

**COMPARATIVE ANALYSIS OF RICE HUSK ASH AND SAW
DUST ASH ON TENSILE STRENGTHS OF CONCRETE**

BY

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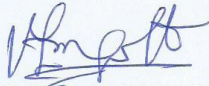
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CERTIFICATION

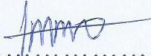
This is to certify that this work, “Comparative Analysis of Rice Husk and Sawdust Ash on Tensile Strengths of Concrete.” was carried out by **Chidiebere Sampson Ezenkwa** (B.Eng) with Reg. No. **20134872808**, in partial fulfillment of the requirements for the award of Master of Engineering (M.Eng) degree in Civil Engineering (Structures), in the School of Engineering and Engineering Technology, Federal University of Technology Owerri, Nigeria.



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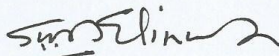
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Dedication

This study is dedicated to Almighty God for giving me the grace and strength to accomplish this great task.

Acknowledgement

I give all the thanks to the almighty God, for his grace and favor bestowed on me in the process of carrying out this study. I also thank my beloved parents and siblings for their kind support.

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Abstract

This study focuses on the comparative analysis of rice husk ash (RHA) and saw dust ash (SDA) on tensile strengths of concrete. RHA and SDA were obtained from open burning. The physical properties and chemical composition of the ashes were investigated. Ordinary Portland cement (OPC) was replaced with RHA, SDA, and RHA-SDA at 5%, 10%, and 15%. Concrete mix design was done using SCALE149 (Structural Calculations Ensemble) Software. Nine concrete cylinders of dimension 150×300mm with 100% OPC or 0% replacement with pozzolan were produced. Eighty one concrete cylinders were also produced for each of the percentage replacement of OPC with RHA, SDA, RHA-SDA, making a total of 252 concrete cylinders. The concrete cylinders were cured by immersion and tested for split tensile strength at 28, 90, and 150 days. The results showed that RHA is a better pozzolana with combined ($\text{SiO}_2 + \text{Al}_2\text{O}_3 + \text{Fe}_2\text{O}_3$) of 87.67% compared to SDA which was 50.03%. The Split tensile strength decreased with increasing RHA, SDA, and RHA-SDA OPC percentage replacement. Laboratory values of 0.86N/mm^2 and 0.78N/mm^2 were obtained at 5% and 10% RHA-OPC replacement, 0.77N/mm^2 and 0.58N/mm^2 were obtained at 5% and 10% SDA-OPC replacement. Similarly, 0.82N/mm^2 and 0.64N/mm^2 were obtained at 5% and 10% RHA-SDA-OPC replacement. Split tensile strength of concrete with RHA, SDA, and RHA-SDA were lower at early age of curing but improved at later ages of curing. For example, at 5% RHA-OPC replacement, 1.42N/mm^2 and 1.85N/mm^2 were obtained at 90 and 150 days of curing. At 5% SDA-OPC replacement, 1.28N/mm^2 and 1.65N/mm^2 were obtained at 90 and 150 days of curing. Similarly, at 5% RHA-SDA-OPC replacement, 1.35N/mm^2 and 1.66N/mm^2 were obtained at 90 and 150 days of curing, as against 1.34N/mm^2 and 1.61N/mm^2 attained by the control at 90 and 150 days of curing. Models were developed to predict the tensile strengths of concrete containing RHA and SDA as binary and ternary blend with OPC at varying percentage replacements and curing ages. The models were tested and found to be adequate.

Keywords-Split Tensile strength, Concrete, Pozzolana, Rice husk ash, Saw dust ash, Binary blend, Ternary blend.

CHAPTER ONE

INTRODUCTION

1.1 Background Information

Cementitious binders are vital for all types of construction activities. Researchers have greatly intensified efforts at sourcing local materials that could be used as partial replacement for Ordinary Portland Cement (OPC) in civil engineering works. Supplementary cementitious materials have been proven to be effective in meeting most of the requirements of durable concrete such that blended cements are now used in many parts of the world (Bakar et al., 2010). Calcium hydroxide $[Ca(OH)_2]$ is obtained as one of the hydration products of Ordinary Portland Cement (OPC). It is responsible for the deterioration of concrete. When blended with Portland cement, a pozzolanic material reacts with the $Ca(OH)_2$ to produce additional calcium-silicate-hydrate (C-S-H), which is the main cementing component. Therefore, the pozzolanic material serves to reduce the quantity of the deleterious calcium hydroxide $Ca(OH)_2$ and increase the quantity of the beneficial calcium-silicate-hydrate (C-S-H). Dwivedia et al. (2006) reported that the cementing quality is enhanced if a good pozzolanic material is blended in suitable quantity with OPC. According to Malhotra and Mehta, (1996) pozzolanas are defined as siliceous or siliceous and aluminous materials which in themselves possess little or no cementing property, but will in a finely divided form in the presence of water chemically react with calcium hydroxide at ordinary temperature to form compounds possessing cementitious properties. These materials were first discovered and dogged in Italy, at Pozzuoli, in the region around Vesuvius (McCann, 1994). Industrial waste pozzolans such as fly ash (FA) and silica fume (SF) are already widely used in many countries (Cisse and Laquerbe, 2000). Attempts are also been made to produce and use pozzolanic agricultural by-product ashes such as rice husk ash (RHA) and saw dust ash (SDA) commercially in some countries.

Recent studies by Ettu et al. (2013a), Ettu et al. (2013b), Ettu et al. (2013c) and Ettu et al. (2013d) have confirmed the suitability of Nigerian RHA and SDA as pozzolanic materials for producing concrete, sandcrete, or soilcrete. Malhotra and Mehta, (2004) found that ground RHA with finer particle size than OPC improves concrete properties, including that higher substitution amounts results in lower water absorption values and the addition of RHA causes an increment in the strength of concrete. Fadzil et al. (2008) studied the properties of ternary blended cementitious (TBC) systems containing OPC, ground Malaysian RHA, and fly ash(FA). They found that at long-term period, the compressive strength of TBC concrete was comparable to the control mixes even at OPC replacement of up to 40% with the pozzolanic material. Of all the plant residues, the ash of rice husks contains the highest proportion of silica. The ash of saw dust also contain silica. Under controlled combustion conditions, amorphous silica with high reactivity is produced. It has been demonstrated that RHA and SDA can be added to concrete mixtures to substitute the more expensive Portland cement to lower the construction cost.

The strength of construction material is essential to engineers. The structural engineers have only two simple ways of testing the strength of a material-in tension (tearing apart) or in compression (pressing down). The reason why tensile strength behavior of concrete is of interest is that concrete structures are subjected not only to compressive forces but also to tensile forces. Tensile strength is used in both serviceability and ultimate limit state calculations for example: the evaluation of the cracking moment for prestressed elements, the design of fibre-reinforced concrete, developing moment curvature diagrams and in the calculation of deflection. It is also used in the design of unreinforced concrete sections, for example, concrete pavements. The knowledge of tensile strength is used to estimate the load under which cracking will develop. The cracking is a form of tension failure.

Until the early seventies researchers utilized rice husk ash and saw dust ash derived from uncontrolled combustion in their investigations. Chopra et al. (1981), have reported that for incineration temperatures up to 700⁰C the silica was predominantly in amorphous form and that the crystals present in the ashes grew with time of burning. According to Hamad et al. (1981), ash prepared at a temperature of about 500 to 600⁰C consists of amorphous silica.

In this study, comparative analysis of RHA and SDA on the tensile strength of concrete was investigated. The ashes were burnt using open burning. RHA concrete and SDA concrete were produced. Knowing that RHA has more reactive silica, it was blended with SDA which is less reactive to produce RHA-SDA concrete. A mathematical model for predicting the tensile strength of concrete is also developed using regression analysis in excel spreadsheet.

1.2 Problem Statement

The use of agricultural by-product pozzolans like Rice husk ash (RHA), and Sawdust ash (SDA), as partial ordinary Portland cement (OPC) replacing material have been tested in Nigeria and the world at large. Researchers have worked so much on the compressive and tensile strength of rice husk ash and saw dust ash concrete. Abalaka, (2012) showed that the split tensile strength of concrete cylinders containing 5% and 10% RHA using a charcoal fired incinerator were higher than that of the control at 28days. Also the tensile strength of concrete cylinders containing 5% and 10% RHA were greater than the control at 28days. Saraswathy and Song, (2007) investigated the effect of rice husk ash (RHA) on the compressive, splitting tensile and bond strength of concrete. Sensale, (2006) investigated the influence of rice husk ash (RHA) (10 and 20%) on the compressive and splitting tensile strength of concrete. Elinwa et al. (2008), and Elinwa and Abdulkadir (2011) have also investigated the suitability of sawdust ash as a pozzolanic material and found that it could be

used in binary combination with OPC to improve the properties of cement composites.

