificial pozzolans are the materials with low pozzolanic activity and need further treatments to achieve pozzolanic activity.

Mineral admixture may also be categorized into:

i. chemically active mineral admixtures

These have been observed to decrease workability and setting time of concrete but decrease the heat of hydration and reactivity.

ii. Micro-filler admixtures

These have been observed to increase workability and setting time of concrete but decrease the heat of hydration and reactivity.

In general, small particle size and specific surface area of mineral admixture are favourable to produce highly dense and impermeable concrete; however, they result in low workability and demand more water which may be offset by adding effective superplasticizer.

2.3.2 Properties of Pozzolans

2.3.2.1 Physical Properties

Physical properties of natural and artificial pozzolans vary widely. The fineness, specific surface area, the shape of particles, and density of pozzolans influence the properties of fresh concrete and the strength development of hardened concrete.

I. Fineness

Fineness of pozzolans is usually measured by wet sieving method. The amount of the sample retained when wet sieved on a 45μ sieve is determined in accordance with ASTM C430. Several standards specify the maximum residue in percentage retained on a 45μ sieve. Depending on the grinding system and grinding time, natural pozzolans are produced from a

few microns up to 200µ. Particle size distribution of pozzolans can be measured by laser particle size analyzer, X-ray sedigraph, and coulter counter (Ali, 2014).

II. Specific Surface Area

The specific surface area of pozzolans, which is the area of a unit mass, is measurable by different techniques. The most common is the Blaine specific surface area technique, which measures the resistance of compacted particles to an air flow. A laser particle size analyzer can also be used for determination of the specific surface area of pozzolans. The Brunauer-Emmett-Teller (BET) nitrogen adsorption technique has also been used for determining the specific surface of the particles, but the results obtained by this method are usually higher than the results obtained by the Blaine specific surface area technique or particle size analyzer. The specific surface area of blended pozzolans is usually similar to that for the normal Portland cement at about 300–400 kg/m^2. Larger surface area with smaller particles of natural pozzolans can be produced by separate grinding. Higher surface area of about 500 kg/m^2 has increased the pozzolanic activity of most natural pozzolans (Ali, 2014).

III. Shape

The shape of particles of natural pozzolans is usually angular or irregular. Some of them have micro porous character and increase the water demand for constant consistency in concrete mixtures.

IV. Specific Gravity

The specific gravity of pozzolans is determined according to ASTM C188 (2009) test method similar to Hydraulic cements. The specific gravity of different natural pozzolans varies over a wide range, like the other physical properties. The specific gravity of volcanic ashes varies between 1.8 and 2.4 and reaches to 2.9 for the high density compacted tuffs.

2.3.2.2 Chemical Properties

The chemical composition of natural and artificial pozzolans varies within wide ranges and depends on their sources. Silica content is usually high in pozzolans. The other major oxide elements are aluminum, iron, calcium and magnesium oxides. The alkali content is not high but may vary between 3 and 10 %. Loss on ignition is generally low for natural pozzolans but may reach to 9 % in some pozzolans (Ali, 2014). However, the LOI may be high for artificial pozzolans depending on the burning conditions. Minor elements are present and are very variable. The mineralogical composition of natural and artificial pozzolans also varies at different sources. Owing to their formation process, the natural pozzolans consist of crystalline and non-crystalline particles as glassy particles.

2.3.3 Pozzolanic Activity (Pozzolanic Reaction)

The reaction between a pozzolans and calcium hydroxide is called the Pozzolanic reaction. Pozzolanic reaction is defined as the reaction between the active phases of the pozzolan with lime. It is an acid-base reaction between calcium hydroxide, also known as Portlandite, or $Ca(OH)_2$, and silica acid (H_4SiO_4, or $Si(OH)_4$). The amount of amorphous material usually determines the reactivity of pozzolans. It is fairly difficult to determine the reactive constituents of pozzolans and pozzolanic reaction is usually assessed by the consumption of lime in a mixture of lime-pozzolan or measurement of the silica and alumina soluble in acid.

2.3.3.1 Factors Affecting Pozzolanic Activity

The Pozzolanic activity of a material primarily depends on two factors namely:

i. The amount of calcium hydroxide available for reaction with the pozzolans.

The amount of available calcium hydroxide depends on the chemical properties of the pozzolans used, the nature of its active phase, the content of SiO_2 in the active pozzolans, and the $Ca(OH)_2$ to pozzolan ratio in the mixture.

ii. The reaction rate at which this combination occurs.

The reaction rate depends on physical factors, such as the surface area of pozzolan, the solid to water ratio of the mixture and the temperature.

2.3.3.2 The Pozzolanic Reaction

A pozzolanic reaction occurs when a siliceous or aluminous material gets in touch with calcium hydroxide in the presence of humidity to form compounds exhibiting cementitious properties (Papadakis et al., 2002). In the cement hydration development, the calcium silicate hydrate (C-S-H) and calcium hydroxide ($Ca(OH)_2$, or CH) are released within the hydration of two main components of cement namely tricalcium silicate (C_3S) and dicalcium silicate (C_2S); where C, S represent CaO and SiO_2 (Omotosoa et al., 1995). Hydration of C_3S, C_2S also C_3A and C_4AF (A and F symbolize Al_2O_3 and Fe_2O_3) respectively, is important.

In the Pozzolanic reaction, the additive acts in three ways namely:

i. Filler: these admixtures are finer than cement, so when added to concrete they occupy the small pores previously left vacant.
ii. Nucleating: these fine particles accelerate the rate of hydration and precipitation starts.
iii. Pozzolanic: when cementing material reacts with water the following takes place:

C_2S + HCSH+ CH; C_3S + HCSH + CH

Simply, this reaction can be schematically represented as follows:

$$\text{Pozzolan} + Ca(OH)_2 + H_4SiO_4 \xrightarrow{\text{slow}} Ca^{2+} + H_2SiO4^{2-} + 2H_2O \longrightarrow$$

$$CaH_2SiO_4.2H_2O \qquad (2.3)$$

Upon wetting, the following reactions occur (Englehard et al., 1995):

$$2(3CaO.SiO_2) + 6H_2O \longrightarrow 3CaO_2SiO_2.3H_2O + 3Ca(OH)_2 \qquad (2.4)$$

$$2(2CaO.SiO_2) + 4H_2O \longrightarrow 3CaO.2SiO_2.3H_2O + Ca(OH)_2 \quad (2.5)$$

$$3CaO.A1_2O_3 + 31H_2O + 3CASO_4 \longrightarrow 3CaO.A1_2O_3.3CaSO_4.31H_2O \quad (2.6)$$

$$.A1_2O_3.Fe_2O_3 + 10H_2O + 2Ca(OH)_2 \longrightarrow 6CaO.Al_2O_3.Fe_2O_3.12H_2O \quad (2.7)$$

In the pozzolan-lime reaction, OH^- and Ca^{2+} react with the SiO_2 or Al_2O_3-SiO_2 framework to form calcium silicate hydrate (C-S-H), calcium aluminate hydrate (C-A-H), and calcium aluminate ferrite hydrate:

Tobermorite gel: $SiO_2 + Ca(OH)_2 + H_2O \longrightarrow CaO.SiO_2.H_2O$ (2.8)

Calcium aluminate hydrate: $Ca(OH)_2 + H_2O + Al_2O_3 \longrightarrow A1_2O_3.Ca(OH)_2.H_2O$ (2.9)

Calcium aluminate ferrite hydrate:

$$Ca(OH)_2 + Fe_2O_3 + A1_2O_3 + H_2O \longrightarrow Ca(OH)_2.A1_2O_3.Fe_2O_3.H_2O \quad (2.10)$$

This can be summarised in abbreviated notation of cement chemists:

$$\text{Pozzolans} + CH + SH \xrightarrow{\text{slow}} C\text{-}S\text{-}H \quad (2.11)$$

The product formed is a calcium silicate hydrate (C-S-H). C-S-H is responsible for strength while CH is a soluble material that reacts and dissolves in water leaving behind pores. The C-S-H gel generated by the hydration of C_3S and C_2S in equations (2.4) and (2.5) is the main strengthening constituent. Calcium hydroxide and Ettringite ($3CaO.3CaSO_4.31H_20$, equation 2.4) that are crystalline hydration products are randomly distributed and form the frame of the gel-like products. Hydration of C_4AF (equation 2.7), consumes calcium hydroxide and generates gel-like products. Excess calcium hydroxide can be detrimental to concrete strength, due to tending the crystalline growth in one direction. It is known that by adding pozzolanic material to mortar or concrete mix, the pozzolanic reaction will only start when CH is released and pozzolan/CH interaction exist (Cocina et al., 2003).

The benefits of Pozzolanic materials is due to their physical and chemical characteristics, such as their effects on particle packing and their ability to provide amorphous silica to react with $Ca(OH)_2$ during the cement hydration reactions (Cordeiro et al., 2009).The advantage of using pozzolans cements is derived mainly from three features of the Pozzolanic reaction. First, the reaction is slow; therefore, the rates of heat liberation and strength development will be accordingly slow. Second, the reaction is lime-consuming instead of lime-producing, which has an important bearing on the durability of the hydrated paste in acidic environments. Third, pore size distribution studies of hydrated cements have shown that the reaction products are very efficient in filling up capillary spaces, thus improving the strength and impermeability of the concrete. The effect of the pozzolanic reaction produces more cement gel (i.e. C-S-H and C-A-H) reducing the pore size, blocks the capillary and produces denser concrete thus making it stronger and more durable.

The crystallized compound of C-S-H and C-A-H, which are called cement gel, hardened with age to form a continuous binding matrix with a large surface area and are components responsible for the development of strength in the cement paste (Kassim et al., 2004). Pozzolan-lime reactions are slow, generally starting after one or more weeks (Englehard et al., 1995). The behaviour of the delay in pozzolanic reaction will result in more permeable concrete at early ages and gradually becomes denser than plain concrete with time. This behaviour is due to two reasons: Firstly, pozzolan particles become the precipitation sites for the early hydration C-S-H and CH that hinders pozzolanic reaction. Secondly, the strong dependency of the breaking down of glass phase on the alkalinity of the pore water which could only attain the high pH after some days of hydration.

The addition of pozzolans to concrete/mortar has been observed to decrease the formation of CH and increased production of C-S-H gel that can improve the strength and durability of

concrete. Amorphous silica that is found in some pozzolanic materials (Habeeb et al., 2009) reacts with lime more eagerly than those of crystalline form (Lin et al., 2003).

2.3.3.3 Evaluation of Pozzolanic Activity

Evaluation of the pozzolanic activity of fly ash and other pozzolans falls into three categories: chemical, physical, and mechanical (Ali, 2014). A number of methods to assess the pozzolanic activity of different pozzolans have been proposed in various literatures. The most direct and also the most complicated is the accelerated chemical method, where the amount of Ca(OH)2 consumed in the reaction with a pozzolan is measured. The most widely-used method to measure pozzolanic activity, although the most time-consuming and more useful in practice than the evaluation of pozzolan-lime or pozzolan Portland cement reactivity, is the strength activity index method, where the compressive strengths of cement mortars with and without pozzolan are compared in percentage terms, according to ASTM C311 (2013)(Wansom, et al., 2009). Pozzolanic Activity Index (PAI) is defined as:

PAI (%) = strength (PC/ Pozzolanic mixture) x 100/ strength (PC mixture).

The XRD technique has been used to monitor the progress of the lime uptake in pozzolan Portland cement containing pozzolans. The results obtained by this method indicated good correlation between the lime combined in the reaction and the compressive strength of mortars at 6 months and 1 year (Ali, 2014).

2.3.4 Rice Husk Ash (RHA)

The milling of paddy rice (Oryza sativa) produces husk as an agricultural waste that is difficult to dispose of in an environmentally friendly way, and in Nigeria 748,000 - 990,000 tons of rice husk was projected to have been produced in 2010 based on estimated paddy rice production figure of 3.4 - 4.5 million tonnes (FAO, 2008; Flake and David, 2009) with about 10^8 tonnes of rice husk is generated annually in the world (Domke, et al., 2012). The rice being produced

could potentially produce 720,000 tonnes of RHA that can generate economic activity for rice farming communities and equally empower such communities to use this material for cheaper building construction.

Rice plant is one of the plants that absorbs silica from the soil and assimilates it into its structure during growth (Smith et al., 1986). Rice husk is the outer skin of the rice grain with a high concentration of silica, generally more than 80-85% (Siddique, 2008). It occupies about 30% of gross weight of a rice kernel and normally contains 80% of organic and 20% of inorganic substances (Anwar, et al., 2000). It can contribute about 20% of its weight to Rice Husk Ash (RHA) after incineration (Anwar et al., 2001).

It is a hard material with golden, brown or black colour, consisting of 15-20% silica, 50% of cellulose and 25-30% of lignin. On burning, cellulose and lignin are removed leaving behind silica ash. Completely burnt rice-husk is grey to white in color, while partially burnt rice husk ash is blackish (Ali, 2014). The controlled temperature and environment of burning yields better quality of rice-husk ash as its particle size and specific surface area are dependent on burning condition. The very high content of amorphous silica of the husk makes it and the ash obtained after combustion; a very valuable property for the production of cement. Burning of the husk under controlled temperature of about 800 ^{o}C produces about 25% ash containing 85% to 90% amorphous silica plus about 5% alumina, which makes it highly pozzolanic. However, according to Ali (2014), the reactivity of rice husk ash is generally decreased by the increase of burning temperature and the heating duration as this will result in the production of crystalline silica at 1000^{0}C and combustion time of over 5 minutes.

The most important constituents for any mineral admixtures are silica and alumina oxides. The non-crystalline silica and high specific surface area of the RHA are responsible for its high pozzolanic reactivity. According to Khan, et al. (2014) in their research, the microstructure of RHA through field emission scanning electron microscope (FESEM) after grinding has shown

that RHA has most uneven particle size distribution. Hence, it may infer that grinding procedure and duration are very important in case of RHA; which has particles of irregular shape having multiple layer structure. RHA has been used in lime-pozzolan mixes and could be a suitable partial replacement for Portland cement (Smith et al., 1986; Zhang et al., 1996; Nicole et al., 2000; Sakr, 2006; Sata et al., 2007; etc). Studies have shown that to obtain the required particle size, the RHA needs to pass through 45 μm–10 μm sieve size.

The works of Nehdi et al. (2003) and Rodrý´guez de Sensale (2006) indicate that country of origin, incineration technology and conditions of rice husk affect composition of RHA and its reactivity in concrete. Though different incineration methods have different peak temperatures, incinerating rice husk at temperatures of 750 ^{0}C and 830 ^{0}C has been proven to produce reactive RHA (Nehdi et al., 2003). Figure 2.1 shows rice unburnt rice husk and rice husk ash with various amount of burning while figure 2.2 shows the x-ray diffraction of a sample of RHA at various burning time and temperatures.

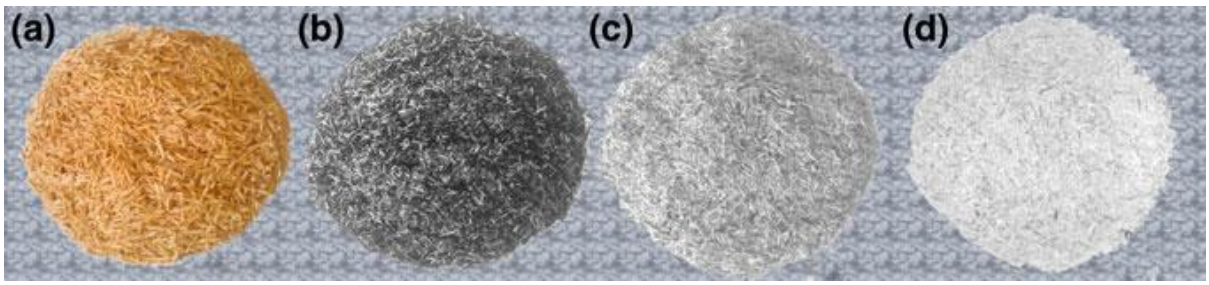

Fig. 2.1: a. Rice husk. b. High carbon RHA. c. Optimum RHA. d. RHA with crystalline silica (Ramezanianpour et al., 2009).

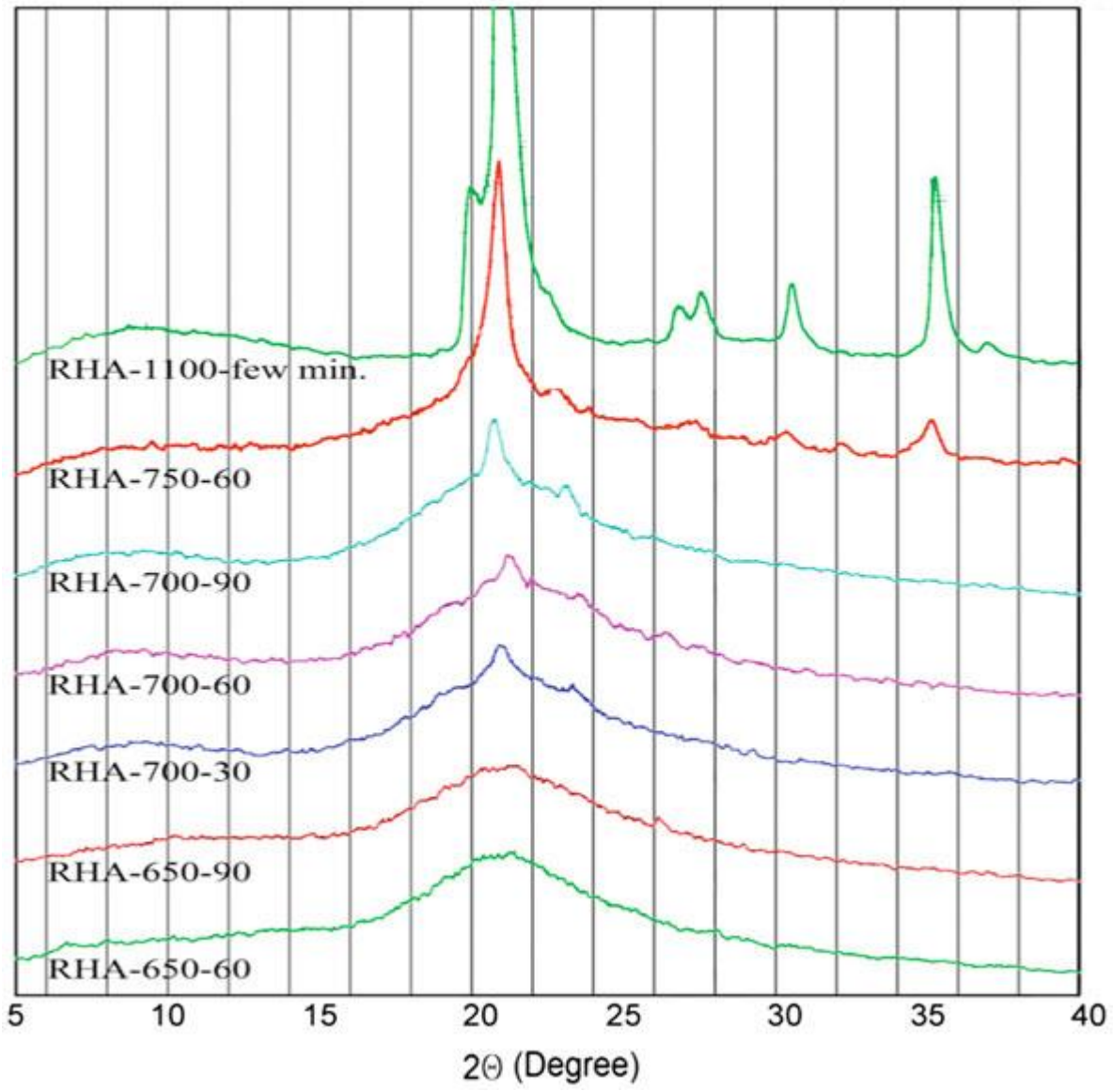

Fig. 2.2: Results of XRD on rice husk ash samples for different burning time and temperature (Ramezanianpour et al., 2009).

Tables 2.3-2.5 show the physical and chemical properties of RHA and SDA from previous studies.

Table 2.3: Physical Properties of RHA from previous researches

Property	Values			
	Mehta (1992)	Zhang et al. (1996)	Feng et al. (2004)	Bui et al. (2005)
Mean particle size (μm)	-	-	7.4	5.0
Specific gravity	2.06	2.06	2.10	2.10
Fineness: Passing 45 μm (%)	99	99	-	-

Table 2.4: Chemical Compositions of RHA from previous researches

Composition	Percentages		
	Mehta (1992)	Zhang et al. (1996)	Bui et al. (2005)
SiO_2	87.20	87.30	86.98
Al_2O_3	0.15	0.15	0.84
Fe_2O_3	0.16	0.16	0.73
CaO	0.55	0.55	1.40
MgO	0.35	0.35	0.57
Na_2O	1.12	1.12	0.11
K_2O	3.68	3.68	2.46
SO_3	0.24	0.24	-
LOI	8.55	8.55	5.14

2.3.5 Saw Dust Ash (SDA)

Saw dust is an organic waste resulting from the mechanical milling or processing of timber (wood) into various shapes and sizes. Sawdust is an industrial waste in the timber industry. It is obtained as loose particles or wood chippings from sawing of timber into standard useable sizes. This process is a daily activity causing heaps of saw dust to be generated each day. It poses a nuisance to both the health and environment when not properly managed. The dust is usually used as domestic fuel.

The resulting ash known as saw-dust ash (SDA) is a form of pozzolan. Dry saw dust concrete weighs only 30% as much as normal weight concrete and its insulating properties approximate those of wood. With proper cement to sawdust ratios, it is not flammable.

SDA is a latent hydraulic material and it contains mainly silicates of approximately 67%. It needs more water for consistency and when added to cement it triggers off a pozzolanic reaction with the excess $Ca(OH)_2$ produced during the cement hydration. Therefore, SDA deferred the hydration of the paste and extends the setting time. It has pozzolanic properties

and has been shown to react chemically with the calcium hydroxide released from the hydration of Portland cement, to form cement compounds (Elinwa and Mahmood, 2002 in Onwuka, et al., 2013).The advantages of using SDA for concrete production are numerous. It acts as a retarder prolonging the setting times, reduces the heat of hydration, encourages a healthier environment by reducing green gas emission and abundantly available as a waste. SDA has been used as partial replacement in mortar and concrete works (Elinwa and Mahmood, 2002; Elinwa and Ejeh, 2004 in Onwuka, et al, 2013). It has also been used as a powder material in the production of self-compacting concrete (SCC) (Elinwa and Mamuda, 2008) and in combination with metakaolin as a ternary blend with 3 % added to act as an admixture.

Table 2.5: Physical and Chemical Properties of SDA from previous researches

Property	Values			
	Marthong (2012)	Mageswari et al (2008)	Raheem et al (2012)	Raheem et al (2013)
Specific gravity	2.51	2.50	2.19	-
Bulk density (kgm^{-3})	-	1250	1040	-
Fineness modulus	-	1.78	-	-
P^H	11.12	-	-	-
Composition	Percentages			
SiO_2	50.20	65.3	65.75	65.31
Al_2O_3	1.02	4.0	5.23	6.09
Fe_2O_3	14.23	2.23	2.09	3.49
CaO	5.45	9.6	9.62	4.21
MgO	0.09	5.8	4.09	3.39
MnO	5.60	0.01	-	-
Na_2O	0.07	0.07	0.06	1.00
K_2O	9.57	0.11	2.43	11.09
P_2O_5	0.56	0.43	-	-
SO_3	0.58	0.45	1.09	2.89
LOI	3.67	-	4.30	-

2.4 Calcination Methods

Chemical compositions of pozzolans such as RHA and SDA have been observed to be affected due to burning process, the temperature of burning, heating rate and burn time. The quality of ash prepared depends on different heating conditions such as temperature, heating rate, burn

time, and grinding conditions. Silica content in pozzolanic ashes has been reported to increase with higher burning temperatures. Different methods of burning therefore produce Pozzolanic ashes with different characteristics.