Split tensile strength increases in concrete containing RHA and SDA as individual replacement in binary combination have also been reported in other investigations (Godwin et al., 2013; Ganesan, 2008; Nehdi et al., 2003; Sakr, 2006).

The problem now is can we use RHA and SDA in ternary combination with OPC to optimize the split tensile strength of concrete while at the same time satisfying strength and quality requirement?

Hence this study comparative analysis of rice husk ash and saw dust ash on tensile strengths of concrete is very vital because the ashes were used to produce concrete both as binary combination (that is RHA-concrete, SDA-concrete) and ternary combination (that is RHA-SDA-concrete).

1.3 Objectives of the Study

The main objective of this research is to do a comparative analysis of rice husk ash and sawdust ash on Tensile strength of Concrete. The specific objectives are:

- i. To characterize rice husk ash (RHA) and sawdust ash (SDA) as pozzolans
- ii. To determine the effect of varying RHA, SDA, and RHA-SDA to OPC percentages on tensile strength of concrete.
- iii. To Formulate mathematical models of RHA-OPC, SDA-OPC, and RHA-SDA-OPC concrete that will predict the tensile strength of concrete at any given curing age and percentage replacement of OPC with RHA, SDA, and RHA-SDA.

1.4 Justification of the Study

- i. This research will be of immense benefit to owners of concrete industries especially in South-Eastern Region as it will help them to get the required strength with locally available materials.
- ii. The mathematical models developed could be used by any concrete industry and researchers as a basis of generating combinations of mix proportions from which the final product of concrete could be produced. This will save cost and time as trial running operations are eliminated.

1.5 Scope of the Study

This work is limited to the search of locally occurring pozzolanic materials with respect to Rice Husk Ash (RHA) and Saw Dust Ash (SDA) in South Eastern Nigeria that could be used as partial cement replacement of Ordinary Portland Cement. The above materials were prepared from open air calcinations. Experimental determination of the aggregate grading, specific gravity of the materials used, Bulk density and concrete mix design were carried out. The pozzolanas were mixed with OPC in variation in the production of concrete cylinders. These cylinders produced were tested for split tensile strength and findings recorded. It also covers the development of mathematical models for the prediction of tensile strength.

CHAPTER TWO

LITERATURE REVIEW

2.1 Concrete

Cyril, (2006) defined concrete as a composite stone-like material formed by mixing an aggregate (such as stones of irregular shape or crushed rock) with cement (which acts as the binding material) and water, then allowing the mixture to dry and harden.

Prab et al., (2006) defined concrete as a composite material. The dry mix consisting of cement, coarse and fine aggregate.

Concrete referred as plain concrete is a material obtained by cementing together with the inert materials such as sand, gravel, crushed rock or some other suitable material which is hard and durable (Vazirami and Chandola, 2009).

Bhavikatti, (2001) defined Concrete as an intimate mixture of binding materials (like cement, lime, polymers, etc), fine aggregate (sand), coarse aggregate (crushed aggregates of inert stones) and water.

2.1.1 Composition of Concrete

Concrete is composed of: Cement, Aggregates, Water and Admixtures.

2.1.1.1 Cement

Cement is an extremely finely ground product obtained by burning together at high temperature specifically proportioned amounts of calcareous and argillaceous raw materials, adding nothing else to the burnt product except gypsum in small percentage (Singh, 2008).

Nardi, 2002 stipulated that cement is made by heating a material rich in calcium. In general, cement is an adhesive substance which has binding capabilities used in building and civil Engineering construction.

ASTM C150 defines Portland cement as a hydraulic cement produced by pulverizing clinkers consisting essentially of hydraulic calcium silicates, and a small amount of one or more forms of calcium sulfate as an interground addition.

Cement when mixed with water sets and hardens by a chemical process known as hydration. Due to their hydrating properties, constructional cements, which sets and harden under water, are often called hydraulic cements; the most important of these being Ordinary Portland Cement.

i. Types of cement

- a. Portland Cement:- This can be further divided into Ordinary Portland Cement; Rapid Hardening Portland Cement; Sulfate Resisting Portland Cement; and Low Heat Portland Cement.
- b. Slag Cement:- Under this type of cement are Portland Blast Furnace Cement; Low Heat Portland Blast Furnace Cement, and Super Sulfated Cement.
- c. High-Alumina Cement
- d. Pozzolanic Cement

ii. Oxide Composition of Cement

The raw materials used for the manufacture of cement consist mainly of lime, silica, alumina and iron oxide. These oxides interact with one another in the kiln at high temperature to form more complex compounds. The relative proportions of these oxide compositions are responsible for influencing the various properties of cement in addition to the rate of cooling and fineness of grinding.

Table 2.1 Approximate Oxide Composition Limits Of OPC.(Shetty, 2006).

Oxide	Percentage Content
CaO	60-67
SiO ₂	17-25
Al ₂ O ₃	3.0-8.0
Fe ₂ O ₃	0.5-6.0
MgO	0.1-4.0
Alkalytes (K ₂ O, Na ₂ O)	0.4-1.3
SO ₃	1.3-3.0

iii. Hydration of Cement

Anhydrous cement does not bind fine and coarse aggregate. It acquires adhesive property only when mixed with water. The chemical reactions that take place between cement and water is referred to as hydration of cement. Anhydrous cement compounds when mixed with water react with each other to form hydrated compounds of very low solubility. The hydration of cement can be visualized in two ways. The first is through solution mechanism. In this the cement compounds dissolve to produce a supersaturated solution from which different hydrated products get precipitated. The second possibility is that water attacks cement compounds in the solid state converting the compounds into hydrated products starting from the surface and proceeding to the interior of the compounds with time.

The most important hydraulic constituents are the calcium silicates, C₂S and C₃S. Upon mixing with water, calcium silicates react with water molecules to form calcium silicate hydrate (3CaO.2SiO₂.3H₂O) and calcium hydroxide Ca(OH)₂.

iv. Heat of Hydration.

This is the quantity of heat energy (measured in joules per gram) produced upon completion of hydration at a given temperature. The temperature at which hydration takes place greatly affects the rate of heat development during the early stages of concrete. On the other hand, there is little effect of temperature on the long time value of heat of hydration. The most common method of measuring the heat of hydration is by comparing the heat of hydration of unhydrated and hydrated cement in a mixture of nitric acid and hydrofluoric acid; the difference between the values represent the heat of hydration. The heat of hydration measured as described above consists of the chemical heat of hydration and heat of adsorption of water on the surface of the gel formed by the process of hydration. This means that heat of hydration is a composite quantity. It should be noted that it is the rate at which heat is generated and not the total liberated heat that affect the rise in temperature in practice. Therefore, the heat characteristic of a particular type of cement must be taken into consideration while considering its suitability for a specific project, especially in projects such as Dams.

v. Water Requirement for Hydration

It has long been established that the water content of the concrete at the time of hardening plays a large role in determining the ultimate strength and durability of the concrete. It has also been reported that as the water content is increased, the ultimate strength will decrease. This is correct because as the water content is increased, the particles of anhydrous cement are pushed further apart. This reduces the ability of the cement to bond to itself and to the aggregate.

During the hydration process, water is chemically combined with cement in the ratio of approximately 25 grams of water to 100 grams of cement which is about a water-cement ratio of 0.25 plus extra water (physically bonded) that is required to maintain contact with all the cement. The physically bonded water

required is an additional 20 grams of water. Therefore, the total amount of water required for total hydration can be stated as 45 grams of water per 100 grams of cement, or a water/cement ratio of 0.45. In addition, there must be an accounting for the water lost from the system through evaporation. This would require the water-cement ratio to be something higher than 0.45 (Hover, 2011).

vi. Effect of Hydration on Strength of Hardened Concrete.

The rate of hydration at the early age of concrete determines the strength of concrete produced. Under normal conditions, once cement and water are mixed the hydration process will begin and the concrete or mortar will gain strength and increase in durability with the passage of time. If the rate of hydration is high, this helps in early stripping of formwork especially for non-critical structural elements. Therefore, it is necessary that there should be enough water in the cement paste for complete hydration to take place.

2.1.1.2 Aggregates

BS 882: 1992 defines aggregate as a granular material obtained by processing or from natural materials. Aggregate is a collective term for the mineral materials such as sand, gravel and crushed stone that are used with a binding medium (such as bitumen, Portland cement, lime, etc.) to form composite materials (such as asphalt concrete or Portland cement concrete). By volume, aggregate generally accounts for about 70 to 75 percent of concrete (Troxel et al., 1968). Aggregate is also used for base and sub base courses for both flexible and rigid pavements; the aggregate serves as reinforcement to add strength to the overall composite material. Aggregates are much cheaper than cement and maximum economy is obtained by using as much aggregate as possible in concrete making. Apart from economy, aggregate offers stability and durability to concrete. Before now it was common to think of aggregates as inert filler in concrete but the physical and sometimes chemical composition of aggregates

and their characteristics affect to a varying degree the properties of concrete in both the plastic and hardened states (Jackson and Dhir, 1978).

i Sources of Aggregates

Aggregates can either be natural or manufactured. Natural aggregates are generally extracted from larger rock formations through an open excavation (quarry). Extracted rock is typically reduced to usable sizes by mechanical crushing. Manufactured aggregate is often the byproduct of other manufacturing industries. In addition, there are some (minor) materials that are used as special lightweight aggregates: clay, pumice, perlite, and vermiculite. Aggregates can also be sourced from rivers e.g. river stone which have smooth and round surface.

ii. Classification of Aggregates

Aggregate can be classified as fine or coarse. Aggregate mainly passing a 5.0 mm BS 410 test sieve and containing no more coarser material than is permitted for the various grading in this specification are known as fine aggregate (sand) while those larger than 5.0 mm are referred to as coarse aggregate. Fine aggregates produced by crushing rocks to sand sizes is referred to as manufactured sand (Tam, 2003).

a) Classification of Aggregates according to Weight.

The variability in density can be used to produce concrete of widely different unit weights. The most common classification of aggregates on the basis of bulk specific gravity is lightweight, normal-weight, and heavyweight aggregates. (BS EN 206-1:2000)

i) Lightweight aggregates:

Lightweight aggregates are aggregates which may be natural or synthetic and weigh less than 1100 kg/m^3 . The lightweight is due to the cellular or high

internal porous microstructure, which gives this type of aggregate a low bulk specific gravity. The most important aspect of lightweight aggregate is the porosity. They have high absorption values, which require a modified approach to concrete proportioning. For instance, slump loss in lightweight concrete due to absorption can be an acute problem, which can be alleviated by pre-wetting (but not saturating) the aggregate before batching. Lightweight aggregates and their method of testing are covered in BS 3681.

ii) Normal Weight Aggregate.

Normal weight aggregates can be natural or artificial. They are suitable for most purposes and produce concrete of density ranging from 2300 to 2500kg/m³. Normal weight aggregate can be obtained by crushing granite (Igneous) rock to the required size, it can also be provided by extraction of gravel from alluvial deposits or glacial action. Some can also be obtained by dredging of sea or river bed. The properties of aggregate are function of its composition, grain size and texture. For example, granite has a low fire resistance because of high coefficient of expansion of its quartz (metamorphic) content. Sandstone has high porosity and as a result, it produces concrete of high drying shrinkage. Broken-brick aggregate has high fire resistance but should not be used for normal concrete if its sulfate content exceeds 1%. To maximize the potential of an aggregate, it should be washed to remove clay and silt before it is used. The chloride content of marine aggregate should be less than 1% before it can be used as aggregate for structural concrete.

iii) Heavyweight aggregates:

Heavyweight concrete contains aggregates that are natural or synthetic which typically weigh more than 4,000 kg/m³ and can range up to 5,500 kg/m³. Heavy weight aggregate is most commonly used for radiation shielding, counterweights and other applications where a high mass-to-volume ratio is

desired. It is usually very difficult to obtain a mix which is both workable and not prone to segregation using heavyweight aggregate.

b) Classification of Aggregates according to Particle Shape and Texture.

According to shapes, aggregate can be classified as; rounded, irregular, angular, flaky and elongated. The surface texture of an aggregate can be glassy, smooth, granular, rough, crystalline and honeycombed.

Aggregate shape and surface texture influence the properties of freshly mixed concrete more than the properties of hardened concrete. Rough-textured, angular, and elongated particles require more water to produce workable concrete than smooth, rounded compact aggregate. Consequently, the cement content must also be increased to maintain the water-cement ratio. However, with rough aggregates, there is better mechanical bond in the hardened concrete, so strength is higher (if concrete with the same w/c ratio is compared). Hence, when smooth aggregates are replaced with rough aggregates, concrete of similar flow properties and strength can be produced by adding a little bit more water. The external characteristics of aggregates can be assessed by observation and classification in accordance with BS 812. The particle shape can be assessed by direct measurement of the particles to determine the flakiness, elongation and angularity.

iii. Strength Characteristic of Aggregate

a) Aggregate Abrasion

This is the measure of the ability of the aggregate to resist wearing or abrasive force. Aggregate abrasion is tested for mostly in aggregates that will be used in the construction of the floors of warehouses and industrial buildings and aggregate that will be used in the construction of road pavements. The following methods can be used for the determination of abrasion value; Deval attrition Test, Dory Abrasion Test and Los Angeles Test.

b) Aggregate Modulus of Elasticity

The ratio of the stress applied to a body to the strain that results in the body in response to it. The modulus of elasticity of a material is a measure of its stiffness and for most materials remains constant over a range of stress. This depends on the aggregate composition, texture and structure. Modulus of elasticity of aggregate influences properties of concrete such as shrinkage, creep and the elastic behavior of concrete produced with it.

iv. Weight and Moisture Content Characteristics

a) Bulk Density

The bulk density or unit weight of an aggregate provides useful information about shape and grading of the aggregate. This parameter shows how densely the aggregate is packed when filled in a standard manner and it is used in mix design to convert weighed proportions into volumes in the site where weighing equipment is not available. A comprehensive test description for bulk density is found in BS 812-2: 1995.

b) Specific Gravity

This is the ratio of the weight in air of a given volume of a material at a standard temperature to the weight in air of an equal volume of distilled water at the same stated temperature. A comprehensive test description is found in BS 812-2: 1995.

c) Moisture Content

The amount of water that an aggregate can absorb tends to be an excellent indicator as to the strength or weakness of the aggregate. Strong aggregate will have an absorption figure below 1%. Above 4%, you need to perform further test on the aggregate to determine its acceptability. A test for aggregate moisture absorption is described in BS EN 1097-6: 2000.

d) Bulking of Fine Aggregate

This is the increase in volume of a given mass of sand caused by layer of water pushing the sand particles apart. This does not have any effect on the proportioning of materials when batching is by mass. In the case of volume batching, bulking results in the deficiency of fine aggregate and the mix will appear stony. The concrete produced in this case may be prone to segregation and honeycombing. The degree of bulking depends on the amount of moisture present in the sand and its fineness. Fine sand bulk more and a very fine sand bulk even up to 35- 40%. Maximum Bulking occur at particular moisture content as shown in the Figure 2.1 below (Schrader, 2007).

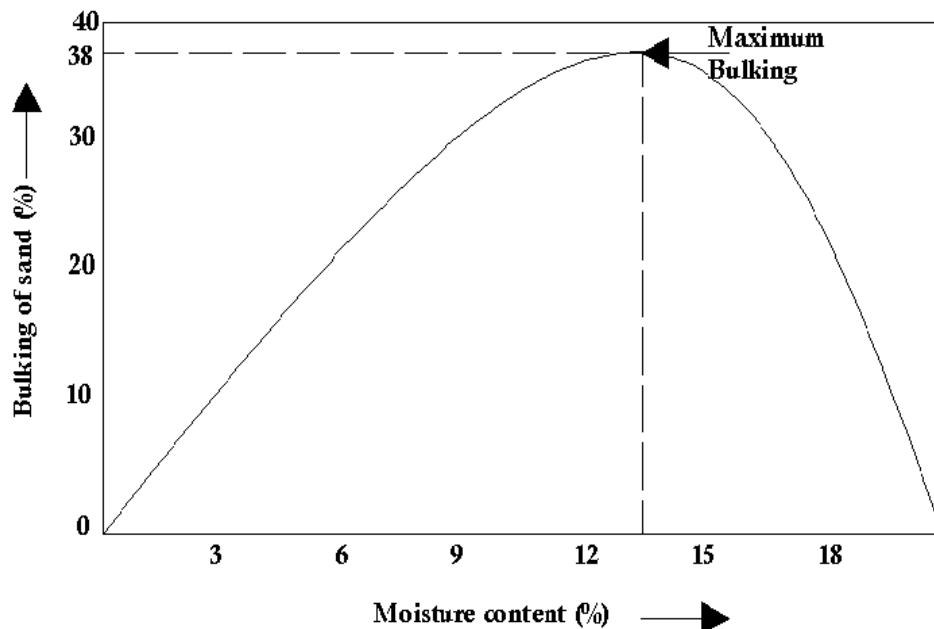


Figure 2.1: Bulking of Fine Aggregate

v. Grading of Aggregates.

The particle size distribution of an aggregate (gradation), shape, and surface texture have an important effect on the workability and finishing of fresh concrete and definitely on the properties of hardened concrete. If an aggregate is well graded, that is, it contains all standard fractions of aggregate in the right proportion, such that the sample contain minimum void, minimum paste will be

required to fill the void. Therefore, the concrete produced using such aggregate will consume less cement, less water, which means increased economy, higher strength, lower shrinkage and greater durability.