Methods of burning Pozzolanic materials vary from local methods such as simple open air burning that produces Pozzolanic ashes with high carbon content to industrial processes such as fluidized bed and Torbed reactor technologies that produce ashes with lower carbon content. Fluidized bed combustion is used for industrial production of RHA; the technology for industrial production of RHA is adopted from existing technologies for solid fuel combustion (Abalaka, 2012).

Studies have shown that RHA and SDA resulting from the burning of rice husks and saw dust at controlled temperatures have physical and chemical properties that meet ASTM C 618(2015). Previous researches have indicated that to produce quality pozzolanic ashes, the raw materials should be burnt at a temperature of 750^0C-830^0C (Nehdi et al, 2003). This high temperature of burning will result in ash with low LOI and therefore, reduce water requirement and dosage of super plasticizer for workability of the blended cement concrete. Study by Houston (1972) shows that RHA produced by burning rice husk between 600 and 700°C temperatures for 2 hours, contains 90-95% SiO_2, 1-3% K_2O and < 5% unburnt carbon.

The following are the three calcination methods used in this research work.

2.4.1 Open- Air Calcination

In this calcination method, the sun dried rice husks and saw dust are burnt by placing them individually in heaps and allowed to burn in the open air for several hours, at temperature generally lower than about 650^0C until the ashes are formed. Studies showed that rice husks of Enyong Creek (Akwa Ibom state) wetland rice calcinated using the open heap method yielded

RHA with 94.47% total silica $(SiO)_2$ content, and LOI of 2.11% (Essien, 2006 in Abalaka et al., 2007).

2.4.2 Local Stove Calcination

In this calcination method, the rice husks and sawdust are placed in a container called the local stove and burnt. The local stove consists of a cylindrical metallic container with a perforation on its side, and a cover which has a hole at the top of the cylindrical container; just like the local stove used for cooking. The rice husks and saw dust are placed in the cylindrical container and allowed to burn for several hours, at temperature generally lower than about 650^0 C until ashes are formed.

The burning of rice husks from Minna metropolis using the perforated drum furnace produced RHA with 67.3% total SiO and LOI of 17.78 % (Abalaka et al., 2007).

2.4.3 Furnace Calcination

In this method, a pit crucible furnace is used to burn the rice husks and saw dust at a controlled temperature of about 700^0C at about 10^0C/min for several hours until the ashes are obtained. The heat for burning the rice husks and saw dust is supplied by charcoal and the temperature is measured using thermocouple.

This method has been reported to produce ash with high quality and low carbon content due to the fact that it produces a relatively uniform combustion temperature and avoids the extremes in temperature that occur in other types of combustion.

The study of Nehdi et al. (2003) showed that at 12.5% RHA replacement, RHA produced using a Torbed reactor operated at 750 ^{0}C, produced higher compressive strength at 28 days in concrete than RHA produced using fluidized bed technology.

Whatever the method of RHA and SDA production used, the challenge is in producing RHA and SDA that satisfy the standard requirements of the ASTM standard with maximum permissible LOI of 6%. LOI is a measure of residual carbon content of the ash and this carbon content varies according to the combustion process. According to ASTM C618 (2015) specification, the LOI must be less than 10% for mineral admixture class N and also less than 6% for class F and C. It is known that a high value of LOI results in increased water requirement and dosage of super plasticizer (Sata, et al., 2007). Thus, loss on ignition must be evaluated before using any agricultural by-product materials as partial cement replacement in concrete. The best temperature and time to obtain less proportion of unburnt carbon is when the burning temperature of 1000 C° and 800ºC were employed for 2–6 hours and 4.5-6 hours, respectively. Burning at higher temperatures and times is also necessary to obtain a loss on ignition in accordance with ASTM 618 (2015).

2.5 Properties of Fresh Concrete

Fresh concrete or plastic concrete is a freshly mixed material which can be moulded into any shape. The relative quantities of cement, aggregates and water mixed together, control the properties of concrete in the wet state as well as in the hardened state.

2.5.1 Workability

Workability of concrete is the ease with which concrete is mixed, transported and place. According to Road Research Laboratory, U.K., in Shetty (2006), workability is the property of concrete which determines the amount of useful internal work necessary to produce full compaction. Concrete passes through the processes of mixing, transporting, pouring, compacting and finishing. Workability has three components namely: consistency, mobility and compactibility. Mobility is the ease with which a mix can flow into and completely fill the formwork or mould. Consistency is a measure of the fluidity or wetness of the concrete.

Compactibility is the ease with which a given mix can be fully compacted and all the trapped air removed. The workability of fresh concrete can be determined using the slump test or the compacting factor test.

At w/c ratio of 0.45 the slump value of the concrete containing 5% RHA was the same compared to control. At higher w/b ratio of 0.50 and 0.55, the effect of the RHA on concrete slump indicated slump increase at 5% content, and subsequent increase in RHA content resulted in slump reduction. The higher slump recorded at 5% content was due to the effect of reduction in cement flocculation resulting from RHA addition (Abalaka, 2012). The reduction in the slump as RHA content increased, was due to water absorption of the RHA particles because of the cellular structure of the particles. There had been reported linear increase in water demand as RHA content increased in cement mortar at standard consistency (Abalaka et al., 2011). Due to the high specific area of finer RHA, there is increase in water demand. According to Khan et al. (2014) in their research, increase in fineness of RHA causes reduction in slump. Kene et al. reported that the workability of RHA concrete decreased with increase in RHA replacement. This is because concrete requires approximate increase in water cement ratio due to increase in percentage of RHA, since RHA is highly porous material. However, incorporation of RHA into concrete has been observed to enhance workability of concrete (Coutinho 2002; Habeeb et al., 2009; Mahmud et al., 2004).

Furthermore, workability decreased upon the inclusion of SDA. Thus, mixes containing SDA will require higher water content than the corresponding conventional mixes. This behaviour could be as a result of the fact that the higher the grade of cement the more fine it is. Finer cement requires more water to wet the surface particle. Hence, more water is required for wetting the particles, as the total surface area of the particle is increased (Marthong, 2012). Concrete becomes less workable as the SDA percentage increases meaning that more water is required to make the mixes more workable. This means that SDA concrete has higher water

demand. The high demand for water as SDA increases is due to increased amount of silica in the mixture (Raheem and Sulaiman, 2013).

In general, water demand greatly depends on the particle size, specific surface area, particle shape, replacement level and reactivity of particle mineral admixture used in concrete. General, smaller particle size and higher specific surface of mineral admixture increase the water demands of concrete for the desired workability to be attained.

2.5.2 Setting Time

The setting of concrete is the onset of solidification in a fresh concrete mixture. The setting time of concrete depends upon the water-cement ratio, temperature conditions, type of cement, use of mineral admixtures, and use of plasticizers.

Setting times increased in all grades of OPC upon the addition of SDA but are in the range recommended for pure cement. Setting time of cement with inclusion of SDA increased as compared to pure cement paste. This behaviour may be due to the low rate of hydration in the paste containing SDA (Marthong, 2012). Cement content is usually accompanied by high rate of hydration and it is usual that the reduction in cement content by replacing with SDA which is virtually inert at the early age would be accompanied by a low heat of hydration.

The studies by Ganesan et al. (2008), and Cook (1986), showed that RHA increases the setting time of pastes. Just like other hydraulic cement, the reactivity of rice husk ash cement depends very much upon the specific surface area or particle size. The rice husk ash cement with finer particles exhibits superior setting time behaviour. Dakroury et al. (2008) contended that this may be due to the slower pace of heat induced evaporation of water from the cement–RHA. Zhang, et al. (1996) in their studies comparing silica fume (SF) and rice husk ash (RHA), observed the initial setting time and final setting time of RHA to be about 29 and 60 minutes respectively longer than that of SF.

2.5.3 Bleeding

Bleeding is a particular form of segregation in which some of the water from concrete comes out to the surface of the concrete, being of the lowest specific gravity among all the ingredients of concrete. Sometimes, along with the water that comes out, certain quantity of cement also comes to the surface and this can reduce the amount of cement available for hydration. Addition of RHA and SDA has been observed to reduce bleeding of the concrete mix. Zhang et al. (1996) reported that concrete containing RHA does not produce bleeding water in concrete.

2.5.4 Segregation

Segregation is the separation of the constituent materials of concrete. Segregation causes weakness in concrete and lack of homogeneity is also going to induce all undesirable properties in the hardened concrete. According to Givi et al. (2010) the use of rice husk ash leads to enhanced resistance to segregation of fresh concrete compared to a control mixture with Portland cement alone.

2.6 Properties of Hardened Concrete

2.6.1 Flexural Strength

Neville (1981), rightly observed that concrete is among the most commonly used structural materials alongside steel. The knowledge of the properties of concrete makes possible the selection of a more suitable economic mix. The strength of the concrete is of utmost importance. It is the maximum load (stress) that the concrete can carry (Jackson and Ravindra, 1996). As the strength of concrete increases, its other properties usually improve. Strength is the most desired quality of a good concrete. It should be strong enough, at hardened state, to resist the various stresses to which it would be subjected.

Flexural strength is one measure of the tensile strength of concrete. It is a measure of an unreinforced concrete beam or slab to resist failure in bending. Flexural strength, also known as modulus of rupture, bend strength, or fracture strength is a material property defined as the stress in a material just before it yields in flexure test. It is the maximum stress at the outermost fibre on either the compression (inside of the bend or concave face) or tension side (outside of the bend or convex face) of the specimen. Flexural tests are generally used to determine the flexural modulus or flexural strength of a material.

The transverse bending test is most frequently employed, in which a specimen having either a circular or rectangular cross-section is bent until fracture or yielding using a three point flexure test technique. It is measured by loading 6 x 6-inch (150 x 150mm) concrete beams with a span length at least three times the depth. The material is laid horizontally over two points of contact (lower support span) and then a force is applied to the top of the material through either one or two points of contact (upper loading span) until the sample fails as shown in figure 2.3. The flexural strength is expressed as Modulus of Rupture (MR) in psi (MPa) and is determined by standard test methods ASTM C78, 2010 (third-point loading) as shown in figure 2.4, or ASTM C 293, 2010 (centre-point loading).

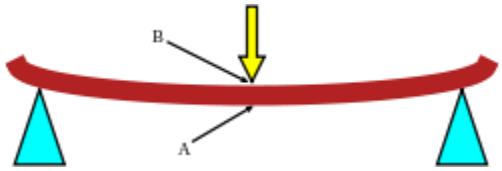

Fig 2.3: Beam of material under bending. Extreme fibres at B (compression) and at point A (tension).

2.6.1.1 Types of Flexural Tests

The two most common types of flexural tests are the:

i. **Three – point flexural bending test**

A three-point flexural bending test consists of the sample placed horizontally upon two points and the force applied to the top of the sample through a single point so that the sample is bent in the shape of a “V”. The three point flexural bending comprises the third-point loading and the centre-point loading. In the third-point loading, half the load is applied at each third of the span length as shown in figure 2.4. Maximum stress is present over the centre third portion. In the centre-point loading, the entire load is applied at the centre of the span. Maximum stress is present only at the centre of the beam. The three-point flexural test is ideal for the testing of a specific location of the sample.

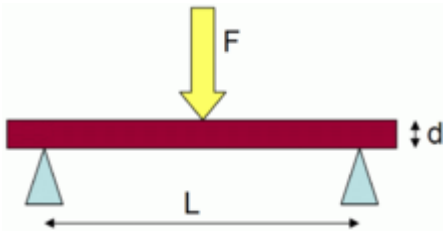

Fig 2.4: Beam under three-point bending

For a rectangular sample, the resulting stress under an axial load or force is given by the formula:

$$\sigma = \frac{F}{bd} \tag{2.12}$$

This stress is not the true stress, since the cross section of the sample is considered to be invariable (engineering stress).

The resulting stress for a rectangular sample under a load in a three-point bending setup is given by the formula:

$$\sigma = \frac{3FL}{2bd^2} \tag{2.13}$$

Usually, L is much bigger than d, so the fraction $\frac{3L}{2d}$ is bigger than one (1), where F is the axial load (force) at the fracture point (N),

L is the length of the support span (mm),

b is the width of the beam or sample (mm),

d is the thickness of the beam or sample (mm).

Flexural strength is about 10 to 20 per cent of compressive strength depending on the type, size, and volume of coarse aggregate used. The MR determined by the third-point loading is lower than the MR determined by the centre-point loading, sometimes by as much as 15 %.

ii. Four – point flexural bending test

A four-point flexural bending test is roughly the same as a three-point flexural bending test except that instead of the force applied through a single point on top, it is applied through two points so that the sample experiences contact at four different points and is bent more in the shape of a "U" as shown in figure 2.5. The four-point flexure test is more suited towards the testing of a large section of the sample, which highlights the defects of the sample better than a three-point test.

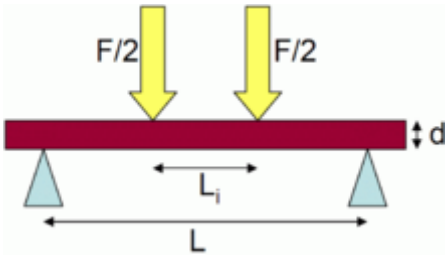

Fig. 2.5: Beam under four-point bending.

For a rectangular sample under a load in a four-point bending setup, where the loading span is one-third of the support span:

$$\sigma = \frac{FL}{bd^2} \qquad (2.14)$$

For the four-point bending setup, if the loading span is $\frac{1}{2}$ of the support span (i.e $L_i = \frac{1}{2} L$):

$$\sigma = \frac{3FL}{4bd^2} \quad (2.15)$$

If the loading span is neither $\frac{1}{3}$ nor $\frac{1}{2}$ the support span for the four-point bending setup:

$$\sigma = \frac{3F(L-Li)}{2bd^2}, \quad (2.16)$$

where L_i is the length of the loading (inner) span.

2.6.1.2 Significance of Flexural Strength and Effect of Pozzolans (RHA and SDA) on Flexural Strength

Flexural tensile strength of concrete is an important parameter for designing flexural members. Unlike a compression test or tensile test, a flexural test does not measure fundamental material properties. When a specimen is placed under flexural loading all three fundamental stresses are present: tensile, compressive and shear; and so, the flexural properties of a specimen are the result of the combined effect of all three stresses as well as (though to a lesser extent) the geometry of the specimen and the rate load is applied.

The flexural strength of concrete, increases with increase in age of concrete (Ahmed et al., 2014). The deflection and cracking behaviour of concrete structure depend on the flexural tensile strength of concrete. Many factors have been shown to influence the flexural tensile strength of concrete, particularly the level of stress, size, age and confinement to concrete flexural member, etc. Therefore, these factors are of prime importance while studying the flexural tensile strength of concrete.

According to the research of Kene (2004) the maximum 28 days flexural strength was obtained with 5% rice husk ash mix. The use of rice husk ash in concrete mix has been observed to increase compressive and flexural strengths (Zhang et al., 1996; Ismaila, 1996; Rodriguez

2005). Patel and Singh (2014) observed that the flexural strength of beam specimen kept on decreasing as the percentage of RHA increases and that the value of flexural strength for 20% replacement by RHA increased in comparison to 15% replacement during 7 day and 28 day tests. RHA blended concrete can improve the compressive strength as well as the tensile and flexural strength of concrete. RHA helps in enhancing the early age mechanical properties as well as long-term strength properties of cement concrete (Givi et al, 2010).

Habeeb et al. (2009), and Sakr (2006) in their respective studies reported that the use of RHA in concrete improved tensile strength and flexural strength of concrete significantly with increasing RHA replacement. Habeeb et al. (2009) reported that the coarser RHA particle mixture showed the least improvement in tensile and flexural strength. Zhang et al. (1996) concluded that the addition of RHA to concrete exhibited an increase in the flexural strength and the higher strength was for the finer RHA mixture due to the increased pozzolanic reaction and the packing ability of the RHA fine particles. Kartini et al. (2006) reported that the flexural strength of concrete specimens incorporating 20% RHA decreased as compared to the OPC (control). This is attributed to higher w/c ratio of the RHA concrete.

The Flexural strength of the beam of the concrete for all mix increases with age of curing and decreases as the SDA content increases (Mageswari, and Vidivelli, 2009).

2.6.2 Water Absorption

Marthong (2012) reported that water absorption of SDA concrete up to 20% replacement decreased with increase in grades of OPC. Thus, permeability of paste with coarser cement particle is higher. The water absorption of SDA concrete also varies with age of concrete. The results also depict that water absorption too decreased with age. With age the water absorption decreased because gel gradually fills the original water filled spaces.

Literature studies have identified that commonly permeability of blended cement concrete is less than plain cement paste. It was observed that the incorporation of RHA in the composites

could cause an extensive pore refinement in the matrix and in the interface layer, thereby decreasing water permeability (Rodrigues et al., 2006). The permeability will decrease rapidly with the progress of the hydration. The presence of pozzolan leads to greater precipitation of cement gel products (Feng et al., 2004) than occurs in Portland cement alone, which more effectively block the pores helping to reduce permeability. Saraswathy et al. (2007) studied the effect of partial replacement of cement with RHA at different replacement levels on the porosity and water absorption of concrete and reported that the coefficient of water absorption for rice husk ash replaced concrete at all levels was less than control concrete. RHA blended concrete can decrease the total porosity of concrete and modifies the pore structure of the cement, mortar, and concrete, and significantly reduce the permeability which allows the influence of harmful ions leading to the deterioration of the concrete matrix (Givi et al, 2010).

2.6.3 Durability

Inclusion of SDA as partial replacement of cement does not improve the durability when exposed to sulphate environment. This is because for each grade of cement the strength of ordinary cube and that partially replaced by SDA immersed in sulphate solution have less compressive strength than the corresponding referral cubes immersed in tap-water (Marthong, 2012). However, Coutinho (2002) reported an increased durability of concrete containing RHA. Incorporation of rice husk ash with chemical treatment, ChRHA in concrete exhibited lower permeability and hence, enhances durability properties by refining its pore structure. The total absorption and porosity values, lower than 3 and 10% respectively, are useful parameters to describe compaction and durability (Rastogi and Barhadiya, 2013).

2.6.4. Bulk Density

Bulk density or unity weight shows how densely the aggregates are packed filled in standard manner in the concrete (Shetty, 2006). The bulk density depends on the particle size

distribution and shape of the particles. The parameter of bulk density is used in concrete mix design for converting the proportions by weight into proportions by volume when weigh batching equipment is not available. Specific gravity of mineral admixtures has been observed to be lower or lesser than specific gravity of ordinary Portland cement (OPC). Therefore, more volume is expected when any mineral admixture replaces OPC by mass.

The addition of mineral pozzolans such as rice husk ash (RHA) and saw dust ash (SDA) have been reported to reduce the unit weight of blended concrete. RHA when added in the concrete reduces the weight of the concrete up to 15% after 90 days of curing (Kene, 2004). Kulkarn, et al. (2014) the reported that the bulk density of RHA concrete reduced with increase in RHA content. The change in density is minimal with addition of rice husk ash but there is much difference when sawdust is used (Aliu, and Daramola, 2013). Mageswari and Vidivelli (2009) in their study 'the use of sawdust ash as fine aggregate replacement in concrete' reported that the density of the concrete was 2502 kg/m^3 at 0% replacement of SDA but at 30% replacement of SDA, the density decreased to 2341 kg/m^3.

2.7 Regression Analysis

Regression analysis is a statistical tool for the investigation of relationships between variables. Usually, the investigator seeks to ascertain the causal effect of one variable upon another. Regression models involve linear transformation of the predictor variable into the predicted variable. Regression analysis is used to predict a continuous dependent variable from a number of independent variables (Tabachnick and Fidell, 1989). The purpose of regression analysis is to come up with an equation of a line that fits through the cluster of points with the minimal amount of deviations from the line. The deviation of the points from the line is called "error." A regression line of a variable Y on X is an equation or model that expresses the relationship between Y (the dependent variable) and X (the independent variable) (Inyama and Iheagwam,

1995). Independent variables (IVs) used in regression analysis can be either dichotomous (i.e. only two variables) or continuous (i.e. more than two variables). A widely used procedure for obtaining the regression line of Y on X is the least square method developed by Carl Friedrich Gauss, a German mathematician (Tabachnick and Fidell, 1989).

2.7.1 Types of Regression Analysis

Regression analysis could be simple regression analysis or multiple regression analysis.

i. Simple Linear Regression

A regression line involving only two variables is called a simple regression line. Simple linear regression is used to predict values of one variable, given the values of another variable (Anyiwe, 1991), e.g. predicting the effect of curing age on the compressive strength of concrete.

ii. Multiple Regression

The general purpose of multiple regressions is to learn more about the relationship between several independent or predictive variables and a dependent or criterion variable (Tabachnick and Fidell, 1989). Multiple regressions are techniques that allow additional factors to enter the analysis separately so that the effect of each can be estimated. It is valuable for quantifying the impact of various simultaneous influences upon a single dependent variable. Further, because of omitted variables bias with simple regression, multiple regressions are often essential even when the investigator is only interested in the effects of one of the independent variables. Multiple regression analysis is in fact capable of dealing with an arbitrarily large number of explanatory variables.

Standard multiple regression uses several independent variables to predict the dependent variable (Anyiwe, 1991). For instance, predicting the flexural strengths of concrete from the curing age and percentage replacement of OPC with pozzolanic ashes such RHA and SDA.

The resulting output would reveal how much of the variance of flexural strength was accounted for by the joint predictive power of curing age and replacement level. In addition to revealing the predictive value of the overall model, standard multiple regression shows how well each independent variable predicts the dependent variable, controlling for each of the other independent variables.

2.7.2 Regression Correlation Coefficient, R

Whenever there is some connection or dependence between two or more variables, we have what is called correlation. Correlation coefficient is the actual measure of the amount of relationship that exits between two given variables (i.e. independent variables and dependent variables). Regression correlation coefficient is used to determine the direction of the relationship between the independent variables and the dependent variables by looking at the regression coefficient associated with each independent variable (Tabachnick and Fidell, 1989). The degree to which two or more predictors (independent or X variables) are related to the dependent (Y) variable is expressed in the correlation coefficient R, which is the square root of R-square.

There are two kinds of regression coefficients namely:

i. B or unsaturated coefficients

The B coefficient associated with each variable is given in terms of the units of each variable.

ii. Beta or standard coefficients

The beta coefficient uses a standard unit that is the same for all variables in the equation. Beta coefficients are useful because it helps to compare two variables that are measured in different units.

A regression coefficient may be positive or negative. If the regression coefficient is positive, then there is a positive relationship between the dependent variable and the independent variables. In other words, a regression correlation is said to be positive if an increase in one

variable (IVs) is accompanied by an increase in the other (DV). But if the regression coefficient is negative, then there is a negative relationship between the dependent variable and the independent variables, i.e. an increase in one variable (IVs) is accompanied by a decrease in the other variable (DV) (Tabachnick and Fidell, 1989).

The regression coefficient, R falls between -1 and 1i.e. $-1 \leq R \leq -1$. If $0.5 \leq R \leq 1$ or $-1 \leq R \leq -0.5$, we say that there is a good correlation. But if $R \leq 0.5$ or $0 \geq R \geq -0.5$, we say that there is a poor or non-existent correlation. When $R = +1$, we have a perfect or hundred per cent correlation, but when $R = 0$, there is no correlation.

2.7.3 Significance Predictor (P-Values)

To check if each of the independent variables is significant predictor of the dependent variable, the significance level associated with the independent variables should be checked in the print out (Tabachnick and Fidell, 1989). The p-values for each term test the null hypothesis that the coefficient is equal to zero (no effect). Significance levels of 0.05 and lower would be considered significant, and significance levels between 0.05 and 0.1 would be considered marginal. A low p-value (<0.05) indicates that the null hypothesis can be rejected. Conversely, a large (insignificant p-value) suggests that changes in the predictor are not associated with changes in the responses.