The size of aggregate particles normally used in making concrete varies from 37.5 to 0.15 mm. BS 882 places aggregate in three different categories: fine aggregate (sand) containing particles majority of which are smaller than 5mm, coarse aggregate which has particles that are greater than 4.75mm and All-in aggregate which comprises of both fine and coarse aggregate. As stated above, grading of aggregate has a considerable influence on the workability and stability of concrete mix and it is an important factor in concrete mix design. BS 812: Part 103 specifies that aggregate particle distribution should conform to one of the grading zones for it to be acceptable in concrete making.

a) Sieve Analysis.

This is a system of dividing a sample of aggregate into various fractions each consisting of particles of the same size in order to establish the particle size distribution of the sample of aggregate (grading). The aggregate fractions from 80mm to 4.75mm are referred to as coarse aggregate and those from 4.75mm to 150 microns are termed fine aggregate. The 4.75mm size is dividing fraction appearing in both coarse and fine aggregate.

The gradation of a sample is assessed by sieving a sample successively through the entire sieve mounted one on top of the other with larger sieve on top. The material retained on each sieve is bigger than it and finer than the one above. Sieving can be done manually or mechanically provided the shaking is done in all direction to give chance to all the particles to pass the sieve. Operation should continue until no particle passes anymore.

The quantity of materials retained on each sieve is weighed and recorded. Also the percentage retained, the percentage passed, cumulative weight retained, cumulative percentage retained and percentage passed are recorded for each

sieve size. A specimen chart can be used to record the result graphically and the curves produced by plotting “nominal aperture size of test sieve against cumulative percentage passing” represent the grading pattern of the aggregate. The fineness modulus can be calculated by finding the sum of cumulative percentage of aggregate retained on each standard sieve between sizes 4.75 mm and 150 micron; and dividing the value by an arbitrary number 100. The coarser the material, the larger the fineness modulus.

The following limits may be taken as a guide to establishing the degree of fineness of an aggregate:

Fine sand : Fineness modulus : 2.2 – 2.6

Medium sand : Fineness modulus : 2.6 – 2.9

Coarse sand : Fineness modulus : 2.9 – 3.3.

Sand having a fineness modulus more than 3.2 will be unsuitable for making satisfactory concrete. The fineness modulus can be used to check the constancy of grading when relatively small change is expected; but it should not be used to compare the grading of aggregates from two different sources. The fineness modulus of fine aggregates is required for mix proportion since sand gradation has the largest effect on workability. Finer sands (low fineness modulus) need more water for good workability

b) Standard Grading Curve

This is a graphic representation of the result of a sieve analysis. This curve makes the understanding of particle size distribution of a material very easy. By using chart it is possible to know at a glance whether the grading of a given sample conform to the specification or is too coarse or too fine or deficient in a particular size.

c) Grading Zones for Fine Aggregates

There have been several approaches to specifying the grading requirement for fine aggregate. The 1973 edition of BS 882 introduced four grading zones for

fine aggregate. The division being based primarily on the percentage passing the $600\mu m$ sieve size.

A grading zone is more easily explained when set down on logarithmic graph paper. It is the area contained between a line drawn through the maximum amounts permissible to pass any particular specified sieve, and a line drawn through the minimum amounts permissible to pass the same specified sieves. The area contained between these two lines is known as the "grading zone". When plotting an actual grading result, provided the "plot" remains within the zone/envelope the aggregate tested is within specification. If the line of the "plot" leaves the grading zone the aggregate sample is out of specification. Recording the results of tested materials on a graphical basis makes it far easier to assess the quality of a material than looking at a string of numbers, and you are able to tell at a glance whether a material is well graded or gap graded. But results presented graphically are not easy to store on databases. The grading limits for the zones (I, II, III and IV) are shown in Table 2.2 below.

Table 2.2: Grading Limits for Fine Aggregate

Sieve Size	Percentage passing			
	Zone I	Zone II	Zone III	Zone IV
10mm	100	100	100	100
4.75mm	90 – 100	90 – 100	90 – 100	95 – 100
2.36mm	60 – 95	75 – 100	85 – 100	95 – 100
1.18mm	30 – 70	55 – 90	75 – 100	90 – 100
* 600μm	16 – 34	35 – 59	60 – 79	80 – 100
300 μ m	5 – 20	8 – 30	12 – 40	15 – 50
150 μ m	0 – 10	0 – 10	0 – 10	0 – 15

Since the values for 600 μ m are not overlapping for different zones, it is used for confirming the zone for fine aggregate. Zone I represents coarse sand and Zone IV represents the finest sand in all the four zones. Fine aggregate that belong to Zone IV should not be used for reinforced concrete work unless test has been conducted for the suitability in the mix proportion.

2.1.1.3 Water

Water is needed for the hydration of cement in concrete, but not all is used up for this purpose. Part of this added water is to provide workability during mixing and for placing. During the hydration process, the cement paste glues the aggregate together, fills voids within it, and makes it flow more freely. As a general rule, water of chemical composition acceptable for drinking, whether treated for distribution through the public supply or untreated, is suitable for making concrete (BS3148:1980).

2.1.1.4 Admixtures

Mehta and Monteiro, (2006) defined an admixture as a material other than water, aggregates, hydraulic cements, and fiber reinforcement that is used as an ingredient of concrete or mortar and added to the batch immediately before or during mixing. Admixtures are grouped as chemical admixtures (surfactants) and mineral admixtures.

i. Chemical Admixtures

Chemical admixtures are materials in the form of powder or fluids that are added to the concrete to give it certain characteristics not obtainable with plain concrete mixes. In normal use, admixture dosages are less than 5% by mass of cement and are added to the concrete at the time of batching/mixing. The common types of chemical admixtures are as follows:

ii. Accelerators

Accelerators speed up the hydration (hardening) of the concrete. Typical materials used are Calcium chloride- CaCl_2 , Calcium trioxonitrate II- $\text{Ca}(\text{NO}_3)_2$ and Sodium trioxonitrate- NaNO_3 . However, use of chlorides may cause corrosion in steel reinforcement and is prohibited in some countries, so that nitrates may be favored.

iii. Retarders

Retarders slow the hydration of concrete and are used in large or difficult pours where partial setting before the pour is completely undesirable. Typical polyol retarders are sugar, sucrose, sodium gluconate, glucose, nitric acid, and tartaric acid.

iv. Air-Entrainment Admixture

Air-entrainment admixtures add and entrain tiny air bubbles in the concrete, which reduces damage during freeze-thaw cycles, increasing durability. The

chemical formula of a typical air-entraining admixture consists of a non polar hydrocarbon chain with an anionic polar group. However, entrained air entrains a trade off with strength, as each 1% of air may decrease compressive strength 5%. The mechanisms by which air voids are entrained and stabilized when a surfactant is added to the cement-water system is as follows: At the air-water interface the polar groups are oriented toward the water phase lowering the surface tension, promoting bubble formation, and counteracting the tendency for the dispersed bubbles to coalesce. At the solid-water interface where directive forces exist in the cement surface, the polar groups become bound to the solid with the non polar groups oriented toward the water, making the cement surface hydrophobic so that air can displace water and remain attached to the solid particle as bubbles.

v. Plasticizers

Plasticizers increases the workability of plastic or fresh concrete, allowing it to be placed more easily, with less consolidating effort. A typical plasticizer is lignosulphate. Plasticizers can be used to reduce the water content of a concrete while maintaining workability and are sometimes called water reducers due to its use. Superplasticizers also called high-range water-reducers are a class of plasticizers that have few deleterious effects and can be used to increase workability more than is practical with traditional plasticizers. Compounds used as super-plasticizers include sulfonated naphthalene formaldehyde condensate, sulfonated melamine formaldehyde condensate, acetone formaldehyde condensate and polycarboxylate ethers.

vi. Pigments

Pigments are added to concrete mixtures to change the color of the concrete for aesthetics.

vii. Corrosion Inhibitors

Corrosion inhibitors are used to minimize the corrosion of steel and steel bars in concrete.

viii. Bonding Agents

Bonding agents are used to create bond between old and new concrete (typically a type of polymer) with wide temperature tolerance and corrosion resistance.

ix. Pumping Aids

Pumping aids improve pumpability, thicken the paste and reduce separation and bleeding.

2.1.2 Transition Zone in Concrete

Transition Zone (TZ) represents the interfacial region between the particles of coarse aggregate and the hydrated cement paste. Generally TZ is the weakest link of the chain and is considered the strength limiting phase in concrete (Kamran, 2015).

2.1.2.1 Characteristics of the Transition Zone

- i. Large crystals of Ettringite and CH with preferred orientation.
- ii. Porous structure.
- iii. Transition zone exists on a thin shell, typically 10-15 μ m thick around large aggregates.
- iv. It is generally weaker than either of the two main components of concrete, and it therefore imposes a far greater influence on the mechanical behavior of concrete than is reflected by its size.
- v. In freshly compacted concrete, water film form around large aggregate particles. This accounts for high w/c ratio that exists closer to large aggregates than in the bulk mortar.

2.1.2.2 Significance of Transition Zone

- a. Concrete is brittle in tension, but relatively tough in compression.
- b. α_t (tensile strength) is almost $1/10^{\text{th}}$ of α_c (compressive strength).
- c. At a given w/c ratio, mortar is stronger than the corresponding concrete.
- d. Cement paste and aggregate are elastic, concrete is not.
- e. Coefficient of permeability of mortar is much lower (1/100) than typical concrete of the same w/c ratio.

2.1.2.3 Ways of Increasing the Strength of the Transition Zone

- i. Use of low w/c ratio.
- ii. Use of Silica fume (high surface area).
- iii. Use of different types of aggregate.

2.1.3 Concrete Mix Design

Mix design can be defined as the process of selecting suitable ingredients of concrete and determining their relative quantities with the purpose of producing an economical concrete which has certain properties, notably workability, strength and durability (Neville, 1996)

Basically, the problem of designing a concrete mix consists of selecting the correct proportions of cement, fine and coarse aggregate and water to produce concrete having the specified properties. Sometimes additional ingredients such as chemical admixtures (Plasticizers, Retarders, Air-Entrainers, etc) or pozzolanic admixtures (Ground Granulated Blast-furnace Slag (GGBS), Pulverized-Fuel Ash (PFA), Silica Fume, etc) are used. There are many properties of concrete that can be specified, e.g. workability, strength, density, thermal characteristics, elastic modulus and durability requirements. The properties most usually specified being; a) The workability of the fresh concrete; b) The compressive strength of hardened concrete at a specified age; and c) The durability of the concrete, specified in terms of minimum cement

content and/or the maximum free-water/cement ratio, and in some cases, requiring the specification of types of materials to be used.

2.1.3.1 Basic Concepts of Mix Design by the British Design of Experiment (DOE) Method

i. Strength Margin

Because of the variability of concrete strengths, the mix must be designed to have a considerably higher mean strength than the strength specified. The difference between the specified characteristic strength and the target mean strength is called the '**margin**'. This margin is based on the knowledge of the variability of the concrete strength obtained from previous production data expressed as a standard deviation, or alternatively a substantial margin is applied until an adequate number of site results are obtained.

ii. Free-water Content

Teychenne et al., (1997) stipulated that the total water in a concrete mix consists of the water absorbed by the aggregates to bring them to a '**saturated surface-dry**' condition, and the free-water available for the hydration of the cement and for the workability of the fresh concrete. In practice aggregates are often wet and they contain both absorbed water and free surface water so that the water added at the mixer is less than the free water required. The workability of concrete depends to a large extent on its free-water content; if the same total water content were used with dry aggregates having different absorptions, then the concrete would have different workabilities. Similarly the strength of concrete is better related to the **free-water/cement ratio** since on this basis the strength of the concrete does not depend on the absorption characteristics of the aggregates.

iii. Workability

The workability that is considered desirable depends on two factors. The first of these is the size of the section to be concreted and the amount and spacing of reinforcements; the second is the method of compaction to be used. Therefore if a section is narrow and compacted, the concrete must have higher workability so that full compaction can be achieved with a reasonable amount of effort.

iv. Cement Content

In designing a mix, it is sensible to aim at an economic cement content because cement is more expensive than aggregate. A moderate cement content also confers a technical advantage of a lower cracking potential, in the case of mass concrete and structural concrete. However for durability considerations, cement content has to be at least equal to that laid down by specifications of relevant standards.

v. Choice of Aggregates

There is insufficient difference between the behavior of rounded and irregular aggregates in concrete to justify the use of separate classifications for these two shapes of aggregate, both of which are usually uncrushed and smooth textured aggregates. There are however significant differences between these aggregates and angular aggregates which are usually rough in texture and invariably produced by a crushing process.

Two of the characteristics of aggregate particles that affect the properties of concrete are particle shape and surface texture. Particle shape affects the workability of the concrete, and the surface texture mainly affects the bond between the matrix and the aggregate particles and thus the strength of the concrete. Generally, crushed aggregates consist of rather angular particles having a rough surface texture resulting in a concrete of lower workability but higher strength compared with a similar mix made with uncrushed aggregates. However, in line with the principles of taking only major factors into account in

designing the initial trial mix, only these two types of aggregate are considered, i.e. crushed and uncrushed. The type of aggregate becomes of greater importance for concrete having a high specified strength. If the specified strength at 28 days is 50N/mm^2 or more, it may become necessary to use crushed aggregate rather than uncrushed gravel. The higher the specified strength the more critical is the selection of the source of the aggregate.

vi. Durability

A durable concrete is one which gives a satisfactory performance during an adequate life in a given environment; this includes providing protection to the steel against corrosion in reinforced and prestressed concrete. There are some durability problems associated with the constituent materials and others due to the effect of hostile environments.

A major factor in providing durable concrete is the production of a dense, impermeable concrete, having adequate cement content and low free-water/cement ratio, which is fully compacted and properly cured. To be durable in hostile environments, Codes and Standards may specify the use of particular materials, or limits on the cement content or free-water/cement ratio. Provision is made in the DOE Mix Design Method for these to override the values obtained from strength and workability requirements.

To ensure adequate protection against durability challenges, BS 8110: Part 1: 1985 specifies higher grades of concrete as the severity of the exposure conditions increases; it also specifies minimum cement contents and maximum free-water/cement ratios, depending on the degree of exposure. Corrosion problems are aggravated by the presence of chlorides in either aggregates or admixtures. Limits are specified in BS 882 and BS 8110 and materials complying with these requirements should be used.

Concrete in the ground may be subject to attack by sulfates. Minimizing the effect of such attacks requires the use of Sulfate-resisting Portland cement or

other materials, and the mix proportions should comply with the requirements given. Concrete that is exposed to freezing when wet and to the action of de-icing salts is liable to spalling and deterioration. The resistance of concrete to such deterioration is greatly improved if it contains entrained air as required in BS 5328: 1(1997).

vii. Cost Considerations.

The cost of concreting just as any other type of construction procedure is composed of cost of constituent materials, plant and labor. Cement as a construction material is much costlier than other components, as a result, it is desirable to avoid high cement content during selection of mix proportion. This will make way for the production of economical concrete. Apart from economy, lean mixes bestow many other technical advantages on the concrete such as reduction in the heat generated in mass concrete structures such as dams. In structural concrete, lean mixes help in the reduction of high shrinkage and excessive cracking associated with rich mixes.

While trying to produce an economical concrete, it is important to note that the specified minimum or characteristic strength should not be compromised because that is the basis for acceptance or rejection of the concrete. Other cost implications in concreting come from the quality control, due to the cost of supervision and batching equipment which are not necessary in most cases. Therefore the extent of quality control should depend on the size and type of construction. It is essential that the degree of quality control is estimated at the onset of material selection to minimize the difference between the mean and characteristic strength (Neville, 1996).

Labor is also another source of expenditure and this can be influenced much by workability of the mix. If the mix is stiff, the cost of effort required to compact it will be much or trying to save cost, insufficiently compacted concrete will

result to failure. In all, the effective cost of concreting is controlled by planning of the job and equipment.

2.1.3.2 Characteristic Strength of Concrete

It has become the increasing practice to specify the quality of concrete not as a minimum strength but as a ‘characteristic strength’ below which a specified proportion of the test results, often called ‘defectives’, may be expected to fall (Teychenne et al., 1997) The characteristic strength may be defined to have any proportion of defectives. BS 5328 and BS 8110 adopt the 5% defective level in line with the CEB/FIP international recommendations for the design and construction of concrete structures.

2.1.3.3 Margin for Mix Design

As a result of the variability of concrete in production it is necessary to design the mix to have a mean strength greater than the specified characteristic strength by an amount termed the margin. Thus:

$$f_m = f_c + ks \dots\dots\dots 2.1$$

where f_m = the target mean strength of concrete

f_c = the specified characteristic strength

ks = the margin, which is the product of:

s = the standard deviation, and

k = a constant

The constant k is derived from the mathematics of the normal distribution and increases as the proportion of defectives is decreased, thus:

k for 10% defectives = 1.28

k for 5% defectives = 1.64

k for 2.5% defectives = 1.96

k for 1% defectives = 2.33

Hence, for the 5% defective level specified in BS 5328, $k = 1.64$, for f_c of 30N/mm^2 and a standard deviation of 6.1N/mm^2 target mean strength f_m ,

$$\begin{aligned}f_m &= 30 + (1.64 \times 6.1) \\ &= 30 + 10 \\ &= 40\text{N/mm}^2\end{aligned}$$

2.1.3.4 Factors Governing the Selection of Mix Proportions

i. Durability

The durability of concrete is its resistance to the aggressive environmental conditions. High strength concrete is generally more durable than low strength concrete. In the situations where high strength is not necessary but the conditions of exposure are such that high durability is vital, the durability requirement will determine the water-cement ratio to be used.

ii. Workability.