An independent variable that is a significant predictor of a dependent variable in a simple linear regression may not be significant in multiple regressions (i.e. when other IVs are added into the equation). This could happen because the variance that the first IV shares with the DV could overlap with the variance that is shared between the second IV and the DV. Therefore, the first IV is no longer uniquely predictive and thus, would not show up as being significant in the multiple regressions.

2.7.4 The Regression Equation

A line in a two dimensional or two-variable space is defined by the equation

$$Y = a + bX, \quad (2.17)$$

i.e. the Y variable can be expressed in terms of a constant (a) and a slope (b) times the X variable. The constant is also known as the intercept, and the slope as the regression coefficient or B coefficient.

In a multivariate case, when there is more than one independent variable, the regression line cannot be visualized in the two dimensional space, but can be computed just as easily. In general then, multiple regression procedures will estimate a linear equation of the form:

$$Y = a + b_1X_1 + b_2X_2 + \ldots + b_pX_p \quad (2.18)$$

In this equation, the regression coefficients (or B coefficients) represent the independent contributions of each independent variable to the prediction of the dependent variable.

$$b = \frac{(\Sigma x_2^2)(\Sigma x_1 y) - (\Sigma x_1 x_2)(\Sigma x_2 y)}{(\Sigma x_1^2)(\Sigma x_2^2) - (\Sigma x_1 x_2)^2} \quad (2.19)$$

$$c = \frac{(\Sigma x_1^2)(\Sigma x_2 y) - (\Sigma x_1 x_2)(\Sigma x_1 y)}{(\Sigma x_1^2)(\Sigma x_2^2) - (\Sigma x_1 x_2)^2} \quad (2.20)$$

$$a = \bar{y} - b\bar{x}_1 - c\bar{x}_2 \quad (2.21)$$

$$\Sigma x_1 y = \Sigma X_1 Y - \frac{(\Sigma X_1)(\Sigma Y)}{N} \quad (2.22)$$

$$\Sigma x_2 y = \Sigma X_2 Y - \frac{(\Sigma X_2)(\Sigma Y)}{N} \quad (2.23)$$

$$\Sigma x_1 x_2 = \Sigma X_1 X_2 - \frac{(\Sigma X_1)(\Sigma X_2)}{N} \quad (2.24)$$

i. Predicted and Residual Scores

The regression line expresses the best prediction of the dependent variable (Y), given the independent variables (X). However, there is no perfect prediction, and usually there is substantial variation of the observed points around the fitted regression line. The deviation of a particular point from the regression line (its predicted value) is called the residual value.

ii. R-Square Variance and R-Square

R-Square, also known as the coefficient of determination is a commonly used statistic to evaluate model fit. R-square is 1 minus the ratio of residual variability. When the variability of the residual values around the regression line relative to the overall variability is small, the predictions from the regression equation are good. In most cases, the ratio of variance and R-square fall between 0.0 and 1.0. The R-square value is an indicator of how well the model fits the data. For instance, an R-square close to 1.0 indicates that almost all the variability with the variables specified in the model has been accounted for.

$$R^2 = \frac{b\Sigma(x_i - \bar{x})(y_i - \bar{y})}{\Sigma(y_i - \bar{y})^2} \qquad (2.25)$$

CHAPTER THREE

MATERIALS AND METHOD

3.1 Materials

3.1.1 Rice Husk Ash (RHA) and Saw Dust Ash (SDA)

The saw dust used in this work was obtained from the Timber Market in Owerri, Imo State; the rice husk was obtained from a rice mill in Afikpo, Ebonyi State, both in South Eastern Nigeria.

The rice husk and saw dust were adequately sun dried to remove moisture from them so as to facilitate burning. The pozzolanic ashes used for this project work were obtained by burning the rice husk and saw dust using three different calcination methods of open-air burning, local stove combustion, and furnace calcination using charcoal fired pit crucible furnace at a temperature of 800^0C for four hours.

The pozzolanic ashes (RHA and SDA) obtained were then sieved with a 300 micron sieve after cooling, to obtain very fine particles. The particles retained on the 300 μm sieve were either re-burnt or discarded, while those passing the sieve aperture were stored in air-tight containers. No grinding or any special treatment to improve the quality of the ash and enhance its pozzolanicity was applied because the researcher wanted to utilize simple processes that could be easily replicated by local community dwellers.

The RHA had specific gravity of 1.79, 1.65, and 1.66 for RHA-OA, RHA-F, and RHA-S respectively; while the SDA had corresponding values of 1.85, 1.56, and 1.60 for SDA-OA, SDA-F, and SDA-S respectively. Also, RHA had bulk densities of 850kgm^{-3}, 795kgm^{-3}, and 810kgm^{-3} for RHA-OA, RHA-F, and RHA-S respectively; while SDA had corresponding values of 830kgm^{-3}, 815kgm^{-3} and 840kgm^{-3} for SDA-OA, SDA-F and SDA-S respectively. RHA-OA, RHA-F, and RHA-S had fineness modulus of 1.32, 1.53, and 1.44 respectively;

while those of SDA-OA, SDA-F and SDA-S were 1.42, 1.53, and 1.53 respectively. These physical properties of RHA and SDA compare with the ones obtained from some previous studies shown in Tables 2.3-2.5.

3.1.2 Ordinary Portland Cement

The Dangote brand of ordinary Portland cement (OPC) used for this work was purchased at Nkwo-Ukwu Ihiagwa Market in Owerri West L.G.A., Imo State. It had a bulk density of 1675 kgm^{-3} and specific gravity of 3.13.

3.1.3 Fine and Coarse Aggregates

The fine aggregate (river sand) used in this work was obtained from Otamiri River in Umuchima, Ihiagwa in Owerri West Local Government Area of Imo State. The coarse aggregate (sandstone) used was obtained from Okigwe in Imo State. Both were free of clay, loam, dirt and any organic matter with the river sand having a bulk density of 1560kgm^{-3}, specific gravity of 2.17, and fineness modulus of 4.28; while the sandstone had a bulk density of 1635kgm^{-3}, specific gravity of 2.77, fineness modulus of 4.40, abrasion value of 53.52% and impact value of 28.14%.

3.1.4 Water

The water used in this work was potable. It was obtained from a tap of a borehole in the university and was stored in clean GP tank.

3.2 Method

3.2.1 Batching of Constituent Materials

Batching refers to the measurement of the proportions of the various materials for the concrete making. The method of batching could be volume batching or weight batching. Weight batching was used in this research work. Batching by weight involves the use of very sensitive

weighing balance to weigh out each constituent of the concrete mix. Before the batching was done, computations were done to determine the exact amounts in kilogram for each constituent material using a mix ratio of 1:2:4 and a water to cement ratio of 0.6.

Table 3.1 shows the total quantities of batched constituent materials for 9 OPC (control) concrete beams, 81 OPC-RHA concrete beams, and 81 OPC-SDA concrete beams, making a total of 171 concrete beams.

Table 3.1: Total Quantities of Constituent Materials of OPC, OPC-RHA, and OPC-SDA Concrete Beams.

Material	OPC	Sharp Sand (F.A)	Sandstone (C.A)	Water	RHA	SDA
Mass (kg)	386.46	1020.60	2040.12	305.64	38.34	38.34

3.2.2 Mixing of Specimens

The tools used for the mixing and moulding of the concrete include: tamping rod, scoop, hand trowel, shovel, bucket, and spanner.

Mixing of the concrete was done by hand on a smooth concrete surface. The smooth concrete surface where the concrete materials were mixed was properly cleaned of dirt and dust and was wetted with water and allowed to surface dry, so as to check possible absorption of water from the mix by the surface. The weighed fine aggregate sample was spread on the hard floor and on its top, the cement was uniformly spread and the two materials were thoroughly mixed using a shovel to turn it several times. For the batch containing the pozzolans (rice straw ash and saw dust ash); the cement and pozzolans were added to the fine aggregate at various replacement levels and then thoroughly mixed until a uniform mix was obtained. Weighed out water was added gradually as mixing continued until the entire water was added. The entire concrete was thoroughly mixed until a homogeneous and consistent mix of the concrete was obtained.

3.2.3 Moulding of Concrete Beams

The moulding of the beams followed the procedure outlined in BS 1881: 1983 – Methods for making and curing of test specimens.

150 x 150 x 600 mm moulds were used in this work. The moulds were first cleaned of all hardened concrete on it, weighed and then oiled to enable easy de-moulding of the concrete beams. The nuts of the moulds were properly tightened to ensure that the moulds were stable during moulding, in order to obtain the desired shapes of beams. The mixed fresh concrete was poured into the moulds in three layers using hand trowels, immediately after mixing so as to avoid evaporation of moisture from the fresh concrete. Each layer of the concrete was compacted by hand using the tamping rod to uniformly distribute about twenty five strokes over the cross section of the mould. This helps to expel the entrapped air in the concrete. When the moulds were filled, a hand trowel was used to strike off excess concrete and to smoothen the surface. The mould together with the compacted fresh concrete in it was weighed and the mass recorded in order to determine the wet density. Three beams were moulded for each batch of mix.

This gave a total of 171 concrete beams comprising the following:

i. 81 OPC-RHA concrete beams for the three calcination methods of OA, F, and S; consisting of 27 concrete beams for each curing age of 28, 90, and 150 days.
ii. 81 OPC-SDA concrete beams for the three calcination methods of OA, F, and S; consisting of 27 concrete beams for each curing age of 28, 90, and 150 days; and
iii. 9 control (0% RHA and SDA) concrete beams, consisting of 3 concrete beams for each curing age of 28, 90, and 150 days.

3.2.4 Curing of Specimens

The moulded concrete beams were stored in a place free from vibrations for 24 hours from the time of addition of water to the dry ingredients; after which they were de-moulded. The beams were cured by immersing them in water in a curing tank. The beams were cured for 28, 90, and 150 days respectively.

3.2.5 Laboratory Tests Procedure

3.2.5.1 Particle Size Distribution of Samples

Sieve analysis was performed on the fine aggregate (river bed sand) and the sand stone coarse aggregate in accordance with BS 410 (1986). Maximum sieve size of 2.36 mm was used for the river bed sand fine aggregate, while a maximum sieve size of 37.5 mm was used for the sandstone coarse aggregate. About 500 g of the each of the fine and coarse aggregate was poured into a set of firmly clamped sieves and the sieve shaker switched on for about 15 minutes. The mass retained on each sieve was weighed and the percentage passing (percentage finer) for each sieve was obtained.

The particle size distribution of the pozzolanic ashes was performed using the hydrometer method in accordance with ASTM D 422 (1998) and BS 1377 (Part 2: 1990), followed by sieving recommended for sediment size distribution and soil samples containing less than 35% fine particles. The results are shown in Tables B1 to B4 and Figures B1 to B8 of the appendices.

3.2.5.2 Specific Gravity Test

The specific gravity bottle was weighed empty and the mass recorded, M_1. The specific gravity bottle was filled with the specimen (fine aggregate, RHA, and SDA respectively) to the highest calibration mark, weighed and the mass recorded, M_2. Some quantity of the sample was put into the specific gravity bottle, and bottle filled with water was thoroughly shook to enable

water permeate into the sample and fill the voids. The bottle was allowed to stand for some time for the water to thoroughly permeate into the sample. The bottle and its content weighed and the mass recorded, M_3. The sample and water were removed from the specific gravity bottle, and the bottle thoroughly dried and cleaned. The bottle was filled with water and weighed and the mass recorded M_4. Three trials were done and the specific gravity obtained by dividing the total mass of solid particles by the mass of an equal volume of water displaced. The average of the three trials was taken as the specific gravity of the sample. For the sandstone coarse aggregate, a measuring cylinder was used and the same procedure was repeated. The results are shown in Tables C1 and C2 of the appendices.

3.2.5.3 Abrasion Test

The test was performed in accordance with ASTM C131 (2003). The sandstone coarse aggregate was first sieved and the aggregate retained on sieve 2.63 mm was washed and dried. About 5000g of the aggregate retained on the 2.36 mm sieve was placed in the abrasion testing machine and 25 charges (steel balls) were applied to the aggregate. The abrasion testing machine was powered on and allowed to grind the aggregate by completing about 500 revolutions. On completion, the crushed coarse aggregate was removed from the abrasion testing machine and sieved. The weight of aggregate that was retained on the 2.36 mm sieve was subtracted from the original weight to obtain a percentage of the total aggregate weight that has broken down and passed through the 2.36 mm sieve. This is the abrasion loss value. The result is shown in Table D1 of the appendices.

3.2.5.4 Aggregate Impact Value Test

The test was performed in accordance with BS 812: 1975. The sandstone coarse aggregate was washed and dried by direct sunlight for some hours. It was sieved using 12.5 mm and 10.0mm sieves. The aggregates passing through 12.5mm sieve and retained on 10.0mm sieve was

poured into the measuring cylinder in three layers and compacted by applying 25 blows to each layer with the tamping rod. Surplus aggregate was struck off and the weight determined. The impact machine was used to crush the aggregate by applying 25 blows. The crushed aggregate was sieved through a 2.36 mm sieve and the mass of the crushed aggregate determined. The mass of the crushed aggregate was expressed as a percentage of dry aggregate to obtain the impact value. The result is shown in Table D2 of the appendices.

3.2.5.5 Pozzolanic Reactivity

A simple form of pozzolanicity test was carried out for the ash. It consists of mixing a given mass of the ash with a given volume of Calcium hydroxide solution [$Ca(OH)_2$] of known concentration and titrating samples of the mixture against H_2SO_4 solution of known concentration at time intervals of 5, 20, 40, 60, and 90 minutes using Methyl Orange as indicator at room temperature and 75^0C. The titre value was observed to reduce with time, confirming the ash as a pozzolan that fixed more and more of the calcium hydroxide, thereby reducing the alkalinity of the mixture. The results are shown in Tables E1to E4 of the appendices.

3.2.5.6 Chemical Composition of Ash

The chemical analysis of the RHA and SDA was performed using X-ray Fluorescence spectrometer (XRF), in accordance with ASTM C1456 (2013). The samples were oven dried at 110^0C, unwanted materials removed and the samples reduced to < 2µm diameter by crushing. The samples were processed into pellets, transferred to clean prolene foil and then into a sample vial, labelled, arranged in the sample tray and finally transferred to the sample compartment of the XRF equipment and screened for their oxide composition. The results are shown in Table 4.1.

3.2.5.7 Slump Test

The internal surface of the mould was thoroughly cleaned from adherence of any old set concrete before commencing the test. The slump conical mould was placed on the horizontal, rigid and non-absorbent base plate**.** A homogenous concrete mix was produced and the mould was immediately filled with the fresh concrete in three layers, of approximately equal depth, to avoid evaporation of moisture, and each layer was tamped 25 times with the tamping rod to distribute the strokes evenly over the cross section. After the top layer has been rodded, the concrete was struck off level with a trowel and tamping rod. The mould was removed from the concrete immediately by raising it slowly and carefully in a vertical direction. This allowed the concrete to subside or slump. The difference in level between the height of the mould and that of the highest point of subsided or slumped concrete was measured with a metre rule. This difference in height in mm was taken as the slump of the concrete. The results are shown in Tables F1 and F2 and Figures F1 and F2 of the appendices.

3.2.5.8 Flexural Strength

The concrete beams after curing for the various ages of 28, 90 and 150 days respectively, were removed from the curing tank to the laboratory, two hours before conducting the test, to normalize the temperature and to make the beams relatively dry or free from moisture. The beams were weighed prior to crushing and the values recorded to obtain the dry unit weight. The universal flexural machine was set ready. The concrete beams were placed horizontally on the plates of testing machine and crushed using the four – point flexural test and the failure loads recorded. The resulting flexural strength for the rectangular concrete beam sample under a load in a four-point bending setup was calculated. The average flexural strength was the average of that of three beams tested for each trial. The results are shown in Tables 4.2 to 4.25.

3.2.5.9 Multiple Regression Analysis

Multiple regression analysis was applied to derive the equations in order to predict the flexural strengths of the concretes for each of the two pozzolanic ashes; using the three calcination

methods of open-air, furnace and stove. From equation 2.15, of the multiple regressions, the dependent variable Y, predicted in this work was the flexural strength. The independent variable X_1 was the curing ages of 28, 90, and 150 days; while the independent variable X_2 was the percentage replacement of OPC with RHA and SDA at 5%, 10%, and 15% respectively. Microsoft excel was used to perform the multiple regression analysis and the equations predicting the flexural strengths for each of RHA and SDA obtained from the three calcination methods were written. The results of the ANOVA, predicted flexural strengths are shown in Tables 4.26 to 4.37 as well as in equations 4.1 to 4.6.

The test for adequacy of multiple regression analysis was done using statistical student's t-test at 95% accuracy level. The actual or laboratory flexural strengths and the predicted flexural strengths were used for the test. The following two hypotheses were tested using statistical student's t-test.

a) **Null Hypothesis:** There is no significant difference between the flexural strengths of the laboratory concrete beams and the predicted flexural strengths at 95% accuracy level. H_0: $\mu_1 - \mu_2 = 0$. (3.1)

b) **Alternative Hypothesis:** There is a significant difference between the flexural strengths of the laboratory concrete beams and the predicted flexural strengths at 95% accuracy level. H_0: $\mu_1 - \mu_2 \neq 0$. (3.2)

CHAPTER FOUR

RESULTS AND DISCUSION

4.1 Presentation of Results

4.1.1 Chemical Composition of RHA and SDA

Table 4.1 shows the elemental oxides of RHA and SDA from the different calcination methods.

Table 4.1: Chemical Composition of RHA and SDA

Oxide	Composition of RHA (%)			Composition of SDA (%)		
	Open-Air	Furnace	Stove	Open-Air	Furnace	Stove
SiO_2	85.79	85.62	86.37	63.12	78.79	61.71
Al_2O_3	1.45	1.14	1.68	2.16	1.64	4.74
CaO	1.38	2.02	1.22	12.43	3.28	13.52
MnO	0.18	0.18	0.18	0.24	0.24	0.41
ZnO	0.03	0.03	0.03	0.03	0.03	0.03
Fe_2O_3	0.43	0.50	0.45	0.75	1.59	2.72
MgO	2.45	2.51	2.32	7.50	3.27	2.94
K_2O	1.82	1.80	1.77	5.08	3.37	3.72
Na_2O	0.61	0.63	0.67	1.64	0.86	2.10
SO_3	0.79	0.72	0.62	2.59	1.18	3.34
(SiO_2 + Al_2O_3 + Fe_2O_3)	87.67	87.26	88.50	66.03	82.02	69.17

4.1.2 Effect of Replacement of OPC with RHA and SDA on Flexural Strength

Table 4.2 show the flexural strength of control concrete, while Tables 4.3 and 4.4 show the effect of partially replacing OPC with RHA and SDA on the flexural strength of concrete.

Table 4.2: Flexural Strength of Control (100 % OPC) Beam

Beam Mark	Test Load (KN)			Flexural Strength (Nmm^{-2})			Average Flexural Strength (Nmm^{-2})		
	28 Days	90 Days	150 Days	28 Days	90 Days	150 Days	28 Days	90 Days	150 Days
1	52.0	54.0	56.0	5.78	6.00	6.22	5.89	6.18	6.41
2	54.0	57.0	58.0	6.00	6.33	6.44			
3	53.0	56.0	59.0	5.89	6.22	6.56			

Table 4.3: Flexural Strength of RHA-F Concrete Beam

Beam Mark	Percentage Replacement	Test Load (KN)			Flexural Strength (Nmm^{-2})			Average Flexural Strength (Nmm^{-2})		
		28 Days	90 Days	150 Days	28 Days	90 Days	150 Days	28 Days	90 Days	150 Days
1		27.0	51.0	60.0	3.00	5.67	6.67			

2	5	34.0	49.0	60.0	3.78	5.44	6.67	3.56	5.48	6.74
3		35.0	48.0	62.0	3.89	5.33	6.89			
1	10	37.0	44.0	53.0	4.11	4.88	5.89	4.07	5.03	6.15
2		39.0	45.0	57.0	4.33	5.00	6.33			
3		34.0	47.0	56.0	3.78	5.22	6.22			
1	15	26.0	40.0	48.0	2.89	4.44	5.33	3.04	4.41	5.32
2		24.0	38.0	49.0	2.67	4.22	5.44			
3		32.0	41.0	46.5	3.56	4.56	5.17			

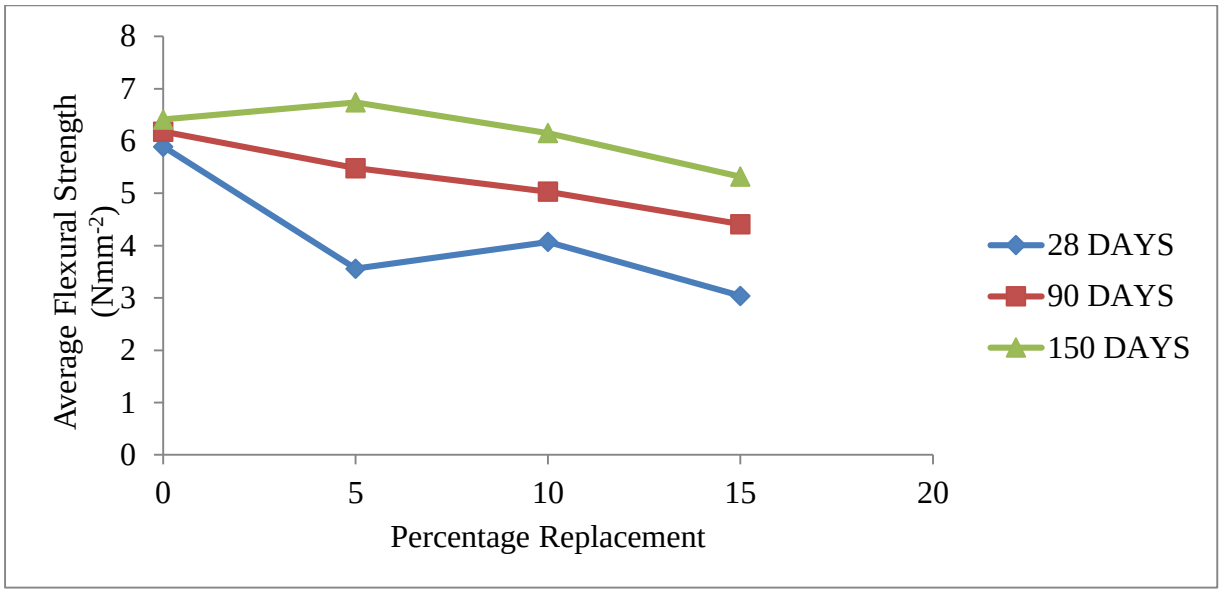

Figure 4.1: Effect of Replacement of OPC with RHA on Flexural Strength

Table 4.4: Flexural Strength of SDA-F Concrete Beam

Beam Mark	Percentage Replacement	Test Load (KN)			Flexural Strength (Nmm^{-2})			Average Flexural Strength (Nmm^{-2})		
		28 Days	90 Days	150 Days	28 Days	90 Days	150 Days	28 Days	90 Days	150 Days

1	5	34.0	45.0	51.0	3.78	5.00	5.67	3.70	5.07	5.85
2		36.0	47.0	52.0	4.00	5.22	5.78			
3		30.0	45.0	55.0	3.33	5.00	6.11			
1	10	28.0	40.0	40.0	3.11	4.44	4.44	2.89	4.11	4.70
2		26.0	37.0	45.0	2.89	4.11	5.00			
3		24.0	34.0	42.0	2.67	3.78	4.67			
1	15	18.0	25.0	30.0	2.00	2.78	3.33	2.22	3.07	3.70
2		20.0	30.0	37.0	2.22	3.33	4.11			
3		22.0	28.0	33.0	2.44	3.11	3.67			

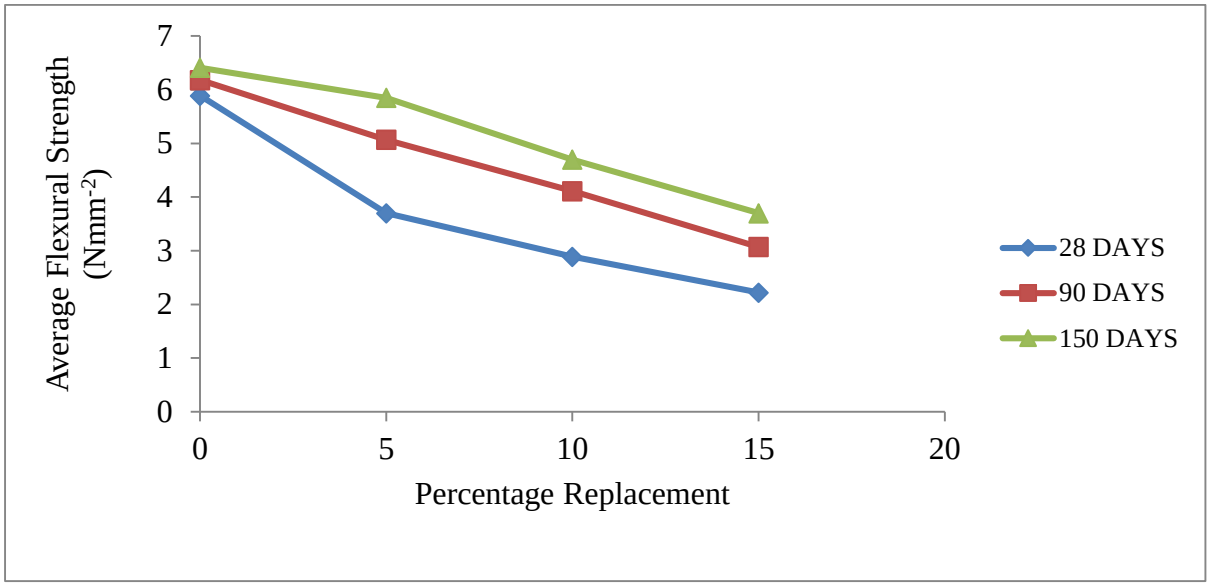

Figure 4.2: Effect of Replacement of OPC with SDA on Flexural Strength

4.1.3 Effect of RHA and SDA Calcination Methods on the Strength of Concrete

The variation of the flexural strengths of OPC-RHA cement composites from open-air, furnace and local stove calcination methods are shown in Tables 4.5, 4.6 and 4.7 and Figures 4.3, 4.4 and 4.5 below.