The degree of workability required depends on three factors. These are the size of the section to be concreted, the amount of reinforcement, and the method of compaction to be used. For the narrow and complicated section with numerous corners or inaccessible parts, the concrete must have a high workability so that full compaction can be achieved with a reasonable amount of effort. This also applies to the embedded steel sections. The desired workability depends on the compacting equipment available at the site condition.

iii. Maximum Size of Aggregate.

Determine the maximum size of coarse aggregate that is economically available and consistent with dimensions of the structure. It is found that the larger the size of aggregate, the smaller is the cement requirement for a particular water cement ratio. Aggregates having a maximum nominal size of 20mm and smaller are generally considered satisfactory.

iv. Grading and Type of Aggregate.

The grading of aggregate influences the mix proportions for a specified workability and water-cement ratio; the coarser the grading, the leaner will be

the mix which can be used, but this is true within certain limits only because a very lean mix that does not contain enough fine material will not produce a cohesive concrete. It is possible to reverse the direction of choice: for instance, if the cement content is fixed, then a grading may be chosen such that concrete of a given water/cement/aggregate proportion and a satisfactory workability can be made. It is worthy to note that there are limits on grading outside which it will be impossible to make a good concrete.

The type of aggregate, its surface texture, shape and properties influence strongly the aggregate-cement ratio for the desired workability and stipulated water cement/ratio. At the onset of selecting a mix proportion, the type of aggregate available and its grading should be known to enhance proper selection.

2.1.4 Properties of Fresh Concrete

2.1.4.1 Workability

Workability is often referred to as the ease with which a concrete can be transported, placed and consolidated without excessive bleeding or segregation. It can also be defined as the internal work required in overcoming the frictional forces between concrete ingredients for full compaction. It is obvious that no single test can evaluate all these factors. In fact, most of these cannot be easily assessed even though some standard tests have been established to evaluate them under specific conditions.

In the case of concrete, consistence is sometimes taken to mean the degree of wetness; within limits, wet concretes are more workable than dry concrete, but concrete of same consistence may vary in workability. Because the strength of concrete is adversely and significantly affected by the presence of voids in the compacted mass, it is vital to achieve a maximum possible density. This requires sufficient workability for virtually full compaction to be possible using a reasonable amount of work under the given conditions. Presence of voids in

concrete reduces the density and greatly reduces the strength: 5% of voids can lower the strength by as much as 30%, and even 2% void can lower strength by as much as 10% (Glanville, et al, 1947).

2.1.4.2 Factors that Affect Workability of Concrete

i. Water Content

If the water-cement ratio is high, the workability will also be high. Since by simply adding water the inter-particle lubrication is increased, high water content results in a higher fluidity and greater workability. Increased water content also results in bleeding. Another effect of increased water content can also be that cement slurry will escape through joints of formwork through bleeding leaving the concrete with insufficient paste, thereby leading to a reduction in the strength of the resulting concrete.

ii. Mix Proportions

An important factor that affects workability is the aggregate/cement ratio. It has been shown that the higher the aggregate/cement ratio, the leaner is the concrete (concrete of low cementitious material content). Lean concrete has low quantity of paste available for providing lubrication, per unit surface area of aggregate and hence the mobility of aggregate is restrained. Conversely, in the case of rich concrete with lower aggregate/cement ratio, there is more paste available to make the mix cohesive and fatty to give better workability.

iii. Size of Aggregate

The bigger the size of aggregate, the lesser is the surface area and therefore, less amount of water required to wet the surface; extra water being available for workability and less cement paste being required for lubricating the aggregate surface to reduce internal friction. Therefore, for a given quantity of water and paste, bigger size of aggregate will give a higher workability.

iv. Shape of Aggregate

Angular, elongated or flaky aggregate makes the concrete very harsh when compared to rounded aggregates or cubical shaped aggregates (Shetty, 2005). This is because cubical and rounded aggregate have less surface area and less void per unit volume of aggregate than flaky and angular aggregate. The frictional resistance also reduces as a result. This is the reason why natural sand or river sand and gravel sand provide more workability than crushed sand aggregate. This factor is very significant when high performance concrete is a priority, in which case a very low water/cement ratio, in the order of about 0.25, is used.

Due to high demand on natural sand, in the near future this aggregate will be very costly or even completely unavailable. This will shift attention to manufactured sand which presently has flaky and angular shapes, this makes it unsuitable for concrete production because it hampers workability but innovations in modern crusher makes it possible to produce a good shaped and well graded manufactured aggregate (Shetty, 2005).

v. Surface Texture of Aggregate

The effect of surface texture of aggregate on workability is due to the fact that the total surface area of rough textured aggregate is more than the surface area of smooth rounded aggregate of same volume. So it is established that rough textured aggregate will show poor workability and smooth or glassy textured aggregate will give better workability.

vi. Grading of Aggregate

Grading of aggregate is one of the factors that will have maximum effect on workability. When aggregates are properly graded such that they have least amount of voids in a given volume, workability is improved. When other factors are constant, the total voids are less; excess paste is available to give better lubricating effect. With excess amount of paste available, the mixture becomes

cohesive and fatty which prevents segregation of particles. With minimal amount of compacting efforts, aggregate particles will slide past each other. The better the grading, the less is the void content and higher the workability.

vii. Use of Admixture

The use of admixtures is the most important factor amongst the above listed factors that affect workability. It has been reported that Plasticizers and Superplasticizers greatly improve the workability many times as much as concrete that contain no such admixtures. Some other agents such as air-entraining agents provide ball bearing action between particles of aggregate thereby reducing the internal friction and in turn providing greater mobility.

viii. Effect of Time and Temperature

The change in workability with time depends on the moisture condition of aggregate (at given total water content); the loss is greater with dry aggregate due to the absorption of water by aggregate. It is also affected by the ambient temperature. This is true because on a hot day the water content of the mix would have to be increased for a constant early workability to be maintained.

2.1.6 Properties of Hardened Concrete

2.1.6.1 Strength of Concrete

2.1.5.1.1 Compressive Strength

The compressive strength of concrete is the most common performance measure used by the engineer in designing buildings and other structures. The compressive strength is measured by breaking concrete specimens (cubic or cylindrical) in a compression-testing machine. Strength of hardened concrete is its most important property. However, for aggressive exposure conditions, durability, volume stability, and impermeability may be equally important. The general assumption is that an improvement in concrete strength will improve its other properties as well, except many important exceptions; for example, an

increase in cement content that may be intended to increase the strength may also increase the amounts of shrinkage and creep.

i. Importance of Compressive Strength

- a. Compressive strength of concrete is its indexing property as concrete has its maximum strength against compression.
- b. In reinforced concrete structures, concrete is mainly used to resist the Compression.
- c. Concrete is classified into different grades on the basis of its 28-days compressive strength.
- d. The 28-days compressive strength requirement forms the basic and main background for designing the concrete mixes.

2.1.5.1.2 Tensile Strength

Tensile strength is the resistance of a material to breaking under tension. Concrete is designed mainly for compressive strength but the tensile strength is as well considered even though concrete is not normally designed to resist direct tension. The knowledge of tensile strength is used to estimate the load under which cracking will develop. The cracking is a form of tension failure.

Tensile strength of concrete is essential to evaluate the load at which the concrete structure may crack. The knowledge gotten is useful in the design of pavement slabs and airfield runway (Gambhir, 1993). Tensile strength is used in both serviceability and ultimate limit state calculations for example: the evaluation of the cracking moment for prestressed elements, the design of fibre-reinforced concrete, developing moment curvature diagrams and in the calculation of deflection. The tensile strength of concrete is also important in the design of concrete roads and railway. For example, flexural strength or modulus of rupture (tensile strength in bending) is used for distributing load over a wide area of road pavement or railway track. The method used in assessing the tensile strength of concrete is by the split cylinder test and this involves diametrically

loading a cylinder along its entire length. The magnitude of the tensile strength at failure f_{ct} is given as

$$f_{ct} = \frac{2F}{\pi ld} \dots\dots\dots 2.2$$

Where F is the maximum applied load, l and d is cylinder length and diameter respectively. Strength in tension is of interest also in unreinforced concrete structures, such as dams, under earthquake conditions. Split tensile strength is used in the design of structural lightweight concrete members to evaluate the shear resistance provided by concrete and to determine the development length of reinforcement.

i. Factors Influencing Tensile Strength

- a. Compressive strength: in general the tensile strength of concrete varies in proportion to the compressive strength.
- b. Coarse aggregate type: concrete containing high-quality crushed rock coarse aggregate tends to have higher tensile strength than concrete made with gravels. However, crushed flint gravels in particular may result in a low tensile strength due to poor bond with the glassy flint surfaces.
- c. Aggregate size: The tensile strength of concrete tends to be higher when using smaller aggregate due to the increase in aggregate surface area and hence reduction in aggregate-cement paste bond stress.
- d. The relative volumes of cement paste and aggregate have little effect on tensile strength of concrete.