Table 4.5: Flexural Strength of RHA-OA Concrete Beam

Beam Mark	Percentage Replacement	Test Load (KN)			Flexural Strength (Nmm^{-2})			Average Flexural Strength (Nmm^{-2})		
		28 Days	90 Days	150 Days	28 Days	90 Days	150 Days	28 Days	90 Days	150 Days

1	5	40.5	41.0	45.5	4.50	4.56	5.06	4.57	4.97	5.35
2		48.5	43.0	47.0	5.39	4.78	5.22			
3		34.5	50.0	52.0	3.83	5.56	5.78			
1	10	20.0	43.0	45.0	2.22	4.78	5.00	3.04	4.06	4.32
2		37.0	34.5	36.5	4.11	3.83	4.06			
3		25.0	32.0	35.0	2.78	3.56	3.89			
1	15	27.0	24.0	29.0	3.00	2.67	3.22	2.89	3.11	3.78
2		25.0	32.0	34.0	2.78	3.56	3.78			
3		26.0	28.0	39.0	2.89	3.11	4.33			

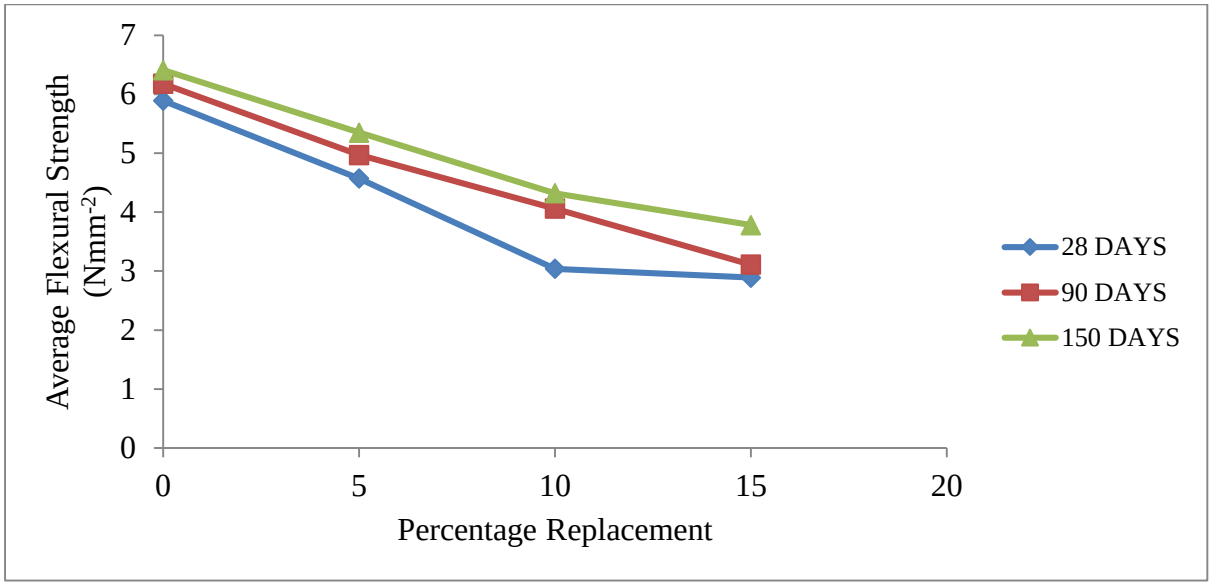

Figure 4.3: Effect of RHA-OA Calcination on Flexural Strength

Table 4.6: Flexural Strength of RHA-F Concrete Beam

Beam Mark	Percentage Replacement	Test Load (KN)			Flexural Strength (Nmm^{-2})			Average Flexural Strength (Nmm^{-2})		
		28 Days	90 Days	150 Days	28 Days	90 Days	150 Days	28 Days	90 Days	150 Days

1	5	27.0	51.0	60.0	3.00	5.67	6.67	3.56	5.48	6.74
2		34.0	49.0	60.0	3.78	5.44	6.67			
3		35.0	48.0	62.0	3.89	5.33	6.89			
1	10	37.0	44.0	53.0	4.11	4.88	5.89	4.07	5.03	6.15
2		39.0	45.0	57.0	4.33	5.00	6.33			
3		34.0	47.0	56.0	3.78	5.22	6.22			
1	15	26.0	40.0	48.0	2.89	4.44	5.33	3.04	4.41	5.32
2		24.0	38.0	49.0	2.67	4.22	5.44			
3		32.0	41.0	46.5	3.56	4.56	5.17			

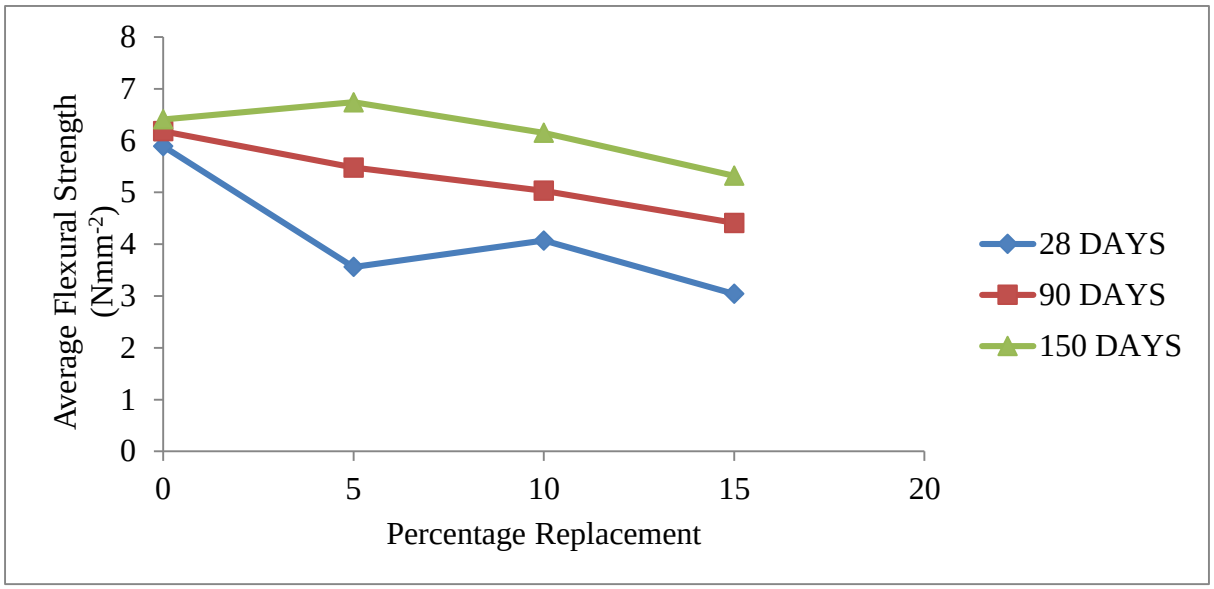

Figure 4.4: Effect of RHA-F Calcination on Flexural Strength

Table 4.7: Flexural Strength of RHA-S Concrete Beam

Beam Mark	Percentage Replacement	Test Load (KN)			Flexural Strength (Nmm^{-2})			Average Flexural Strength (Nmm^{-2})		
		28 Days	90 Days	150 Days	28 Days	90 Days	150 Days	28 Days	90 Days	150 Days

1	5	24.0	41.0	44.0	2.67	4.55	4.89	2.67	4.12	4.51
2		22.0	38.0	40.0	2.44	4.22	4.44			
3		26.0	32.0	38.0	2.89	3.55	4.22			
1	10	22.0	36.0	46.0	2.44	4.00	5.11	2.89	4.42	5.20
2		26.0	44.5	46.0	2.89	4.94	5.11			
3		30.0	39.0	48.0	3.33	4.33	5.39			
1	15	27.5	39.0	42.5	3.06	4.33	4.72	2.57	3.98	4.35
2		20.0	38.0	40.0	2.22	4.22	4.44			
3		22.0	30.0	35.0	2.44	3.33	3.89			

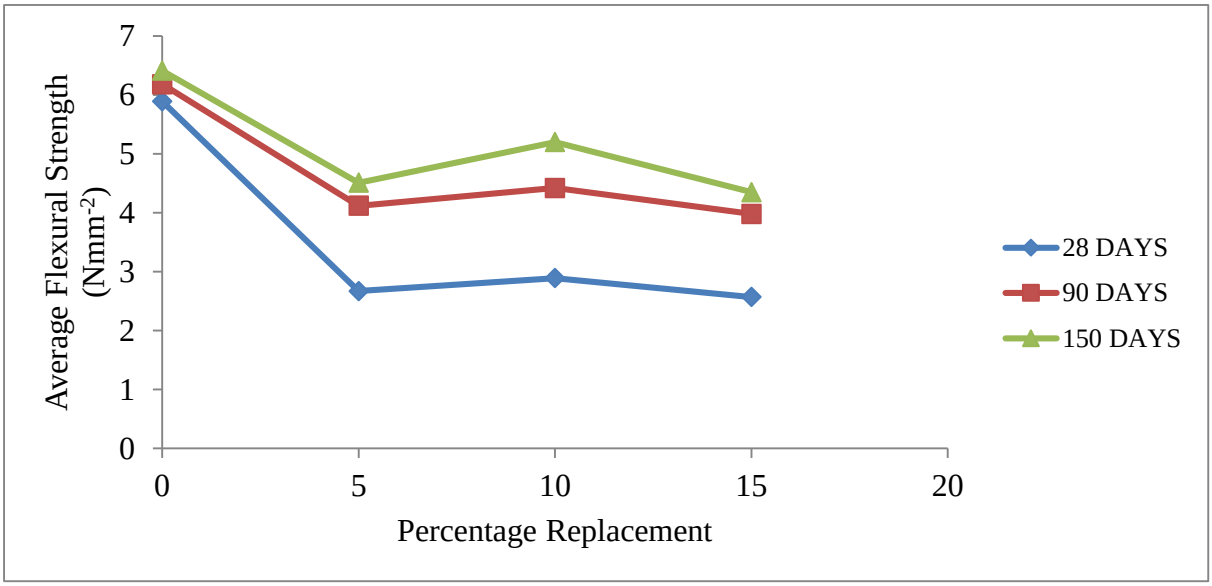

Figure 4.5: Effect of RHA-S Calcination on Flexural Strength

The variation of the flexural strengths of OPC-SDA cement composites from open-air, furnace and local stove calcination methods are shown in Tables 4.8, 4.9 and 4.10 and Figures 4.6, 4.7 and 4.8 below.

Table 4.8: Flexural Strength of SDA-OA Concrete Beam

Beam Mark	Percentage Replacement	Test Load (KN)	Flexural Strength (Nmm^{-2})	Average Flexural Strength (Nmm^{-2})

		28 Days	90 Days	150 Days	28 Days	90 Days	150 Days	28 Days	90 Days	150 Days
1	5	35.0	34.0	40.0	3.88	3.78	4.44	3.77	3.82	4.48
2		29.0	36.0	36.5	3.22	4.00	4.06			
3		38.0	33.0	44.5	4.22	3.67	4.94			
1	10	30.0	37.0	45.0	3.33	4.11	5.00	3.82	4.37	5.51
2		32.0	39.0	50.0	3.56	4.33	5.56			
3		41.0	42.0	44.0	4.56	4.67	4.89			
1	15	22.0	26.0	30.0	2.44	2.89	3.33	2.78	3.32	3.68
2		24.0	34.0	33.5	2.68	3.78	3.72			
3		29.0	29.5	36.0	3.22	3.28	4.00			

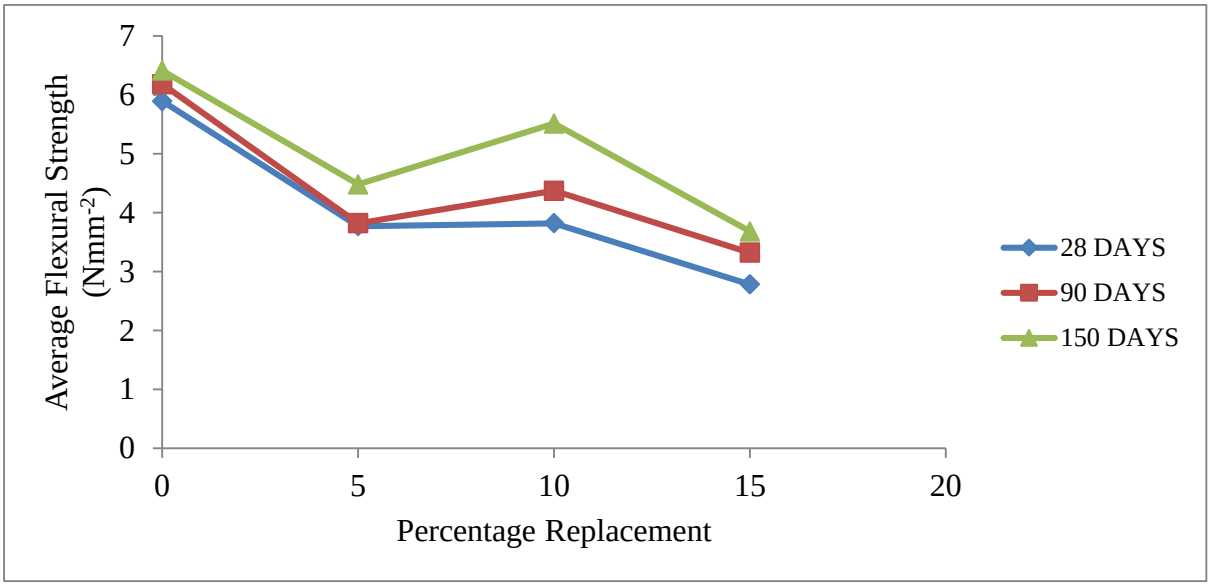

Figure 4.6: Effect of SDA-OA Calcination on Flexural Strength

Table 4.9: Flexural Strength of SDA-F Concrete Beam

Beam Mark	Percentage Replacement	Test Load (KN)	Flexural Strength (Nmm^{-2})	Average Flexural Strength (Nmm^{-2})

		28 Days	90 Days	150 Days	28 Days	90 Days	150 Days	28 Days	90 Days	150 Days
1	5	34.0	45.0	51.0	3.78	5.00	5.67	3.70	5.07	5.85
2		36.0	47.0	52.0	4.00	5.22	5.78			
3		30.0	45.0	55.0	3.33	5.00	6.11			
1	10	28.0	40.0	40.0	3.11	4.44	4.44	2.89	4.11	4.70
2		26.0	37.0	45.0	2.89	4.11	5.00			
3		24.0	34.0	42.0	2.67	3.78	4.67			
1	15	18.0	25.0	30.0	2.00	2.78	3.33	2.22	3.07	3.70
2		20.0	30.0	37.0	2.22	3.33	4.11			
3		22.0	28.0	33.0	2.44	3.11	3.67			

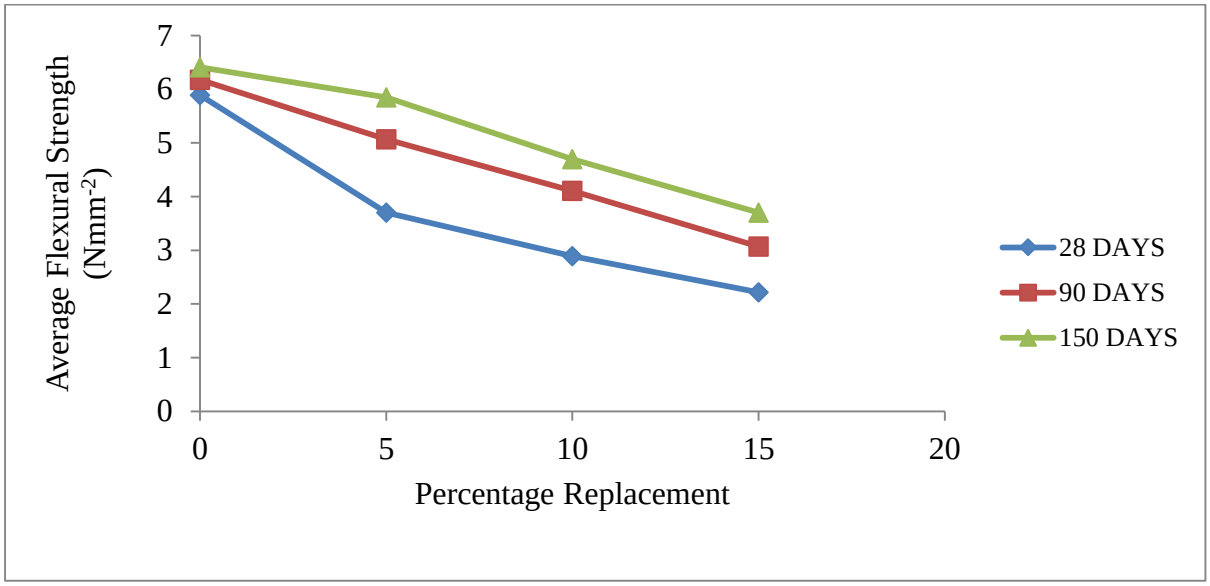

Figure 4.7: Effect of SDA-F Calcination on Flexural Strength

Table 4.10: Flexural Strength of SDA-S Concrete Beam

Beam Mark	Percentage Replacement	Test Load (KN)	Flexural Strength (Nmm^{-2})	Average Flexural Strength (Nmm^{-2})

		28 Days	90 Days	150 Days	28 Days	90 Days	150 Days	28 Days	90 Days	150 Days
1	5	31.0	35.0	38.0	3.44	3.89	4.22	3.55	3.81	4.28
2		34.0	30.0	35.5	3.78	3.33	3.94			
3		31.0	38.0	42.0	3.44	4.22	4.67			
1	10	30.0	41.0	44.5	3.33	4.56	4.94	3.41	4.70	5.06
2		28.5	39.0	42.0	3.17	4.33	4.67			
3		33.5	47.0	50.0	3.72	5.22	5.56			
1	15	29.0	40.0	41.5	3.22	4.44	4.61	2.81	3.66	4.06
2		25.5	30.0	33.0	2.83	3.33	3.67			
3		21.5	31.0	35.0	2.39	3.44	3.89			

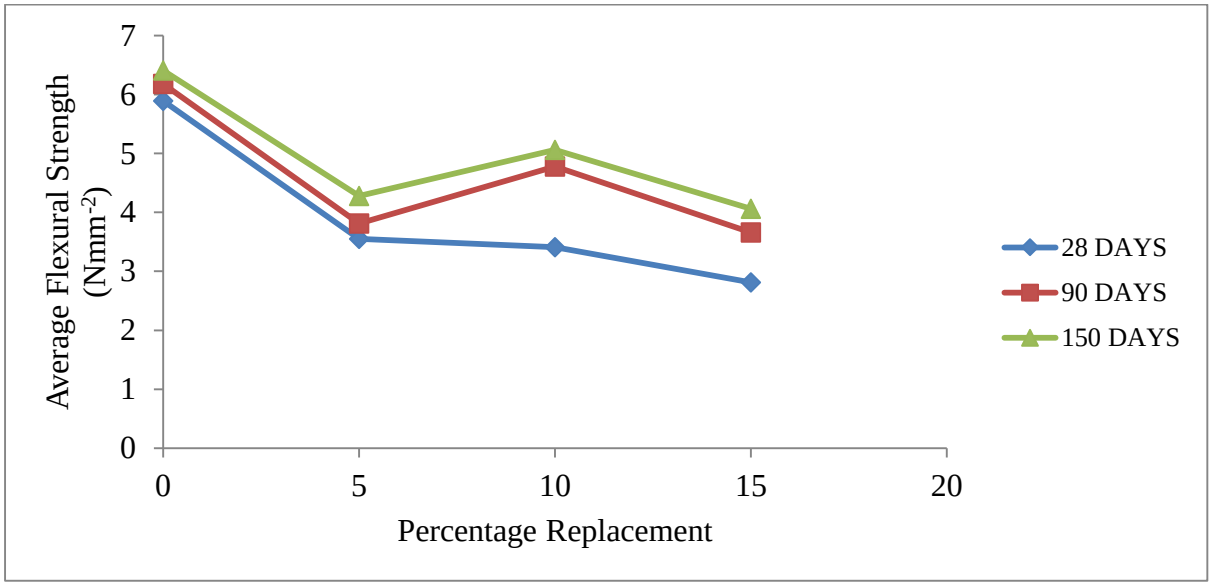

Figure 4.8: Effect of SDA-S Calcination on Flexural Strength

4.1.4 Empirical Model for Predicting Flexural Strengths of RHA and SDA Concretes

Tables 4.11, 4.12, and 4.13 show the regression coefficients for OPC-RHA, which represent the independent contributions of each independent variable, X_1 and X_2 (curing age and replacement percentage respectively) in predicting the dependent variable Y (flexural strength).

Table 4.11: ANOVA for RHA-OA

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%

Intercept	5.390448	0.189431	28.45594	3.98E-10	4.961925	5.818972	4.961925	5.818972
Curing Age (Days)	0.007114	0.00155	4.5885	0.001312	0.003607	0.010622	0.003607	0.010622
Replacement Level (%)	-0.19713	0.013815	-14.2697	1.74E-07	-0.22838	-0.16588	-0.22838	-0.16588

Model for Predicting the flexural strength of RHA-OA (N/mm^2),

$$Y = 5.390448 + 0.007114\ X_1 - 0.19713\ X_2 \quad (4.1)$$

Table 4.12: ANOVA for RHA-F

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	4.671559	0.293571	15.91288	6.75E-08	4.007455	5.335662	4.007455	5.335662
Curing Age (Days)	0.0163	0.002403	6.783475	8.05E-05	0.010864	0.021735	0.010864	0.021735
Replacement Level (%)	-0.1238	0.021409	-5.78248	0.000265	-0.17223	-0.07537	-0.17223	-0.07537

Model for Predicting the flexural strength of RHA-F (N/mm^2)

$$Y = 4.671559 + 0.0163\ X_1 - 0.1238\ X_2 \quad (4.2)$$

Table 4.13: ANOVA for RHA-S

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	4.325457	0.577984	7.483694	3.76E-05	3.017966	5.632949	3.017966	5.632949
Curing Age (Days)	0.013249	0.004731	2.800526	0.020695	0.002547	0.02395	0.002547	0.02395
Replacement Level (%)	-0.14353	0.042151	-3.40521	0.007809	-0.23889	-0.04818	-0.23889	-0.04818

Model for Predicting the flexural strength of RHA-S (N/mm^2)

$$Y = 4.325457 + 0.013249\ X_1 - 0.14353\ X_2 \quad (4.3)$$

The predicted flexural strengths of OPC-RHA concretes obtained from models in 4.1, 4.2, and 4.3, are compared with the actual flexural strengths in Table 4.14.

Table 4.14: Actual and Predicted Flexural Strengths of RHA Concrete

Observation	Actual Strength (Nmm^{-2})			Predicted Strength (Nmm^{-2})			Residuals (Nmm^{-2})		
	RHA-OA	RHA-F	RHA-S	RHA-OA	RHA-F	RHA-S	RHA-OA	RHA-F	RHA-S
1	5.89	5.89	5.89	5.58965	5.12795	4.696418	0.300349	0.762049	1.193582
2	4.57	4.07	2.67	4.60398	4.50895	3.978752	-0.03398	-0.43895	-1.30875
3	3.04	3.67	2.89	3.61831	3.88995	3.261085	-0.57832	-0.21995	-0.37109
4	2.89	3.04	2.57	2.63265	3.27095	2.543418	0.257349	-0.23095	0.026582
5	6.18	6.18	6.18	6.03074	6.13853	5.517832	0.149257	0.041467	0.662168
6	4.97	5.48	4.12	5.04507	5.51953	4.800166	-0.07508	-0.03953	-0.68017
7	4.06	5.03	4.42	4.05941	4.90053	4.082499	0.00059	0.129467	0.337501
8	3.11	4.41	3.98	3.07374	4.28153	3.364832	0.036257	0.128467	0.615168
9	6.41	6.41	6.41	6.45760	7.11651	6.312749	-0.04761	-0.70652	0.097251
10	5.35	6.74	4.51	5.47193	6.49751	5.595082	-0.12194	0.242484	-1.08508
11	4.32	6.15	5.20	4.48627	5.87851	4.877416	-0.16627	0.271484	0.322584
12	3.78	5.32	4.35	3.50060	5.25951	4.159749	0.290381	0.060484	0.190251

Tables 4.15, 4.16, and 4.17 show the regression coefficients for OPC-SDA, which represent the independent contributions of each independent variable, X_1 and X_2 (curing age and replacement percentage respectively) in predicting the dependent variable Y (flexural strength).

Table 4.15: ANOVA for SDA-OA

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	5.081269	0.44442	11.43348	1.16E-06	4.07592	6.086618	4.07592	6.086618
Curing Age (Days)	0.007083	0.003638	1.947139	0.083349	-0.00115	0.015312	-0.00115	0.015312
Replacement Level (%)	-0.16553	0.032411	-5.10738	0.000639	-0.23885	-0.09222	-0.23885	-0.09222

Model for Predicting the flexural strength of SDA-OA (N/mm^2)

$$Y = 5.081269 + 0.007083\ X_1 - 0.16553\ X_2 \quad (4.4)$$

Table 4.16: ANOVA for SDA-F

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	4.959559	0.233612	21.22994	5.36E09	4.431093	5.488025	4.431093	5.488025
Curing Age (Days)	0.012229	0.001912	6.395513	0.000126	0.007903	0.016554	0.007903	0.016554
Replacement Level (%)	-0.20927	0.017037	-12.2832	6.31E-07	-0.24781	-0.17073	-0.24781	-0.17073

Predicted Flexural strength of SDA-F (N/mm^2)

$$Y = 4.959559 + 0.012229\ X_1 - 0.20927\ X_2 \quad (4.5)$$

Table 4.17: ANOVA for SDA-S

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	4.840138	0.509591	9.498089	5.48E-06	3.687364	5.992913	3.687364	5.992913
Curing Age (Days)	0.008517	0.004171	2.042001	0.071533	-0.00092	0.017952	-0.00092	0.017952
Replacement %	-0.1488	0.037163	-4.00394	0.003092	-0.23287	-0.06473	-0.23287	-0.06473

Predicted Flexural strength of SDA-S (N/mm^2)

$$Y = 4.840138 + 0.008517\ X_1 - 0.1488\ X_2 \quad (4.6)$$

The predicted flexural strengths of OPC-SDA concretes obtained from models in 4.4, 4.5, and 4.6, are compared with the actual flexural strengths in Table 4.18.

Table 4.18: Actual and Predicted Flexural Strengths of SDA Concrete

Observation	Actual Strength (Nmm^{-2})			Predicted Strength (Nmm^{-2})			Residuals (Nmm^{-2})		
	SDA-OA	SDA-F	SDA-S	SDA-OA	SDA-F	SDA-S	SDA-OA	SDA-F	SDA-S

1	5.89	5.89	5.89	5.27958	5.301966	5.078617	0.61041	0.588034	0.811383
2	3.77	3.70	3.55	4.45192	4.255633	4.334617	-0.68192	-0.55563	-0.78462
3	3.82	2.89	3.41	3.62425	3.209299	3.590617	0.19574	-0.3193	-0.18062
4	2.78	2.22	2.81	2.79658	2.162966	2.846617	-0.01659	0.057034	-0.03662
5	6.18	6.18	6.18	5.71872	6.060153	5.606678	0.46127	0.119847	0.573322
6	3.82	5.07	3.81	4.89105	5.013819	4.862678	-1.07106	0.056181	-1.05268
7	4.37	4.11	4.70	4.06338	3.967486	4.118678	0.30661	0.142514	0.581322
8	3.32	3.07	3.66	3.23572	2.921153	3.374678	0.08427	0.148847	0.285322
9	6.41	6.41	6.41	6.14369	6.793881	6.117705	0.26630	-0.38388	0.292295
10	4.48	5.85	4.28	5.31602	5.747548	5.373705	-0.83602	0.102452	-1.0937
11	5.15	4.70	5.06	4.48835	4.701215	4.629705	0.66164	-0.00121	0.430295
12	3.68	3.70	4.06	3.66069	3.654881	3.885705	0.01939	0.045119	0.174295

4.2 Discussions of Results

Various parameters which significantly affect the properties of concrete (plastic and hardened state) with the inclusion of RHA and SDA are discussed below.

4.2.1 Chemical Composition of RHA and SDA

The raw materials used in the manufacture of cement consist mainly of lime, silica, alumina, and iron oxides. The results of the percentage by weight of the various oxides of RHA and SDA are shown in Table 4.1

The results show that the proportion of silicon oxide (SiO_2), aluminium oxide (Al_2O_3) and iron oxide (Fe_2O_3) in RHA-OA, RHA-F, RHA-S and SDA-F, when combined together (i.e. $SiO_2 + Al_2O_3 + Fe_2O_3$) meet the ASTM C 618 (1978) specification of ($SiO_2 + Al_2O_3 + Fe_2O_3$) greater than 70% for cement and pozzolanic materials. However, $SiO_2 + Al_2O_3 + Fe_2O_3$ for each of SDA-OA and SDA-S (66.03%, and 69.17% respectively) was slightly lower than 70% as required by ASTM C 618 (1978). The following previous studies shown in Tables 2.3-2.5 indicate the percentage composition of SiO_2 for RHA: Wansom, et al. (2009): 91-93%, Mehta (1992): 87.2%, Bui, et al. (2005): 86.98%, Zhang, et al. (1996): 87.3%, and Abalaka (2012): 95.41%. For SDA, $SiO_2 + Al_2O_3 + Fe_2O_3$ were as follows: Marthong (2012): 65.45%, Raheem, et al. (2012): 73.07%, and Raheem and Sulaiman (2013): 74.89%. However, the pozzolanicity

test confirmed the RHA and SDA as pozzolanic since both fixed some quantities of lime over time.

These higher combined values of SiO_2, Al_2O_3, and Fe_2O_3 in the ashes obtained from furnace calcination could be as a result of the higher calcination temperature and heat rate produced by the furnace calcination. Therefore, RHA can be said to be a better pozzolan than SDA.

From these test results, it can be concluded that the relative proportions of these oxides were responsible for influencing the various properties of the concrete and that RHA and SDA are moderately rich in silica, alumina, and ferrous oxides.

4.2.2 Effect of Replacement of OPC with RHA and SDA on the Flexural Strength of Concrete

Tables 4.2 to 4.4 and Figures 4.1 and 4.2 show the variations in the flexural strengths of the control (OPC), OPC-RHA and OPC-SDA blended concrete with varying percentage OPC replacements with RHA and SDA.

The results show that the flexural strengths decreased with increased amount of RHA and SDA, and increased with the curing age. The highest flexural strengths were developed at 5% replacement and at 150 days curing age with RHA having 6.74 Nmm^{-2}, and SDA having 5.85 Nmm^{-2} compared to 6.41 Nmm^{-2} for the Control. Also, the results show that the blended cement concrete gained strength slowly at early curing age.

4.2.3 Effect of RHA and SDA Calcination methods on the Flexural Strength of Concrete

Tables 4.5 to 4.10 and Figures 4.3 to 4.8 show the effect of RHA and SDA calcination methods on the flexural strengths of the OPC-RHA and OPC-SDA blended concrete.

The 28th day flexural strength shows a decrease in flexural strength from 5.89 Nmm^{-2} for the control (OPC) concrete to 4.57 Nmm^{-2} for RHA–OA at 5%, 4.07 Nmm^{-2} for RHA–F at 5%, 2.89 Nmm^{-2} for RHA–S at 10%. SDA–OA, SDA–F, SDA–S developed strengths of: 3.82 Nmm^{-2} at 10%, 3.70 Nmm^{-2} at 5%, and 3.55 Nmm^{-2} at 5% respectively. These results show that concrete containing RHA and SDA gain strength slowly at early curing age. This agrees with previous findings that blended cement concrete gain strength slowly at early curing age, which could be attributed to low hydration and pozzolanic reaction.

At 90 days curing, there was continuous increase in flexural strength for both the control (OPC) and RHA and SDA concrete with the flexural strength values ranging from 6.18 Nmm^{-2} for the control, to 4.97 Nmm^{-2} for RHA–OA at 5%, 5.48 Nmm^{-2} for RHA–F at 5%, and 4.42 Nmm^{-2} for RHA–S at 10% replacement. Also, SDA–OA, SDA–F, SDA–S developed strengths of: 4.37 Nmm^{-2} at 10%, 5.07 Nmm^{-2} at 5%, and 4.70Nmm^{-2} at 10% respectively.

Also, at 150 days curing, there was continuous increase in flexural strength for both the control (OPC) and RHA and SDA concrete. OPC developed a maximum flexural strength of 6.48 Nmm^{-2}; the OPC-RHA concrete has the maximum flexural strength of 6.74 Nmm^{-2} with RHA-F at 5% replacement, while OPC-SDA has a maximum flexural strength of 5.85 Nmm^{-2} with SDA-F at 5%.

The increase in flexural strength of the RHA and SDA concrete is due to the pozzolanic reaction of the ashes. The particle size distribution shows that the ash obtained from furnace calcination has finer particles. Therefore, the higher strength gain of the blended concrete contained ashes obtained by the furnace calcination method could be due to the heating conditions such as higher temperature, heating rate and longer time of burning in the furnace.

The strength gain observed in the blended cement concrete may be attributed to the decrease in capillarity porosity of hardened cement paste brought about by pozzolanic reaction,

improved densification of the internal or micro–structure of the concrete which is as a result of the occurrence of hydration process followed by pozzolanic reaction in the RHA and SDA concrete mix.

Therefore, it can be concluded that the flexural strengths of OPC-RHA and OPC-SDA concrete are comparable to that of the control concrete, with the ashes obtained from furnace calcination giving the highest strengths. Therefore, RHA and SDA could be satisfactorily used as partial cement replacement in normal or general concrete works, especially in low–load bearing structural members.

4.2.4 Empirical Model for Predicting Flexural Strengths of RHA and SDA Concrete

Multiple regressions give the opportunity to establish the evidence that one or more independent variables (replacement percentage and curing age) cause another dependent variable (flexural strength) to change. Hence, the analysis establishes the relative magnitude of the contribution of each independent variable. The regression analyses show that the actual or laboratory flexural strengths of the OPC-RHA and OPC-SDA concretes are comparable to the predicted flexural strengths obtained from the models.

The R-square (R^2) which is the coefficient of determination shows that there is strong correlation between the criterion variable, Y (flexural strength) and the predictor variables X_1 and X_2 (replacement percentage, and curing age respectively), indicating that the predictions from the models are good. Also, the R^2 which are close to 1.0 indicates that almost all the variability with variables specified in the models has been accounted for. The values of R^2 (61.3 to 95.3 %) suggest that the models are relatively good for predicting the flexural strengths of concrete. As $P < 0.05$, it implies that the regression models are statistically significant. The low P-values indicate that the null hypothesis can be rejected i.e changes in the independent variables, X_1 and X_2 (curing age, and percentage replacement) are associated with changes in

the dependent variable, Y (flexural strength). The t-values and the P-values indicate the significant contribution of curing age and replacement percentage in predicting the flexural strength of RHA and SDA from the different calcination methods. The t-values measure how strongly each of curing age, replacement percentage influence the prediction of the flexural strength. The results show that the contribution of curing age and replacement percentage in determining the flexural strength of RHA and SDA is statistically significant since $P < 0.05$ and adjusted R^2 lie between 61.3 to 95.3 %. This indicates that between 61.3% to 95.3 % of the variations in flexural strengths of the RHA and SDA from the three calcination methods, can be explained by curing age and replacement percentage.

From the student's statistical two-tail (for unequal variance) t-Test for the OPC-RHA and OPC-SDA composites from the different calcination methods, the conditions that t Stat < -t critical two-tail or t Stat > t critical two-tail was not satisfied. Therefore, the null hypothesis, H_0: $\mu_1 - \mu_2 = 0$ was satisfied and adequate, hence, the alternative hypothesis, H_0: $\mu_1 - \mu_2 \neq 0$ was rejected. There was no significant difference between the experimental (laboratory) and the theoretically (predicted) expected results. The null hypothesis, H_0 was satisfied. There was no enough difference in the values of the observed means of the variables (actual or laboratory and predicted flexural strengths) to convincingly say that the average flexural strengths obtained from the laboratory and the predicted flexural strengths differ significantly. Since the predicted strengths by the model were in total agreement with the corresponding experimentally–observed values, the null hypothesis was satisfied. This means that the model equations were valid and adequate.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

Based on the results of the various laboratory tests performed and the analysis made, the following conclusions can be drawn:

i. RHA and SDA are moderately rich in SiO_2, Fe_2O_3, Al_2O_3 and CaO which reacted with $Ca(OH)_2$ forming calcium silicate hydrate (C–S–H). RHA from the three calcination methods and SDA from furnace calcination only satisfied the requirement for such a material by having $SiO_2 + Fe_2O_3 + Al_2O_3$ greater 70%; while SDA from open-air and stove calcinations failed to meet the requirement. Therefore, based on oxide composition; RHA can be said to be a better pozzolan, than SDA.
ii. The 150-day flexural strength values of OPC-RHA and OPC-SDA concretes are comparable to those of purely OPC concretes (with 0% RHA and SDA).
iii. The flexural strengths of the OPC-RHA and OPC-SDA increase with curing age and decrease with increased amount (i.e. increased percentage replacement) of RHA and SDA. Maximum strengths of 6.74 N/mm^2 for RHA and 5.85 N/mm^2 for SDA are obtained at 5% replacement of OPC with RHA, and SDA respectively, at 150 days curing age.
iv. The flexural strengths of OPC-RHA and OPC-SDA concretes vary with the calcination methods of producing RHA and SDA. Maximum flexural strength of 6.74 N/mm^2 for RHA is obtained at 5% replacement using furnace calcination; while maximum flexural strength of 5.85 N/mm^2 for SDA at 5% replacement is obtained using the furnace calcination.

v. Although, RHA and SDA obtained from furnace calcination produced maximum flexural strengths, open-air and local stove calcination methods are suitable for the production of the pozzolanic ashes as indicated by their strength gain.

vi. Multiple regression models developed are effectively used as tools for predicting and validating the flexural Strengths of concrete based on the curing age and replacement level.

5.2 Recommendations

From the results and conclusions drawn from this investigation, I recommend the following:

i. 10% replacement of OPC with RHA and SDA in OPC-RHA, and OPC-SDA concrete is suitable for general concrete works, especially low-load structural members.

ii. Open-air, furnace and local stove calcination methods can be used for burning, since they are easy and can produce strengths comparable with that of OPC.

iii. Stakeholders in the construction and cement industry could develop technology for commercial production of pozzolanic cement, as this will help reduce the cost of civil engineering projects.

iv. Other properties of blended cement concrete such as water adsorption and durability, etc., as well as effect of different calcination temperatures on pozzolanic ashes could be investigated.

5.3 Contributions to Knowledge

i. This research shows that RHA and SDA have significant chemical property that encourages their use as pozzolans.

ii. It has further confirmed that RHA and SDA are effective pozzolans which can contribute to the mechanical properties of concrete, such as flexural strength and therefore, partly replace cement in concrete making.

iii. The research has shown that it is possible to use open-air, furnace (charcoal fired) and local stove calcination to produce RHA and SDA of acceptable quality that could be used in concrete making.

iv. It also reveals that the multiple regression models can be used as a powerful tool to derive explicit formulae, which can be directly used to predict the strength of concrete.

v. Finally, this research work will contribute to already existing literatures in the study of RHA and SDA as alternative cementitious materials and sandstone as coarse aggregate in concrete production.

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APPENDIX

APPENDIX A: BATCHING OF CONSTITUENT MATERIALS OF CONCRETE

Volume of concrete = 0.15 x 0.15 x 0.6 = 1.35 x 10^{-2} m^3

Density of concrete = 2400 Nm^{-3} = $\frac{2400}{9.8}$

= 2448.98 kgm^{-3}.

Mass of 1 concrete beam = Density of concrete x volume of 1 concrete beam

= 2448.98 kg/m^3 x 1.35 x 10^{-2} m^3

= 33.06 kg

Mix ratio: 1:2:4

Mass of cement in 1 concrete beam = 1/7 * 33.06 = 4.72 kg

Mass of fine aggregate (sharp sand) in 1 concrete beam = 2/7 *33.06 = 9.45 kg

Mass of coarse aggregate (sandstone) in 1 concrete beam = 4/7 * 33.06 = 18.89 kg

Water to cement ratio = 0.6

w/c = 0.6

w = 0.6 c = 0.6 * 4.72 = 2.83 kg

Mass of water in 1 concrete beam = 2.83 kg

Control Specimen Containing 100% OPC

Using 9 concrete beams (i.e. 3 beams each for 3 curing ages)

Mass of cement in 9 concrete beams = 9 * 4.72 = 42.48 kg

Mass of fine aggregate in 9 concrete beams = 9 * 9.45 = 85.05 kg

Mass of local stone aggregate in 9 concrete beams = 9 * 18.89 = 170.01 kg

Quantities of Pozzolans: OPC-RHA Concrete and OPC-SDA Concrete

5% Replacement Level

Mass of cement in 1 concrete beam = 0.95 * 4.72 = 4.48 kg

Mass of RHA (or SDA) in 1 concrete beam = 4.72 - 4.48 = 0.24 kg

Mass of cement in 9 concrete beams = 9 * 4.48 = 40.32 kg

Mass of RHA (or SDA) in 9 beams = 9 * 0.24 = 2.16 kg

10% Replacement Level

Mass of cement in 1 concrete beam = 0.9 * 4.72 = 4.25 kg

Mass of RHA (or SDA) in 1 concrete beam = 4.72 – 4.25 = 0.47 kg

Mass of cement in 9 concrete beams = 9 * 4.25 = 38.25 kg

Mass of RHA (or SDA) in 9 beams = 9 * 0.47 = 4.23 kg

15% Replacement Level

Mass of cement in 1 concrete beam = 0.85 * 4.72 = 4.01 kg

Mass of RHA (or SDA) in 1 concrete beam = 4.72 – 4.01 = 0.71 kg

Mass of cement in 9 concrete beams = 9 * 4.01 = 36.09 kg

Mass of RHA (or SDA) in 9 concrete beams = 9 * 0.71 = 6.39 kg

APPENDIX B: RESULTS OF SIEVE ANALYSIS OF SAMPLES

Table B1: Sieve Analysis Result of Fine Aggregate

Sieve Sizes (mm)	Mass Retained (g)	Cumulative Mass Retained (g)	Percentage of Mass Retained	Cumulative Percentage Mass Retained	Percentage Finer
2.36	25.1	25.1	5.02	5.02	94.98
1.18	84	109.1	16.8	21.82	78.18
0.6	80.82	189.92	16.164	37.984	62.016
0.3	162.8	352.72	32.56	70.544	29.456
0.212	120.22	472.94	24.044	94.588	5.412
0.15	18.1	491.04	3.62	98.208	1.792
0.075	6.5	497.54	1.3	99.508	0.492
PAN	1.5	499.04	0.3	99.808	0.192

Effective size, D_{10}	4.8mm
Coefficient of Uniformity, Cu	0.63
Coefficient of Gradation, Cg	1.11
Fineness Modulus	4.28

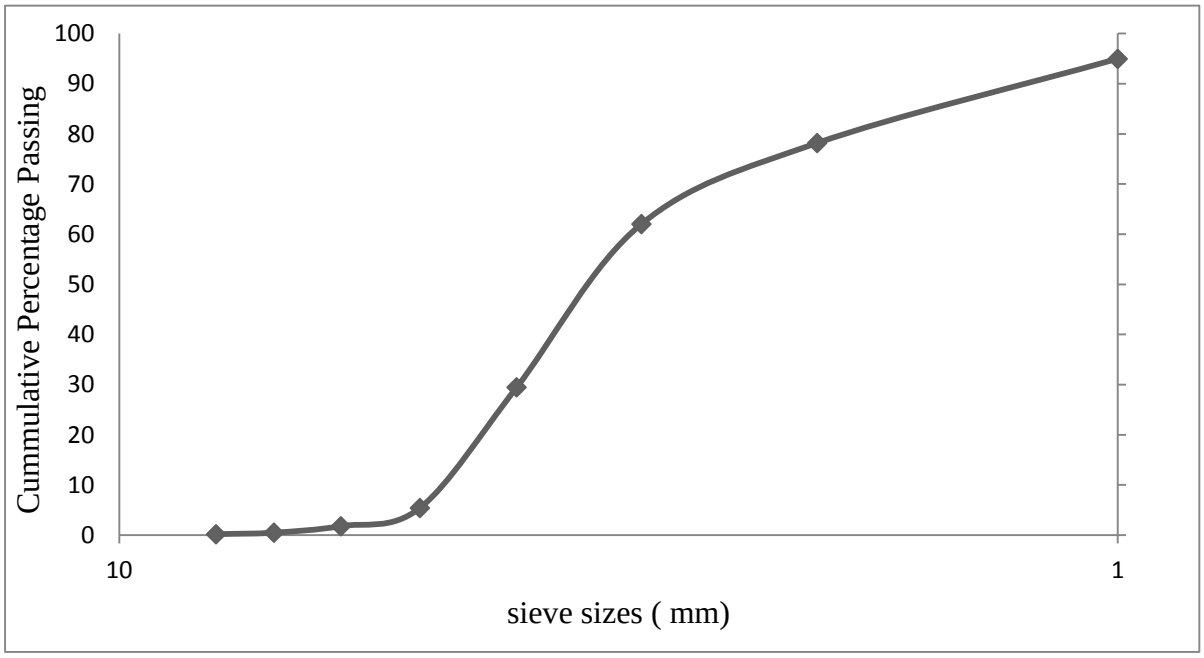

Figure B1: Particle size distribution of fine aggregate

Table B2: Sieve Analysis Result of Sandstone Aggregate

Sieve Size (mm)	Mass Retained (g)	Cumulative Mass Retained (g)	Percentage Mass Retained	Cumulative Mass Retained (%)	Percentage Finer (%)
37.5	0	0	0	0	100
19	814	814	16.28	16.28	83.72
13.2	2200	3014	44	60.28	39.72
9.5	980	3994	19.6	79.88	20.12
6.7	560	4554	11.2	91.08	8.92
4.75	225	4779	4.5	95.58	4.42
2.36	75	4854	1.5	97.08	2.92
Pan	75	4929	1.5	98.58	1.42