2.1.6 Characteristics of Concrete Strength

The only engineering property of concrete that is routinely specified is the characteristic compressive strength. This has a relationship to most other mechanical properties and provides the basis for estimating them. When designing any form of reinforced concrete structure the designer will specify the strength of concrete that has been assumed in the design. This strength of

concrete is usually specified in terms of the characteristic strength. This characteristic strength is based on statistical concepts and is the strength below which no more than 5% of all cubes tested from the chosen concrete mix will fall. Equally it can be expected that 95% of all cube samples will have strengths in excess of the design characteristic strength.

2.1.7 Curing of Concrete

This is the name given to procedures used for promoting the hydration of cement, and consists of a control of temperature and of the moisture movement from and into the concrete. The effect of inadequate curing on strength is greater at higher water/cement ratio and is also greater in concretes with a lower rate of development of strength. Ben-Bassat et al., (1990) reported that the strength of concretes made with Ordinary Portland Cement is more affected by poor curing. Likewise concretes containing fly ash or ground granulated blast furnace slag are more affected than concretes made with Portland cement only. Curing of concrete can be done in two broad ways, viz; Water curing and Membrane curing.

i. Water Curing

It is the best known method of curing concrete samples because it satisfies all the requirements of curing, which are; promotion of hydration, elimination of shrinkage and absorption of the heat of hydration. This method can be achieved in the following ways, (i) Immersion, (ii) Ponding, (iii) Spraying or Fogging and (iv) Wet covering.

Precast concrete units are normally immersed in curing tanks for certain durations. Pavement slabs, roof slab, etc., are covered under water by making small ponds. Vertical retaining wall, plastered surfaces or concrete columns are cured by spraying water on them. Wet coverings such as wet gunny bags, hessian cloth, jute matting, straw, etc., can also be wrapped on vertical surfaces for keeping the concrete wet in some cases. Horizontal surfaces can be covered

with saw dust, earth or sand, and subsequently kept wet to achieve curing (Shetty, 2005).

ii. Membrane Curing

This second method of curing relies on the prevention of loss of water from the surface of the concrete, without the possibility of external water ingress into it. The method can as well be called water-barrier method. It involves the techniques of covering the surface of the concrete with overlapping polyethylene sheeting, laid flat, or with reinforced paper. The sheeting can appear either in black or white where the black is preferred in cold weather and the white in hot weather. The white has the advantage of reflecting solar radiation.

2.1.8 Applications of Concrete

Arora, (2008) Concrete is used for the construction of bored piles.

Precast piles are constructed of concrete (reinforced or pre-stressed) (Onyenuga, 2009).

Ettu, (2003) Reinforced concrete can be used as a material for constructing pressure filters.

Concrete is also used for the construction of gravity dams, etc.

2.2 Pozzolana

Pozzolana is a siliceous or aluminous material, which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide Ca(OH)_2 to form compounds possessing hydraulic cementitious properties.

Mehta and Monteiro, 2006 defined Pozzolan as a siliceous or siliceous and aluminous material, which in itself possesses little or no cementing property but will, in a finely divided form and in the presence of moisture, chemically react

with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties.

Pozzolans are materials containing reactive silica/or alumina which on their own have little or no binding property, but when mixed with lime in the presence of water, will set and harden like a cement.

2.2.1 Historical Use of Pozzolana

The word pozzolana was derived from the name of the town Pozzuoli, Italy where the Romans mined the ashes deposited by the occasional eruption of volcano at Vesuvius. They added these ashes to lime putty to construct their sturdy building. It was the Romans that eventually fully developed the potential of lime pozzolana paste as binder phase in roman concrete used for building and underwater construction.

2.2.2 Geochemistry, Mineralogy and Tpyes of Pozzolana

The chemical composition of pozzolana is variable SiO_2 being the major unaltered pumices. At the basis of the *pozzolanic reaction* stands a simple acid base reaction between calcium hydroxide also known as portlandit or $[\text{Ca}(\text{OH})_2]$ and silicic acid $[\text{H}_4\text{SiO}_4$ or $\text{Si}(\text{OH})_2]$ and can be schematically represented as follows $\text{Ca}(\text{OH})_2 + \text{H}_4\text{SiO}_4 \rightarrow \text{Ca}^{2+} + \text{H}_2\text{SiO}_4^{2-} + 2\text{H}_2\text{O} \rightarrow \text{CaH}_2\text{SiO}_4 \cdot 2\text{H}_2\text{O}$. It can be summarized in abbreviation notation of cement chemists as $\text{CH} + \text{SH} \rightarrow \text{CSH}$, were CSH is calcium silicate hydrate. The reaction that occurs between pozzolanas, water and calcium hydroxide is called *pozzolanic reaction*. This reaction can be represented as

Pozzolana + calcium Hydroxide + water \rightarrow C-S-H Gel (Shetty, 2005).

The underlying principle is that the pozzolanic activity consists of fixing of $\text{Ca}(\text{OH})_2$ by the Pozzolana so that the lower the resulting quantity of $\text{Ca}(\text{OH})_2$ the higher the pozzolanicity (Nevile, 1996). Studies by Chandrasekhar et al. (2003) suggest that soil chemistry as well as climatic and geographic

conditions could affect the physical and chemical properties and consequently the pozzolanicity of agricultural and plant by-products.

Some industrial sources of materials with pozzolanic properties are: class f (siliceous) fly ash from coal-fired power plants, silica fume from silicon production etc. Some agricultural pozzolans are: Rice husk ash, Sawdust ash, coconut ash, corncob ash, palm oil bunch ash, waste ash.

2.2.3 Rice Husk Ash (RHA)

Rice Husks are the hard protecting coverings/ coatings for the rice grain which is made of hard materials including silica and lignin (Agarwal, 2000). Rafat and Mohammad, (2011) wrote that “Rice husk constitutes about 20% of the weight of rice.

Available statistics reveals that about one tone of husks is produced from five tons of rice paddy and it has also been estimated that some 120millions tones of husks could be available annually on a global basis for pozzolana production. (Akindehinde and Oluwotosin, 2010).

2.2.3.1 Processing And Production of RHA

When rice husk is burnt rice husk ash (RHA) is generated. On burning, cellulose and lignin are removed leaving behind silica ash. 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 form of silica obtained after combustion of rice husk depends on the temperature and duration of combustion of rice husk. Using X-ray diffraction, Chopra et al. (1981) observed that at burning temperatures up to 700⁰C, the silica was in an amorphous form. However, Yeoh et al (1979) reported that RHA can remain in the amorphous form at combustion temperatures of up to 900⁰C if the combustion time is less than 1h, while crystalline silica is produced at 1000⁰C with combustion time greater than 5 minutes. The effect of different burning temperatures and the chemical composition of rice husk (Taiwan RHA)

were studied by Hwang and Wu (1989). It was observed that at 400⁰C, polysaccharides begin to depolymerize. Above 400⁰C, dehydration of sugar units occurs. At 700⁰C, the sugar units decompose. At temperatures above 700⁰C, unsaturated products react together and form a highly reactive carbonic residue. At burning temperatures below 800⁰C an ash rich in amorphous silica is formed which is highly reactive. Temperatures above 800⁰C produce crystalline silica which is far less reactive, Cizer et al. (2006). Della et al. (2002) reported that a 95% silica powder could be produced after heat treatment at 700⁰C for 6 hours.

2.2.3.2 Physical Properties of RHA

Rice husk ash (RHA) is a very fine material. The average particle size of RHA ranges from 5 to 10 μ m. Specific gravity and bulk density of rice husk ash are 2.14 and 0.781g/cc (Satish et al., 2013). Physical properties as reported by few other authors are given in Table 2.3 below.

Table 2.3: Physical properties of RHA

Property	Value			
	Mehta (1992)	Zhang et al. (1996)	Feng et al. (2004)	Bui et al. (2005)
Mean particle size (μ m)	-	-	7.4	5
Specific gravity	2.06	2.06	2.10	2.10
Fineness: passing 45 μ m (%)	99	99	-	-

2.2.3.3 Chemical Composition of RHA

Rice husk ash is very rich in silica content. Silica content in RHA is generally more than 80% - 85%. For RHA to be used as pozzolan in cement and concrete, it should satisfy requirements for chemical composition of pozzolans as per ASTM C618. The combined proportion of silicon dioxide (SiO₂), aluminium oxide (Al₂O₃) and iron oxide (Fe₂O₃) in the ash should not be less than 70%,

and LOI should not exceed 12% as stipulated in ASTM requirement. Chemical composition as reported by few authors is given in Table 2.4 below.