Effective size, D_{10}	7.0mm
Coefficient of Uniformity, Cu	2.57
Coefficient of Gradation, Cg	1.30
Fineness Modulus	4.40

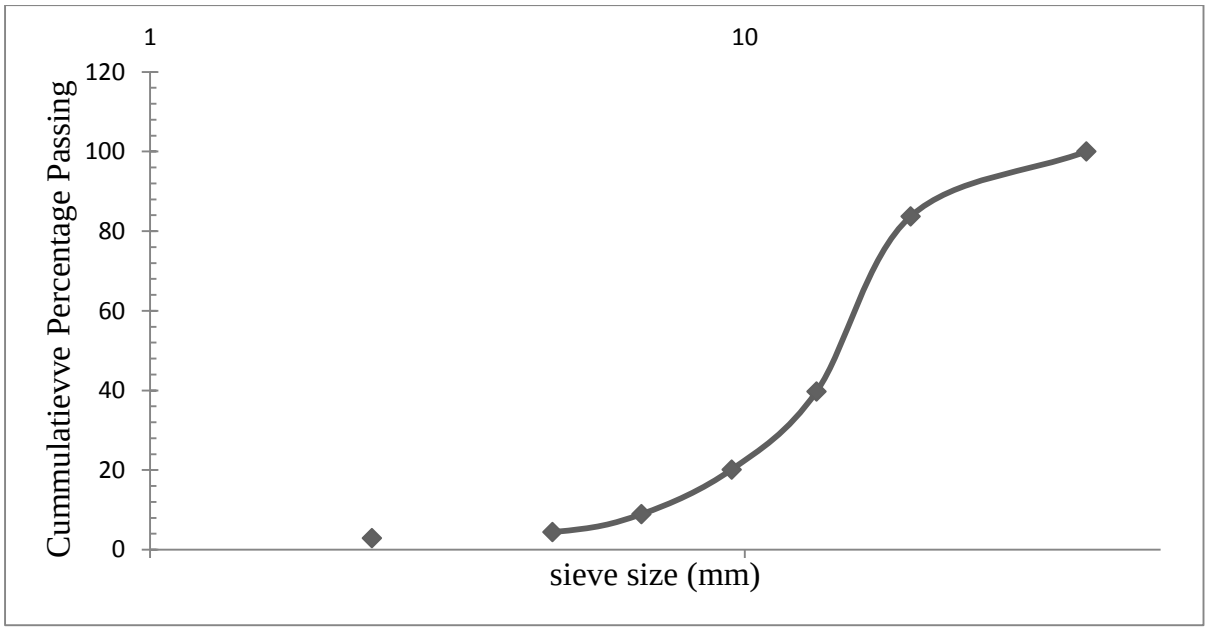

Figure B2: Particle size distribution of Sandstone

Table B3: Sieve Analysis Result of RHA (Using Hydrometer Method)

Particle Size Distribution (PSD)	Method	RHA–OA	RHA–F	RHA–S
< 2 µm (%)	ASTM D 422	3.0	13.0	7.0
>2 µm ≤ 63 µm (%)	ASTM D 422	26.0	27.0	30.0
>63 µm (%)	ASTM D 422	71.0	60.0	63.0
Fineness Modulus		1.32	1.53	1.44

Table B4: Sieve Analysis Result of SDA (Using Hydrometer Method)

Particle Size Distribution (PSD)	Method	SDA–OA	SDA–F	SDA–S
< 2 µm (%)	ASTM D 422	7.0	12.0	9.0
>2 µm ≤ 63 µm (%)	ASTM D 422	28.0	29.0	35.0
>63 µm (%)	ASTM D 422	65.0	59.0	56.0
Fineness Modulus		1.42	1.53	1.53

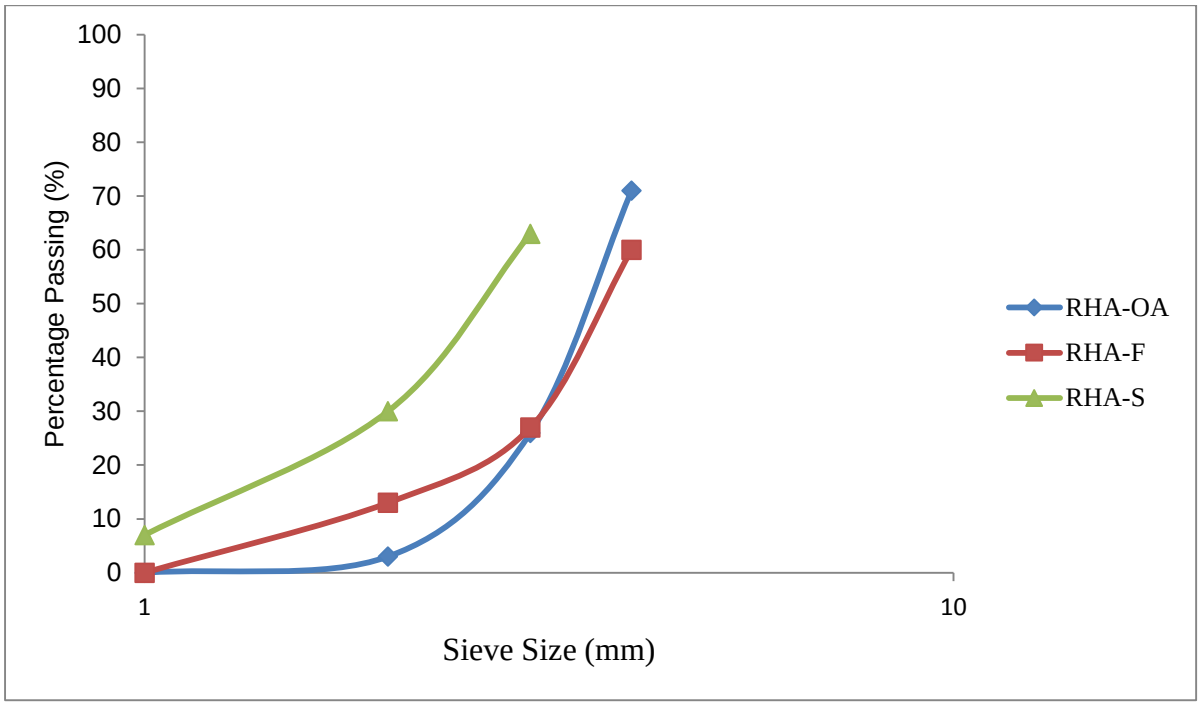

Figure B3: Particle Size Distribution of RHA

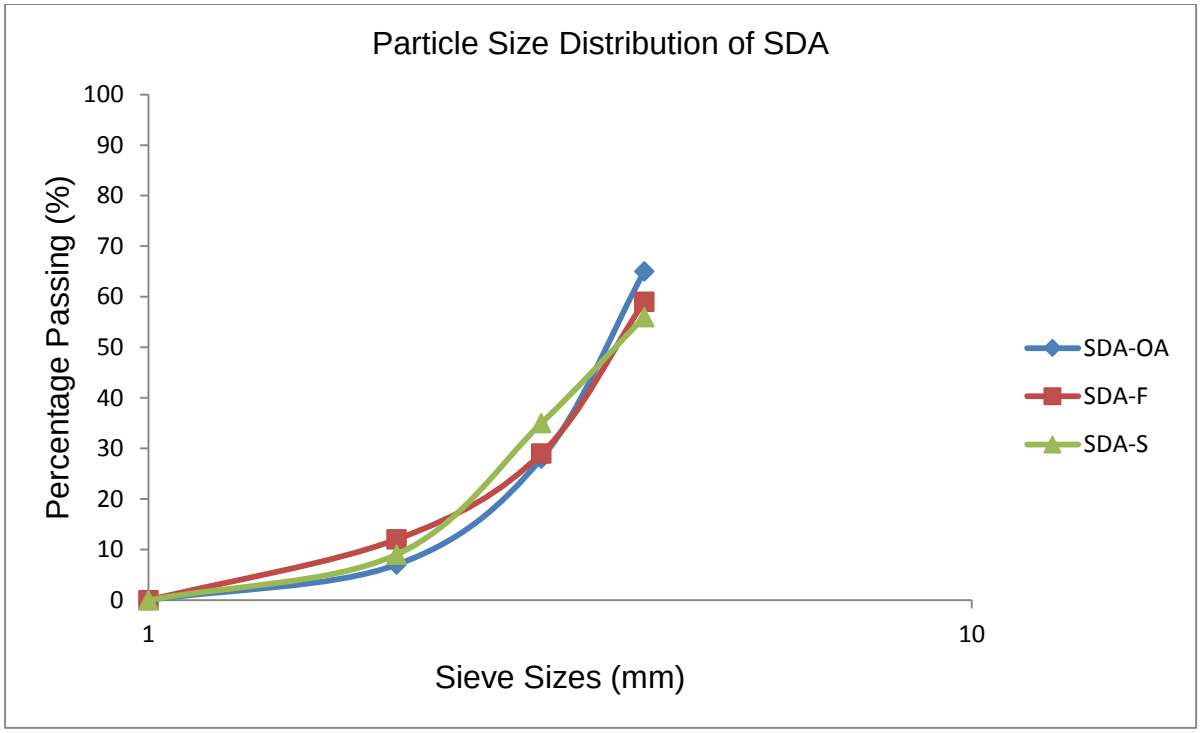

Figure B4: Particle Size Distribution of SDA

APPENDIX C: RESULTS OF SPECIFIC GRAVITY OF SAMPLES

Table C1: Specific Gravity Results of Rice Husk Ash (RHA)

Calcination Method	RHA-OPEN AIR			RHA-FURNACE			RHA –STOVE		
Bottle label	1	2	3	1	2	3	1	2	3
Mass of empty bottle, M_1 (g)	15.97	15.60	15.90	15.97	15.60	15.90	15.97	15.60	15.90
Mass of bottle + Ash, M_2 (g)	38.50	40.00	36.54	36.56	38.40	34.54	37.56	40.32	35.54
Mass of bottle + Ash + water, M_3 (g)	76.50	78.03	76.88	74.50	76.03	74.88	75.66	76.03	74.88
Mass of bottle filled with water, M_4 (g)	66.44	67.50	67.70	66.65	66.53	67.70	67.44	66.63	66.51
Specific Gravity	1.81	1.76	1.80	1.62	1.71	1.63	1.62	1.61	1.74
Average Specific Gravity	1.79			1.65			1.66		

Table C2: Specific Gravity Results of Saw Dust Ash (SDA)

Calcination Method	SDA-OPEN AIR			SDA-FURNACE			SDA-STOVE		
Bottle label	1	2	3	1	2	3	1	2	3
Mass of empty bottle, M_1 (g)	27.23	26.57	28.50	15.97	15.60	15.90	15.97	15.60	15.90
Mass of bottle + Ash, M_2 (g)	37.80	38.54	36.75	28.00	28.54	30.25	36.50	33.50	37.80
Mass of bottle + Ash + water, M_3 (g)	81.71	82.4	82.43	72.00	72.4	73.05	74.50	73.40	76.43
Mass of bottle filled with water, M_4 (g)	77.28	76.65	78.55	68.00	67.65	67.70	67.50	66.54	67.70
Specific Gravity	1.72	1.92	1.89	1.50	1.58	1.59	1.52	1.62	1.66
Average Specific Gravity	1.85			1.56			1.60		

APPENDIX D: RESULTS OF AGGREGATE ABRASION AND IMPACT VALUE TESTS

Table D1: Result of Abrasion test of coarse aggregate (Sandstone)

Mass of sample (kg)	5000
Mass of sample retained on sieve after test (kg)	2324
Aggregate Abrasion value (%)	53.52

Table D2: Aggregate impact value of coarse aggregate (Sandstone)

Mass of empty mould (kg)	Mass of mould + dry sample (kg)	Mass of mould + dry sample after compaction (kg)	Mass of dry sample (kg)
3.414	4.100	4.100	0.686
3.414	4.124	4.122	0.708

After crushing and sieving through 2.36 mm sieve

Test number	Mass of pan + crushed sample (kg)	Mass of empty pan (kg)	Mass of crushed sample (kg)	Impact value (%)	Average impact value (%)
1	0.526	0.331	0.195	28.43	28.14
2	0.522	0.331	0.191	27.84	

APPENDIX E: RESULT OF POZZOLANICITY TEST

Table E1: Pozzolanicity of RHA at Room Temperature

Pozzolan	Titration Values of Acid that reacted at various Time Intervals (cm^3)				
	5 Minutes	20 Minutes	40 Minutes	60 Minutes	90 Minutes
RHA-OPEN AIR	0.13	0.06	0.09	0.80	
RHA-FURNACE	0.06	0.06	0.09	0.10	0.10
RHA-STOVE	0.08	0.08	0.08	0.60	

Table E2: Pozzolanicity of SDA at Room Temperature

Pozzolan	Titration Values of Acid that reacted at various Time Intervals (cm^3)				
	5 Minutes	20 Minutes	40 Minutes	60 Minutes	90 Minutes
SDA-OPEN AIR	0.78	0.88	0.89	0.94	1.00
SDA-FURNACE	0.10	0.07	0.06	0.90	
SDA-STOVE	0.90	0.11	0.10	0.11	0.16

Table E3: Pozzolanicity of RHA at 75^0C

Pozzolan	Titration values of Acid that reacted at various Time Intervals (cm^3)			
	20 Minutes	40 Minutes	60 Minutes	90 Minutes
RHA-OA	0.060	0.040	0.040	0.030
RHA-F	0.050	0.040	0.030	0.030
RHA-S	0.070	0.060	0.050	0.045

Table E4: Pozzolanicity of SDA at 75^0C

Pozzolan	Titration Values of Acid that reacted at various Time Intervals (cm^3)			
	20Minutes	40 Minutes	60 Minutes	90 Minutes
SDA-OA	0.760	0.810	0.830	0.830
SDA-F	0.060	0.050	0.045	0.030
SDA-STOVE	0.110	0.080	0.070	0.050

APPENDIX F: RESULTS OF SLUMP TEST

Table F1: Slump Test Result of RHA and SDA Concrete

Calcination Method	Replacement Level (%)	Height of Subsidence (mm)		Slump (mm)	
		RHA	SDA	RHA	SDA
OPC	0	168	168	132	132
Furnace	5	149	190	151	110
Open-Air		160	180	140	120
Stove		135	155	165	145
Furnace	10	210	210	90	90
Open-Air		220	220	80	80
Stove		245	180	55	120
Furnace	15	260	272	40	28
Open-Air		265	265	35	35
Stove		280	270	20	30

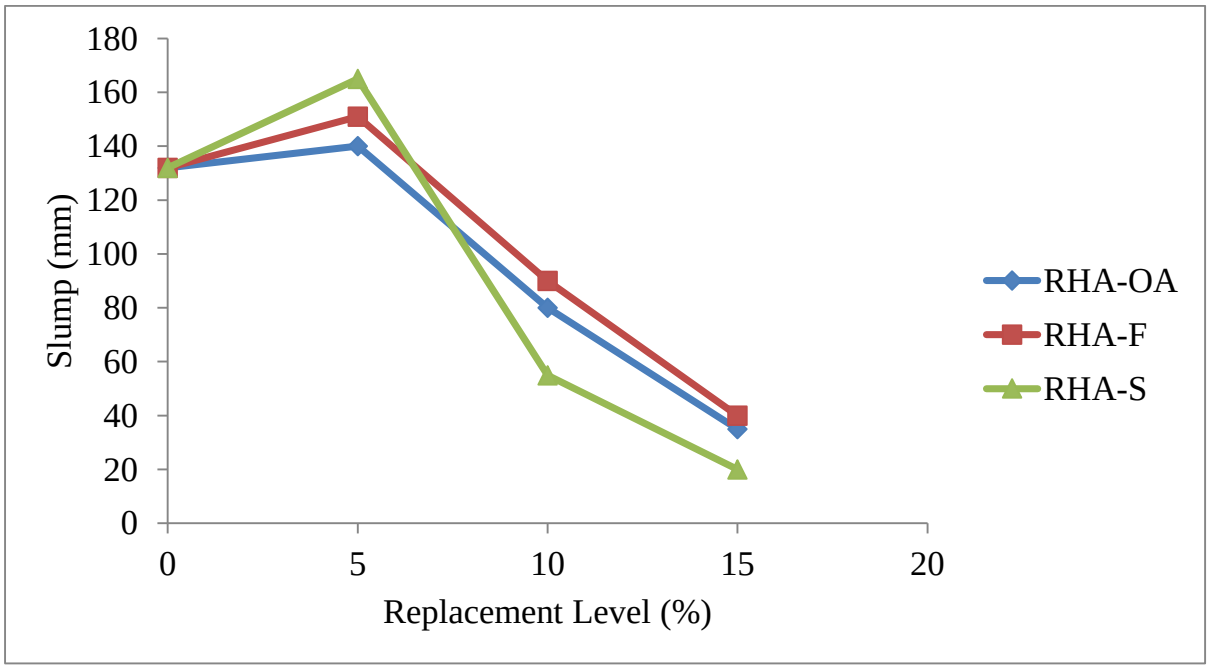

Figure F1: Slump of Control and Rice Husk Ash (RHA) Concrete

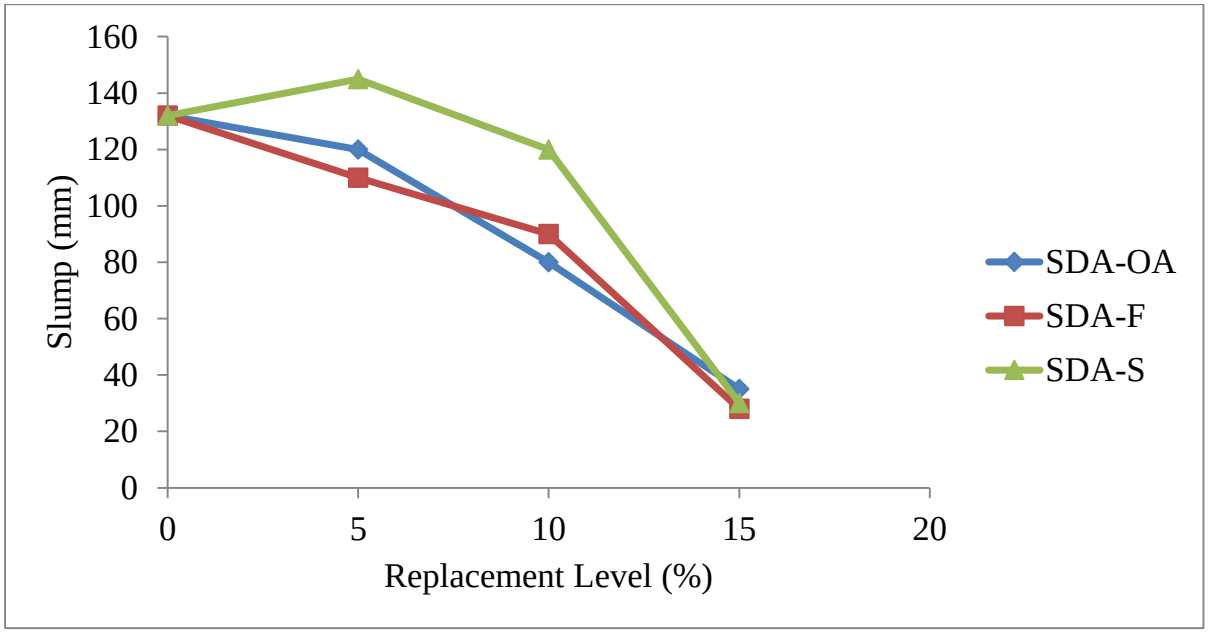

Figure F2: Slump of Control and Saw Dust Ash (SDA) Fresh Concrete

APPENDIX G: RESULTS OF BULK DENSITY

Table G1: Bulk Density of Control Concrete Beams (100% OPC)

Beam Mark	Mass of Beam (kg)		Density of Beam (kg/m^3)		Average Density of Beam (kg/m^3)		Curing Age (Days)
	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	
1	32.30	32.90	2392.59	2437.04	2412.35	2446.91	28
2	33.20	33.40	2459.26	2474.07			
3	32.20	32.80	2385.19	2429.63			
1	33.50	34.40	2481.48	2548.15	2466.67	2535.80	90
2	32.60	33.70	2414.82	2496.30			
3	33.80	34.60	2503.70	2562.96			
1	34.80	35.00	2557.78	2592.59	2479.01	2548.15	150
2	32.80	35.1	2429.63	2600.00			
3	32.80	33.1	2429.63	2451.85			

Table G2: 28th Day Bulk Density of RHA-OA Concrete Beams

Beam Mark	Mass of Beam (kg)		Density of Beam (kg/m^3)		Average Density of Beam (kg/m^3)		Replacement Level (%)
	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	
1	33.20	33.40	2459.26	2474.07	2476.54	2503.70	5
2	33.60	34.00	2488.89	2518.51			
3	33.50	34.00	2481.48	2518.51			
1	31.30	33.60	2318.52	2488.89	2385.19	2449.38	10
2	32.40	32.60	2400.00	2414.81			
3	32.90	33.00	2437.04	2444.44			

1	31.10	31.80	2303.70	2355.56	2338.27	2358.03	15
2	31.50	31.50	2333.33	2333.33			
3	32.10	32.20	2377.78	2385.19			

Table G3: 28th Day Bulk Density of RHA-F Concrete Beams

Beam Mark	Mass of Beam (kg)		Density of Beam (kg/m^3)		Average Density of Beam (kg/m^3)		Replacement Level (%)
	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	
1	33.50	33.60	2481.48	2488.89	2446.91	2459.26	5
2	32.80	32.90	2429.63	2437.04			
3	32.80	33.10	2429.63	2451.85			
1	32.00	32.40	2370.37	2400.00	2385.19	2414.82	10
2	32.40	32.80	2400.00	2429.63			
3	32.20	32.60	2385.19	2414.82			
1	31.90	32.00	2362.96	2370.37	2402.47	2419.75	15
2	32.70	33.00	2422.22	2444.44			
3	32.70	33.00	2422.22	2444.44			

Table G4: 28th Day Bulk Density of RHA-S Concrete Beams

Beam Mark	Mass of Beam (kg)		Density of Beam (kg/m^3)		Average Density of Beam (kg/m^3)		Replacement Level (%)
	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	
1	32.20	33.20	2385.19	2459.26	2385.19	2459.27	5
2	31.00	33.00	2296.30	2444.44			
3	33.40	33.40	2474.07	2474.07			
1	32.60	32.60	2414.81	2414.81	2496.29	2506.17	10
2	33.50	33.90	2481.48	2511.11			
3	35.00	35.00	2592.59	2592.59			

1	32.80	33.00	2429.63	2444.44	2348.15	2365.43	15
2	31.60	31.80	2340.74	2355.56			
3	30.70	31.00	2274.07	2296.30			

Table G5: 28th Day Bulk Density of SDA-OA Concrete Beams

Beam Mark	Mass of Beam (kg)		Density of Beam (kg/m^3)		Average Density of Beam (kg/m^3)		Replacement Level (%)
	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	
1	32.40	32.80	2400.00	2429.63	2488.89	2503.64	5
2	33.60	33.80	2488.89	2503.70			
3	34.80	35.00	2577.78	2592.59			
1	31.90	33.90	2362.96	2511.11	2429.63	2486.42	10
2	33.40	33.70	2496.30	2496.30			
3	32.80	33.1	2429.63	2451.85			
1	30.00	30.20	2222.22	2237.04	2350.62	2362.96	15
2	31.90	32.10	2362.96	2377.78			
3	33.30	33.40	2466.67	2474.07			

Table G6: 28th Day Bulk Density of SDA-F Concrete Beams

Beam Mark	Mass of Beam (kg)		Density of Beam (kg/m^3)		Average Density of Beam (kg/m^3)		Replacement Level (%)
	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	
1	33.00	32.20	2444.44	2385.19	2414.81	2422.22	5
2	31.80	32.90	2355.56	2437.04			
3	33.00	33.00	2444.44	2444.44			
1	33.40	33.60	2474.07	2488.89	2422.22	2441.98	10
2	32.40	32.70	2400.00	2422.22			
3	32.30	32.60	2392.59	2414.82			

1	31.80	32.40	2355.56	2400.00	2367.90	2370.37	15
2	31.20	31.20	2311.11	2311.11			
3	32.90	32.40	2437.04	2400.00			

Table G7: 28th Day Bulk Density of SDA-S Concrete Beams

Beam Mark	Mass of Beam (kg)		Density of Beam (kg/m^3)		Average Density of Beam (kg/m^3)		Replacement Level (%)
	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	
1	33.00	34.20	2444.44	2533.33	2459.26	2533.33	5
2	32.60	33.60	2414.81	2488.89			
3	34.00	34.80	2518.52	2577.78			
1	32.90	33.70	2437.04	2496.30	2427.16	2479.01	10
2	32.20	32.90	2385.19	2437.04			
3	33.20	33.80	2459.26	2503.70			
1	32.20	34.50	2385.19	2555.56	2429.62	2533.34	15
2	32.90	33.70	2437.04	2496.30			
3	33.30	34.4	2466.67	2548.15			

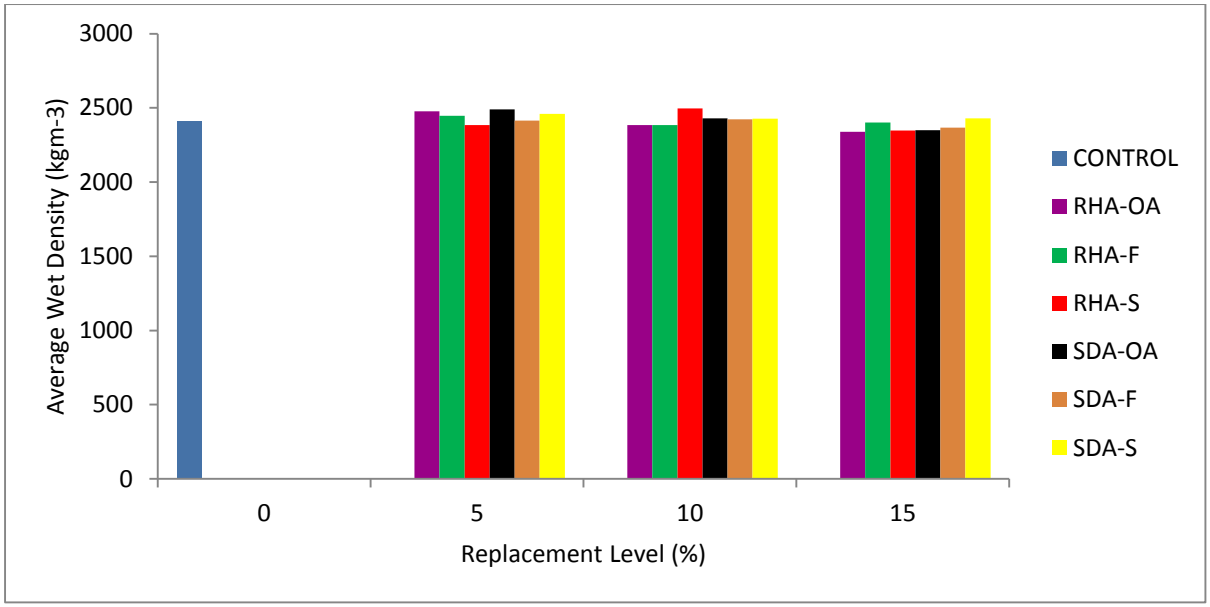

Figure G1: 28th Day Wet Density of Concrete Beam for RHA and SDA

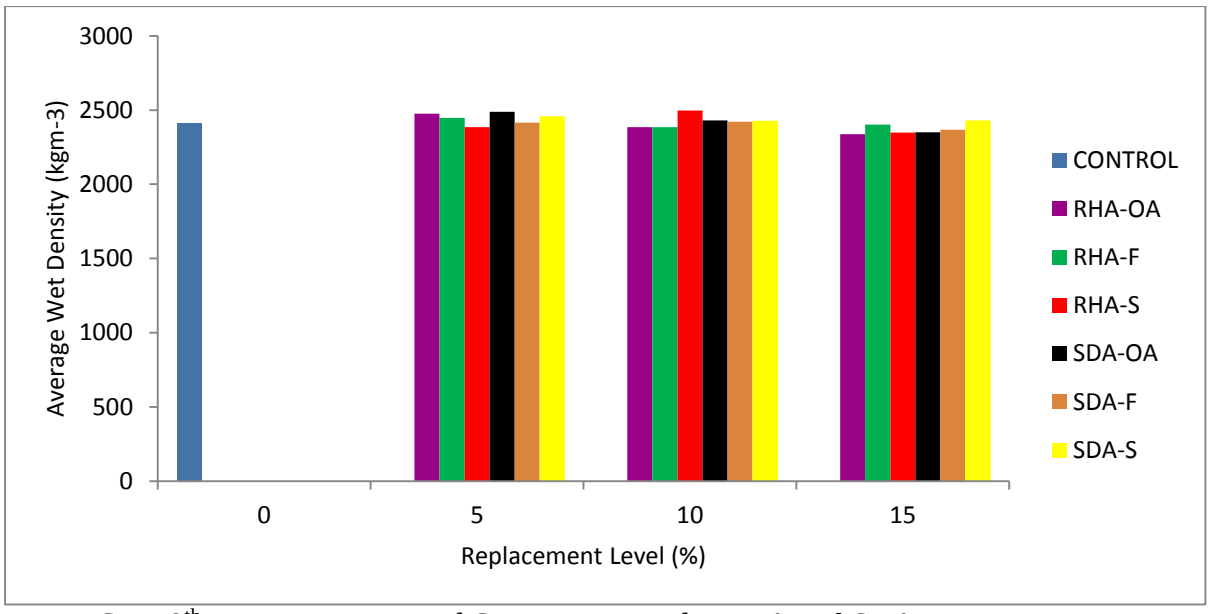

Figure G2: 28th Day Dry Density of Concrete Beam for RHA and SDA

Table G8: 90th Day Bulk Density of RHA-OA Concrete Beams

Beam Mark	Mass of Beam (kg)		Density of Beam (kg/m^3)		Average Density of Beam (kg/m^3)		Replacement Level (%)
	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	
1	33.40	34.20	2474.07	2533.33	2419.75	2479.01	5
2	31.60	32.00	2340.74	2370.37			

3	33.00	34.20	2444.44	2533.33			
1	32.10	33.30	2377.78	2466.67	2382.72	2407.41	10
2	31.80	31.80	2355.56	2355.56			
3	32.60	32.40	2414.81	2400.00			
1	32.70	33.00	2422.22	2444.44	2367.90	2385.53	15
2	31.10	31.60	2303.70	2340.74			
3	32.10	32.50	2377.78	2407.41			

Table G9: 90th Day Bulk Density of RHA-F Concrete Beams

Beam Mark	Mass of Beam (kg)		Density of Beam (kg/m^3)		Average Density of Beam (kg/m^3)		Replacement Level (%)
	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	
1	33.60	33.70	2488.89	2496.30	2474.07	2491.36	5
2	32.70	33.00	2422.22	2444.44			
3	33.90	34.20	2511.11	2533.33			
1	32.00	33.80	2370.37	2503.70	2370.37	2464.20	10
2	32.40	34.10	2400.00	2525.93			
3	31.60	31.90	2340.74	2362.96			
1	33.90	34.50	2511.11	2555.56	2429.63	2486.42	15
2	32.40	32.80	2400.00	2429.63			
3	32.10	33.40	2377.78	2474.07			

Table G10: 90th Day Bulk Density of RHA-S Concrete Beams

Beam Mark	Mass of Beam (kg)	Density of Beam (kg/m^3)	Average Density of Beam (kg/m^3)	Replacement Level (%)

	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	
1	33.10	33.30	2451.85	2466.67	2481.48	2493.83	5
2	33.80	34.00	2503.70	2518.52			
3	33.60	33.70	2488.89	2496.30			
1	33.00	33.30	2444.44	2466.67	2439.50	2456.79	10
2	33.00	33.20	2444.44	2459.26			
3	32.80	33.00	2429.63	2444.44			
1	33.60	33.80	2488.89	2503.70	2375.31	2390.12	15
2	32.80	33.00	2429.63	2444.44			
3	29.80	30.00	2207.41	2222.22			

Table G11: 90th Day Bulk Density of SDA-OA Concrete Beams

Beam Mark	Mass of Beam (kg)		Density of Beam (kg/m^3)		Average Density of Beam (kg/m^3)		Replacement Level (%)
	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	
1	33.70	33.80	2496.30	2503.70	2520.99	2525.93	5
2	34.50	34.60	2555.55	2562.96			
3	33.90	33.90	2511.11	2511.11			
1	32.00	32.20	2370.37	2385.19	2429.63	2439.51	10
2	33.00	33.20	2444.44	2459.26			
3	33.40	33.40	2474.07	2474.07			
1	31.30	32.00	2318.52	2370.37	2395.06	2424.69	15
2	32.40	32.70	2400.00	2422.22			
3	33.30	33.50	2466.67	2481.48			

Table G12: 90th Day Bulk Density of SDA-F Concrete Beams

Beam Mark	Mass of Beam (kg)		Density of Beam (kg/m^3)		Average Density of Beam (kg/m^3)		Replacement Level (%)
	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	
1	32.70	33.00	2422.22	2444.44	2429.63	2451.85	5
2	33.00	33.20	2444.44	2459.26			
3	32.70	33.10	2422.22	2451.85			
1	33.20	33.40	2459.26	2474.07	2422.22	2437.04	10
2	31.80	32.10	2355.56	2377.78			
3	33.10	33.20	2451.85	2459.26			
1	32.60	33.20	2414.81	2459.26	2395.06	2427.16	15
2	32.00	32.40	2370.37	2400.00			
3	32.40	32.70	2400.00	2422.22			

Table G13: 90^{th} Day Bulk Density of SDA-S Concrete Beams

Beam Mark	Mass of Beam (kg)		Density of Beam (kg/m^3)		Average Density of Beam (kg/m^3)		Replacement Level (%)
	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	
1	32.60	33.70	2414.81	2496.30	2412.34	2488.89	5
2	32.00	33.20	2370.37	2459.26			
3	33.10	33.90	2451.85	2511.11			
1	33.20	34.50	2459.26	2555.56	2432.10	2501.24	10
2	32.60	33.30	2414.81	2466.67			
3	32.70	33.50	2422.22	2481.48			
1	32.10	32.60	2377.78	2414.81	2412.35	2412.35	15
2	32.40	32.80	2400.00	2429.63			
3	33.20	33.90	2459.26	2511.11			

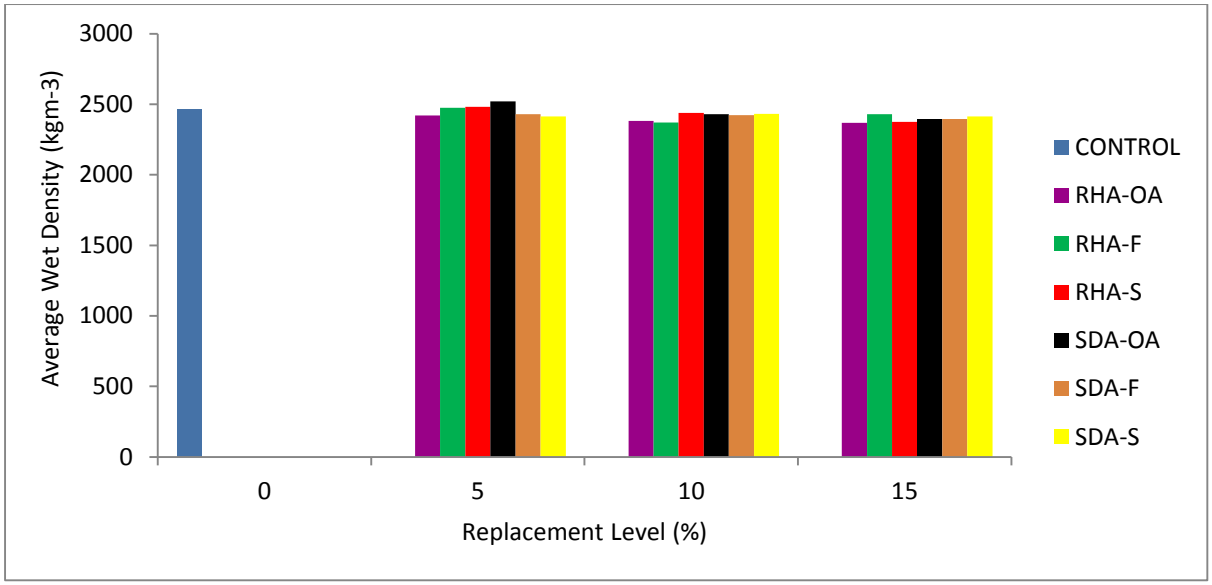

Figure G3: 90th Day Wet Density of Concrete Beam for RHA and SDA

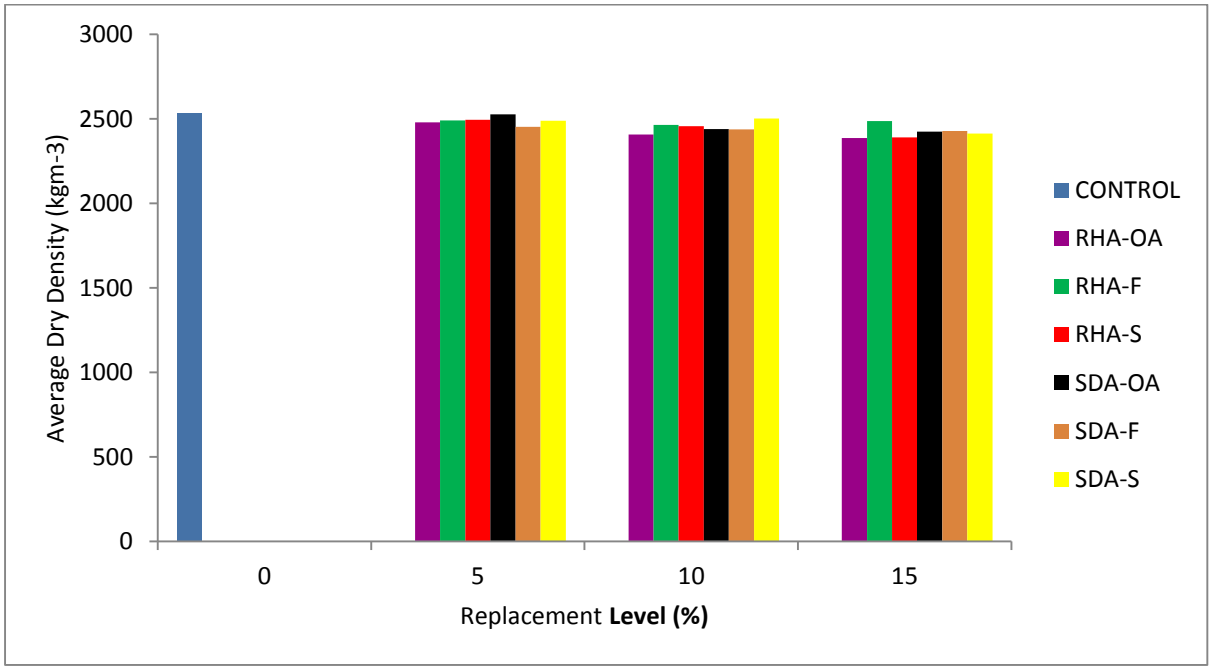

Figure G4: 90th Day Dry Density of Concrete Beam for RHA and SDA

Table G14: 150th Day Bulk Density of RHA-OA Concrete Beams

Beam Mark	Mass of Beam (kg)		Density of Beam (kg/m^3)		Average Density of Beam (kg/m^3)		Replacement Level (%)
	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	
1	32.50	32.60	2407.41	2414.82			

2	32.70	32.80	2422.22	2429.63	2414.81	2422.22	5
3	32.60	32.7	2414.81	2422.22			
1	32.80	33.00	2429.63	2444.44	2444.44	2498.75	10
2	33.20	33.80	2459.26	2503.70			
3	33.00	34.40	2444.44	2548.14			
1	32.20	33.80	2385.19	2503.70	2397.53	2474.07	15
2	31.90	32.20	2362.96	2385.19			
3	33.00	34.20	2444.44	2533.33			

Table G15: 150th Day Bulk Density of RHA-F Concrete Beams

Beam Mark	Mass of Beam (kg)		Density of Beam (kg/m^3)		Average Density of Beam (kg/m^3)		Replacement Level (%)
	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	
1	33.60	34.00	2488.89	2518.52	2513.58	2538.27	5
2	34.20	34.40	2533.33	2548.15			
3	34.00	34.40	2518.52	2548.15			
1	36.30	36.40	2688.89	2696.30	2516.05	2540.74	10
2	33.50	33.80	2481.48	2503.70			
3	32.10	32.70	2377.78	2422.22			
1	31.60	32.20	2340.74	2385.19	2404.94	2437.04	15
2	32.60	33.00	2414.81	2444.44			
3	33.20	33.50	2459.26	2481.48			

Table G16: 150th Day Bulk Density of RHA-S Concrete Beams

Beam Mark	Mass of Beam (kg)	Density of Beam (kg/m^3)	Average Density of Beam (kg/m^3)	Replacement Level (%)

	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	
1	35.10	35.30	2600.00	2614.82	2539.80	2555.56	5
2	33.80	34.00	2509.70	2518.52			
3	33.80	34.20	2509.70	2533.33			
1	33.20	33.50	2459.26	2481.48	2456.32	2476.54	10
2	33.80	34.00	2509.70	2518.52			
3	32.40	32.80	2400.00	2429.63			
1	32.00	32.60	2370.37	2414.82	2385.18	2429.63	15
2	32.00	32.80	2370.37	2429.63			
3	32.60	33.00	2414.81	2444.44			

Table G17: 150th Day Bulk Density of SDA-OA Concrete Beams

Beam Mark	Mass of Beam (kg)		Density of Beam (kg/m^3)		Average Density of Beam (kg/m^3)		Replacement Level (%)
	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	
1	34.80	35.00	2577.78	2592.59	2518.52	2543.21	5
2	34.00	34.40	2518.52	2548.15			
3	33.20	33.60	2459.26	2488.89			
1	33.80	34.00	2509.70	2518.52	2532.39	2545.68	10
2	34.80	35.10	2577.78	2600.00			
3	33.80	34.00	2509.70	2518.52			
1	33.20	34.20	2459.26	2533.33	2513.58	2560.49	15
2	34.20	34.70	2533.33	2570.37			
3	34.40	34.80	2548.15	2577.78			

Table G18:150th Day Bulk Density of SDA-F Concrete Beams

Beam Mark	Mass of Beam (kg)	Density of Beam (kg/m^3)	Average Density of Beam (kg/m^3)	Replacement Level (%)

	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	
1	31.80	32.70	2355.56	2422.22	2335.80	2397.53	5
2	31.20	32.60	2311.11	2414.82			
3	31.60	31.80	2340.74	2355.56			
1	32.80	33.80	2429.63	2503.70	2429.63	2488.89	10
2	33.40	33.90	2474.07	2511.11			
3	32.20	33.1	2385.19	2451.85			
1	30.40	32.80	2251.85	2429.63	2283.95	2444.44	15
2	31.20	33.80	2311.11	2503.70			
3	30.90	32.40	2288.89	2400.00			

Table G19: 150th Day Bulk Density of SDA-S Concrete Beams

Beam Mark	Mass of Beam (kg)		Density of Beam (kg/m^3)		Average Density of Beam (kg/m^3)		Replacement Level (%)
	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	Wet Concrete	Dry Concrete	
1	32.50	33.70	2407.41	2496.30	2424.69	2535.80	5
2	32.70	34.20	2422.22	2533.33			
3	33.00	34.80	2444.44	2577.78			
1	32.60	33.70	2414.82	2496.30	2395.06	2464.20	10
2	32.90	33.80	2437.04	2503.70			
3	31.50	32.30	2333.33	2392.59			
1	30.20	31.80	2237.04	2355.56	2323.46	2407.41	15
2	31.30	32.00	2318.52	2370.37			
3	32.60	33.70	2414.82	2496.30			

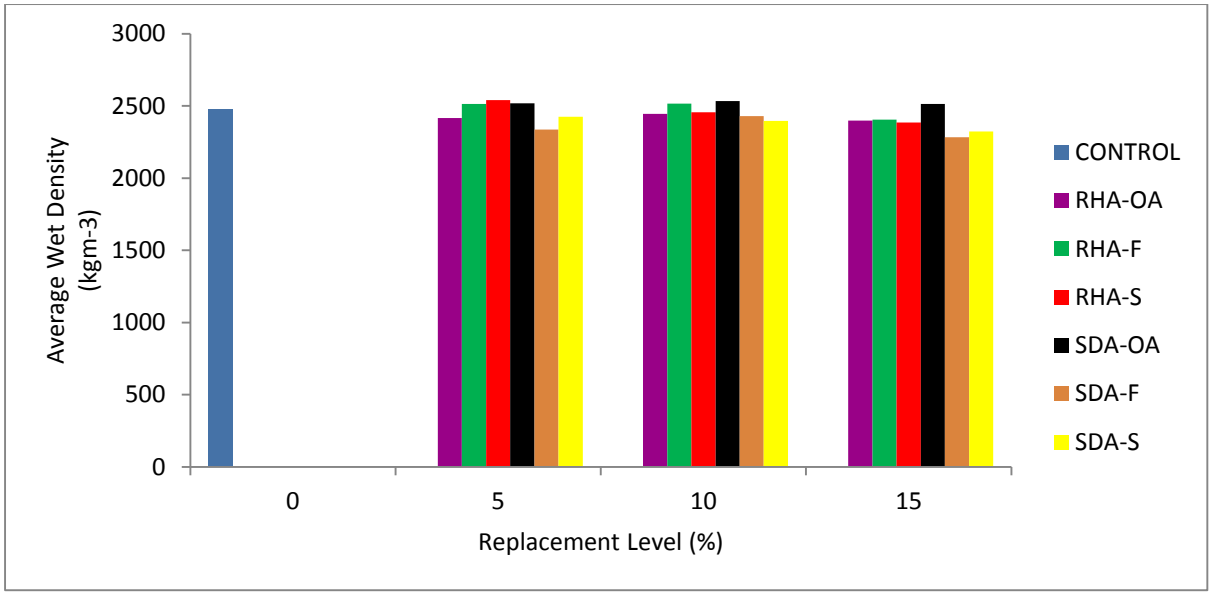

Fig.G5: 150th Day Wet Density of Concrete Beams for RHA and SDA

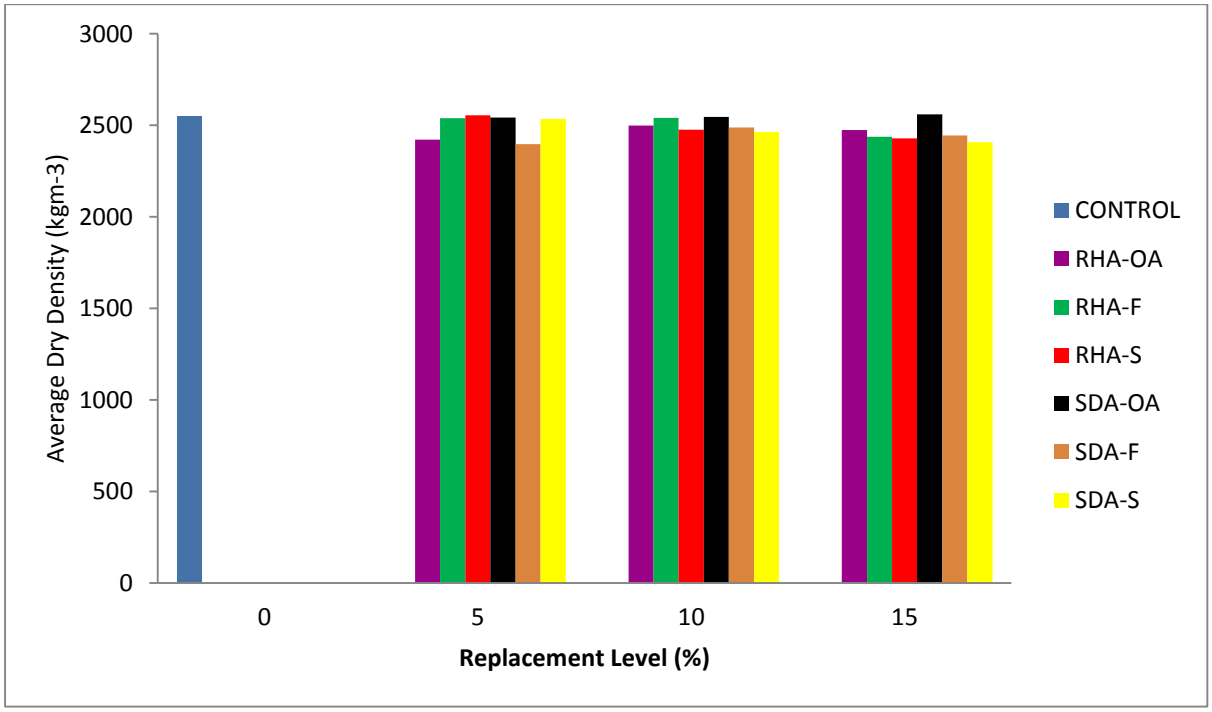

Fig.G6: 150th Day Dry Density of Concrete Beams for RHA and SDA

APPENDIX H: MULTIPLE REGRESSION ANALYSIS

SUMMARY OUTPUT

H1: RHA-OA

Regression Statistics	
Multiple R	0.980554
R Square	0.961486
Adjusted R Square	0.952927
Standard Error	0.267523
Observations	12

ANOV A

	df	SS	MS	F	Significance F
Regression	2	16.07991	8.039954	112.3393	4.32E-07
Residual	9	0.644116	0.071568		
Total	11	16.72403			

H2: RHA-F

Regression Statistics	
Multiple R	0.947761
R Square	0.898251
Adjusted R Square	0.87564
Standard Error	0.414593
Observations	12

ANOVA

	df	SS	MS	F	Significance F
Regression	2	13.65691	6.828453	39.72632	3.42E-05
Residual	9	1.546986	0.171887		
Total	11	15.20389			

H3: RHA-S

Regression Statistics	
Multiple R	0.826756
R Square	0.683526
Adjusted R Square	0.613199
Standard Error	0.816253
Observations	12

ANOVA

	df	SS	MS	F	Significance F
Regression	2	12.9512	6.4756	9.719189	0.005643
Residual	9	5.996426	0.66627		
Total	11	18.94763			

H4: SDA-OA

Regression Statistics	
Multiple R	0.876641
R Square	0.768499

Adjusted R Square	0.717054
Standard Error	0.627629
Observations	12

ANOVA

	df	SS	MS	F	Significance F
Regression	2	11.76896	5.884482	14.93834	0.001382
Residual	9	3.545262	0.393918		
Total	11	15.31423			

H5: SDA-F

Regression Statistics	
Multiple R	0.97733
R Square	0.955175
Adjusted R Square	0.945214
Standard Error	0.329916
Observations	12

ANOVA

	Df	SS	MS	F	Significance F
Regression	2	20.87422	10.43711	95.89011	8.55E-07
Residual	9	0.979601	0.108845		
Total	11	21.85383			

H6: SDA-S

Regression Statistics	
Multiple R	0.831742
R Square	0.691795
Adjusted R Square	0.623305
Standard Error	0.719665
Observations	12

ANOVA

	df	SS	MS	F	Significance F
Regression	2	10.46264	5.231319	10.10067	0.005009
Residual	9	4.661262	0.517918		

Total	11	15.1239	

APPENDIX I: TEST OF ADEQUACY OF THE MULTIPLE REGRESSIONS ANALYSIS MODEL

Table I1: t-Test for RHA-OA

F-Test Two-Sample for Variances		
	Variable 1	Variable 2
Mean	4.5475	4.5475
Variance	1.520365909	1.461809676
Observations	12	12
Df	11	11
F	1.040057358	
P(F<=f) one-tail	0.47461727	
F Critical one-tail	2.81793047	

t-Test: Two-Sample Assuming Unequal Variances		
	Variable 1	Variable 2
Mean	4.5475	4.5475
Variance	1.520365909	1.46180968
Observations	12	12
Hypothesized Mean Difference	0	
Df	22	
t Stat	1.78166E-15	
P(T<=t) one-tail	0.5	
t Critical one-tail	1.717144374	
P(T<=t) two-tail	1	
t Critical two-tail	2.073873068	

Table I2: t-Test for RHA-F

F-Test Two-Sample for Variances	t-Test: Two-Sample Assuming Unequal Variances

	Variable 1	Variable 2
Mean	5.199166667	5.199166667
Variance	1.38217197	1.241536817
Observations	12	12
Df	11	11
F	1.113275057	
P(F<=f) one-tail	0.430966842	
F Critical one-tail	2.81793047	

	Variable 1	Variable 2
Mean	5.199166667	5.19916667
Variance	1.38217197	1.24153682
Observations	12	12
Hypothesized Mean Difference	0	
Df	22	
t Stat	0	
P(T<=t) one-tail	0.5	
t Critical one-tail	1.717144374	
P(T<=t) two-tail	1	
t Critical two-tail	2.073873068	

Table I3: t-Test for RHA-S

F-Test Two-Sample for Variances		
	Variable 1	Variable 2
Mean	4.4325	4.432499833
Variance	1.722511364	1.177381811
Observations	12	12
Df	11	11
F	1.463001507	
P(F<=f) one-tail	0.269273918	
F Critical one-tail	2.81793047	

t-Test: Two-Sample Assuming Unequal Variances		
	Variable 1	Variable 2
Mean	4.4325	4.43249983
Variance	1.722511364	1.17738181
Observations	12	12
Hypothesized Mean Difference	0	
Df	21	
t Stat	3.39038E-07	
P(T<=t) one-tail	0.499999866	
t Critical one-tail	1.720742903	
P(T<=t) two-tail	0.999999733	
t Critical two-tail	2.079613845	

Table I4: t-Test for SDA-OA

F-Test Two-Sample for Variances		
	Variable 1	Variable 2
Mean	4.4725	4.472500167
Variance	1.392202273	1.069905766
Observations	12	12
Df	11	11

t-Test: Two-Sample Assuming Unequal Variances		
	Variable 1	Variable 2
Mean	4.4725	4.47250017
Variance	1.392202273	1.06990577
Observations	12	12
Hypothesized Mean Difference	0	

F	1.301238218	Df	22
P(F<=f) one-tail	0.334958453	t Stat	-3.6795E-07
F Critical one-tail	2.81793047	P(T<=t) one-tail	0.499999855
		t Critical one-tail	1.717144374
		P(T<=t) two-tail	0.99999971
		t Critical two-tail	2.073873068

Table I5: t-Test for SDA-F

F-Test Two-Sample for Variances			t-Test: Two-Sample Assuming Unequal Variances		
	Variable 1	Variable 2		Variable 1	Variable 2
Mean	4.4825	4.4825	Mean	4.4825	4.4825
Variance	1.986711364	1.897656609	Variance	1.986711364	1.89765661
Observations	12	12	Observations	12	12
			Hypothesized Mean		
Df	11	11	Difference	0	
F	1.046928804		Df	22	
P(F<=f) one-tail	0.470369162		t Stat	1.5611E-15	
F Critical one-tail	2.81793047		P(T<=t) one-tail	0.5	
			t Critical one-tail	1.717144374	
			P(T<=t) two-tail	1	
			t Critical two-tail	2.073873068	

Table I6: t-Test for SDA-S

F-Test Two-Sample for Variances			t-Test: Two-Sample Assuming Unequal Variances		
	Variable1	Variable2		Variable 1	Variable2
Mean	4.485	4.485	Mean	4.485	4.485
Variance	1.3749	0.951149	Variance	1.3749	0.95115
Observations	12	12	Observations	12	12
			Hypothesized Mean		
Df	11	11	Difference	0	
F	1.445515		Df	21	
P(F<=f) one-tail	0.275694		t Stat	0	
F Critical one-tail	2.81793		P(T<=t) one-tail	0.5	
			t Critical one-tail	1.720743	
			P(T<=t) two-tail	1	
			t Critical two-tail	2.079614	

APPENDIX J: X-RAY DIFFRACTION OF RHA AND SDA.

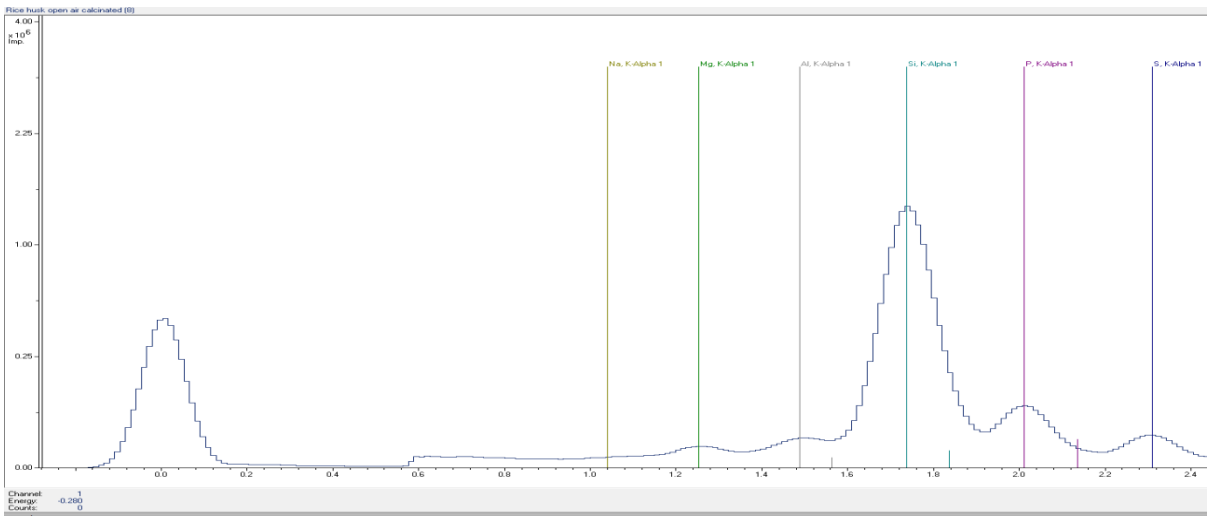

Fig. J1: X-Ray Diffraction of RHA-OA

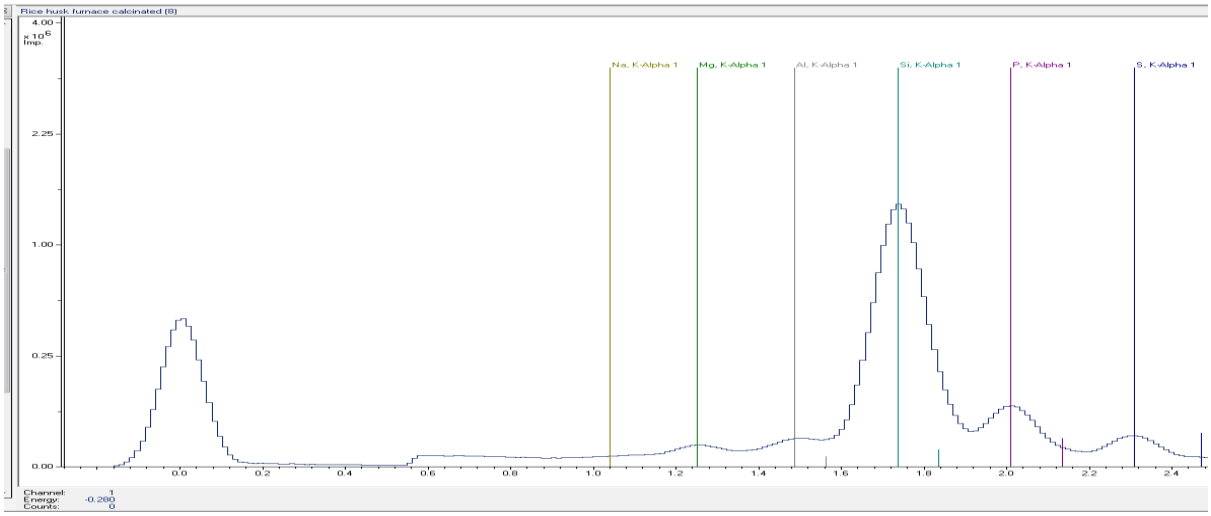

Fig. J2: X-Ray Diffraction of RHA-F

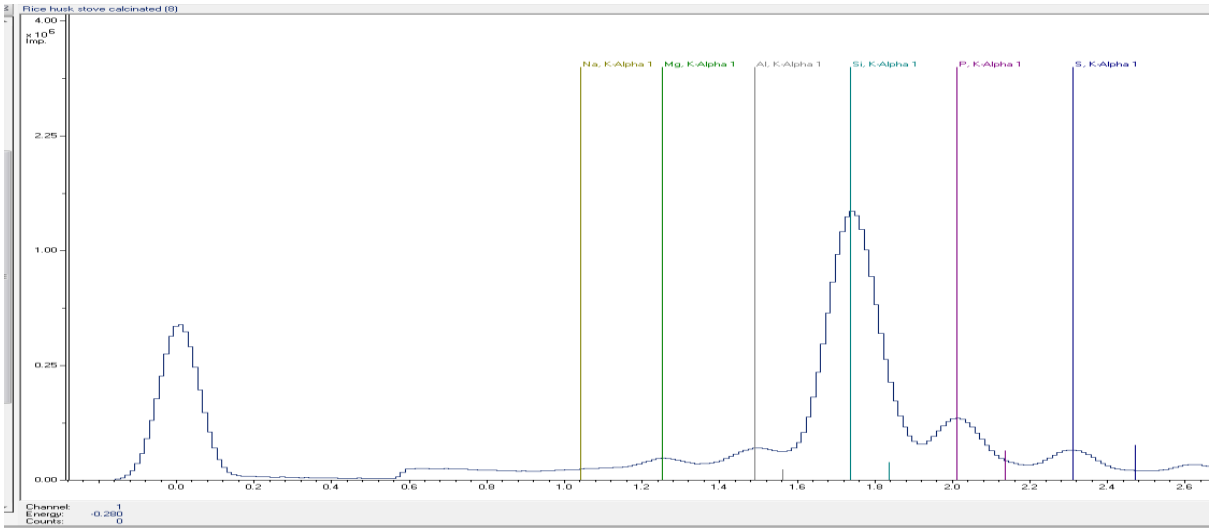

Fig. J3: X-Ray Diffraction of RHA-S

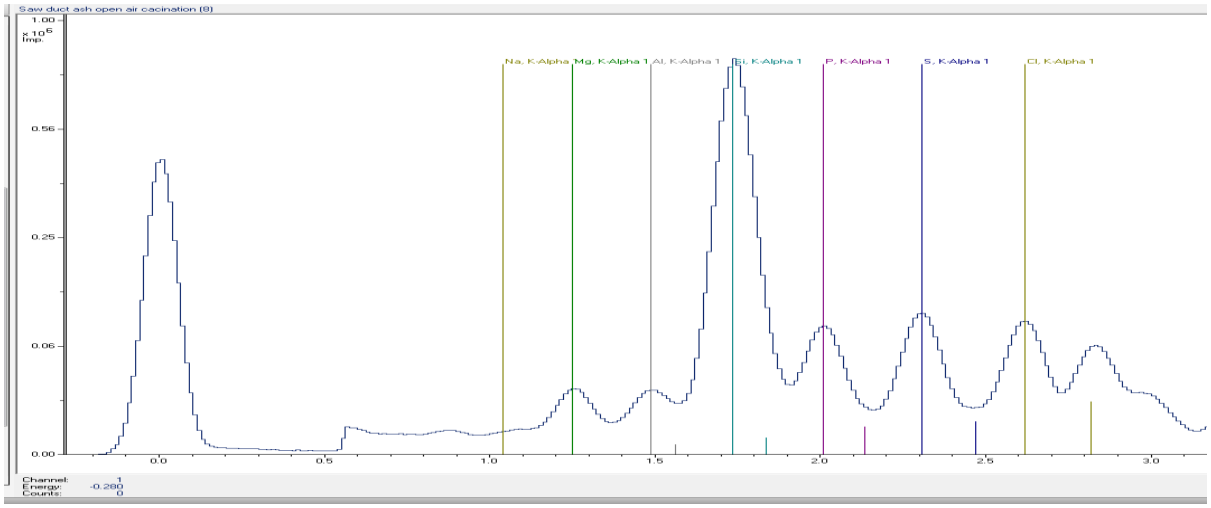

Fig. J4: X-Ray Diffraction of SDA-OA

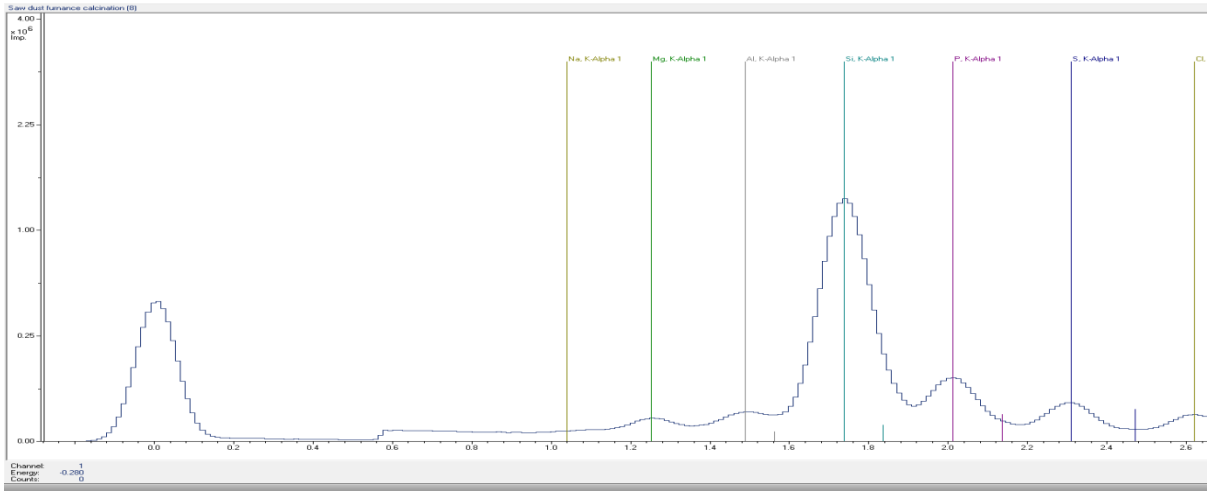

Fig. J5: X-Ray Diffraction of SDA-F

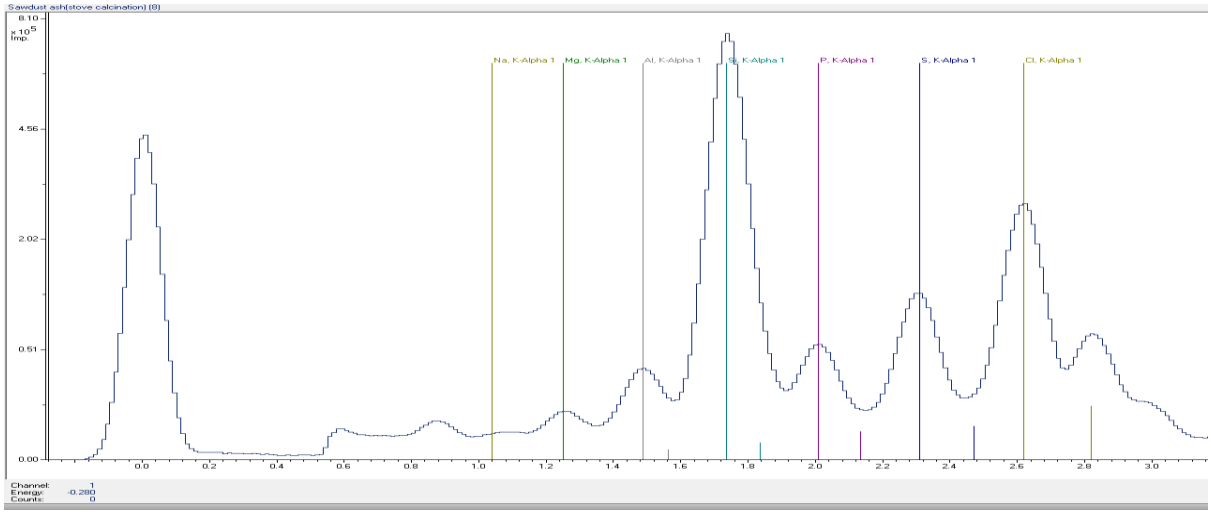

Fig. J6: X-Ray Diffraction of SDA-S

PLATES:

Plate 1: Local stove for burning RHA and SDA

Plate 2: Open-air Calcination of RHA

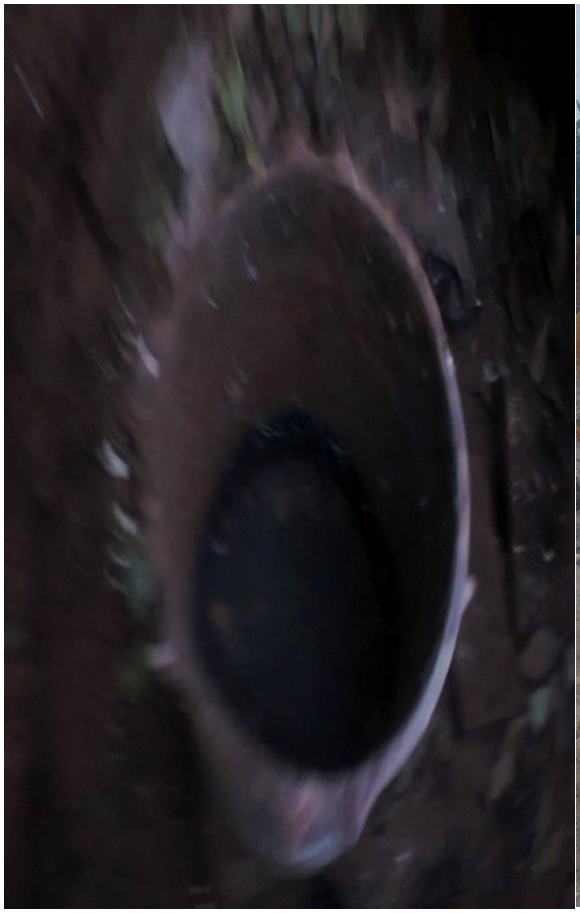

Plate 3: Pit crucible furnace for burning RHA and SDA

Plate 4: SDA from open air calcination method

Plate 5: Moulds