Table 2.4: Oxide composition of RHA

Constituents	Percentage		
	Mehta (1992)	Zhang et al. (1996)	Bui et al. (2005)
Silica (SiO ₂)	87.2	87.3	86.98
Alumina (Al ₂ O ₃)	0.15	0.15	0.84
Iron Oxide (Fe ₂ O ₃)	0.16	0.16	0.73
Calcium Oxide (CaO)	0.55	0.55	1.40
Magnesium Oxide (MgO)	0.35	0.35	0.57
Sodium Oxide (Na ₂ O)	1.12	1.12	0.11
Potassium Oxide (K ₂ O)	3.68	3.68	2.46
Sulfur Oxide (SO ₃)	0.24	0.24	-
LOI	8.55	8.55	5.14

2.2.3.4 Pozzolanic Activity of RHA

Zhang et al. (1996) studied the effect of incorporation of RHA on the hydration, microstructure and interfacial zone between the aggregate and paste. Based on the investigation, they concluded that: (i) calcium hydroxide [Ca(OH)₂] and calcium silicate hydrates [C-S-H] were the major hydration and reaction products in the RHA paste. Because of the pozzolanic reaction, the paste incorporating RHA had lower Ca(OH)₂ content than the control Portland cement paste; and (ii) incorporation of RHA in concrete reduced the porosity and the Ca(OH)₂ amount in the interfacial zone; the width of the interfacial zone between the aggregate and the cement paste compared with the control Portland cement composite was also reduced. However, the porosity of the rice-husk ash composite in the interfacial zone was higher than that of the silica fume composites.

Yu et al., (1999) reported that improvement of concrete properties up on addition of RHA may be attributed to the formation of more C-S-H gel and less

portlandite in concrete due to the reaction between RHA and the Ca^+ , OH^- ions or $\text{Ca}(\text{OH})_2$. Jauberthie et al., (2000) also reported that concentration of silica is high on the external face of the husk, much weaker on the internal face and practically non-existent within the husk. Agarwal (2006) studied the accelerated pozzolanic activity of various siliceous materials, like rice husk ash, silica fume, fly ash, quartz, precipitated silica, and metakaolin. The compressive strength of accelerated tests was compared with cubes cured in water at 7 and 28 days. Maximum activity was observed in case of RHA (\square 45 μm), followed by quartz and silica fume.

2.2.3.5 Fresh Properties of Concrete Containing RHA

i Workability

Ikpong and Okpala, (1992) studied the variation in workability of concrete of designed strengths with the incorporation of RHA. Cement was partially replaced with 0, 20, 25, and 30% of RHA. 28 day designed strength of concretes were 20, 25, 30, and 40 N/mm^2 . Based on their results, they concluded that: (i) to attain the same level of workability, the mixes containing rice husk ash required higher water content than those containing only Ordinary Portland cement as the binder. This was reflected in the water-cement ratios of the three mixes (0%, 30 and 40% RHA contents) for each design strength, which have been calculated on the assumption of a constant cement content by weight (using the cement content of the mix containing 0% RHA); and (ii) with minor variations, this higher water content of mixes containing RHA when compared with the conventional mixes were found to increase with the design strength of the mixes. For the mixes having 30% RHA content the percentage increases were 3.2, 4.4, 8.5 and 5.6 for the design strengths of 20, 25, 30, and 40 N/mm^2 respectively. And for the mixes having 40% RHA content the percentage increases, were 6.3, 8.7, 10.2 and 13.0 for the corresponding design strengths. Ismail and Waliuddin, (1996) reported the effect of rice husk ash (RHA)

passing #200 and #325 sieves as 10–30% partial replacement of cement on the workability of high-strength concrete (HSC). Control mixture had proportion of 1:1.07:1.90 with cement content of 571 kg/m³. Slump and density results are given in Table 2.5 below.

Table 2.5 Fresh concrete properties (Ismail and Waliuddin, 1996)

Mix type	RHA (%)		w/(C+RHA)	Slump (mm)	Density (kg/m ³)
	#200	#325			
A	-	-	0.24	70	2425
Aa-10	10	-	0.31	30	2405
Aa-20	20	-	0.33	60	2400
Aa-30	30	-	0.36	30	2398
Ab-10	-	10	0.30	30	2403
Ab-20	-	20	0.32	45	2396
Abs-30	-	30	0.34	32	2390

It can be seen that slump and density values were reduced with the increase in RHA content. However, fineness of RHA did not have significant effect on both of these properties. Bui et al., (2005) investigated the influence of rice husk ash on the slump of concrete mixtures. Two types (PC30 and PC40) of Ordinary Portland cement were used. Cement PC 30 and PC 40 had Blain specific surface area of 2700 and 3759 cm²/g, respectively. Three water-to-binder ratios (0.30, 0.32, and 0.34) were used. Rice husk ash was used to replace 10%, 15% and 20% by mass of PC. The superplasticizer was added to all mixtures for obtaining high workability. In the mixtures with water to binder ratio of 0.34, the amount of superplasticizer was kept constant to investigate the influence of RHA on workability. They concluded that slump decreased with the increase in RHA content for same level of superplasticizer.

ii Air-Entrainment

Zhang and Malhotra (1996) studied the effect of RHA on the AEA requirement

of concrete mixtures. They used RHA as partial replacement of cement in varying percentages (0, 5, 8, 10 and 15%). They concluded that AEA requirement increased with the increase in RHA content possibly because of high specific surface area of RHA in comparison to cement.

iii Consistency and Setting Times

Singh et al., (2002) examined the effect of lignosulfonate (LS) and CaCl_2 on the setting times, consistence and pozzolanic reaction of blended cement made with RHA. Pastes of 10% RHA-blended Portland cement were made with the addition of 2% CaCl_2 , 1% LS, and (1% LS+ 2% CaCl_2). Both the initial and final setting times were enhanced considerably in the presence of 1% LS indicating that it prolonged the setting behavior. It showed that 1% LS acted as a retarder. The 2% CaCl_2 , on the other hand, reduced both initial and final setting times showing that it acted as an accelerator in the hydration of RHA-blended OPC. However, in the presence of a mixture of 1% LS and 2% CaCl_2 , the setting time values were higher than that in the presence of 2% CaCl_2 and lower than that in the presence of 1% LS. The effect of admixtures on the consistency of 10% RHA-blended OPC is given in Table 2.6. From the table, it is clear that in the presence of 1% LS, the water/binder ratio was reduced considerably as is generally expected in the presence of a superplasticizer. In the presence of 2% CaCl_2 (an accelerator), the water/binder ratio was lower than that of control but higher than that in the presence of 1% LS. However, in the presence of a mixture of 1% LS and 2% CaCl_2 , the water/binder ratio was the lowest. This indicated that water reduction was enhanced in the presence of a mixture of a superplasticizer (1% LS) and an accelerator (2% CaCl_2).

Table 2.6 Variation of water consistency with admixtures (Singh et al., 2002)

Composition	Water-binder
90% OPC +10% RHA	0.42
90% OPC +10% RHA + 2% CaCl ₂	0.40
90% OPC +10% RHA + 1% LS	0.38
90% OPC +10% RHA + 2% CaCl ₂ + 1% LS	0.37

Jaturapitakkul and Roongreung, (2003) studied the normal consistency and setting times of cementing material made from rice husk ash (RHA) and calcium carbide residue (CCR). Calcium Carbide Residue (CCR) is a by-product of acetylene production process. CCR consists mainly of calcium hydroxide, Ca(OH)₂, and is obtained in a slurry form. Chemical composition of CCR was 51.94% CaO, 3.36% SiO₂, 2.56% Al₂O₃, 0.33% Fe₂O₃, 0.46% MgO, 0.03% K₂O, 0.22% SO₃ and 41.72% LOI. The major compound in RHA was SiO₂ (78.22%). The normal consistency of cement paste was 23.9% while those of CCR-RHA pastes were between 43.7% and 62.0%, depending on RHA content. Paste 80C20R (80% of CCR and 20% of RHA) had normal consistency of 43.7% and increased to 62.0% in paste 20C80R (20% of CCR and 80% of RHA). The normal consistency of CCR-RHA pastes needed more water than that of cement paste because of the high porosity of the two materials as well as the high LOI of CCR. The higher the RHA content in the paste, the higher was the water requirement to maintain the same normal consistency. Initial and final setting times of the cement paste were 107 and 195 min, respectively. For CCR-RHA pastes, the shortest setting time occurred in paste 50C50R and the longest setting time was paste 80C20R. The initial and final setting times of paste 50C50R were 345 and 635 min, respectively, while those of paste 80C20R, the longest setting times, were 502 and 680 min, respectively. The initial setting time of the new cementing material was about 3.2–4.6 times longer than that of the cement paste. The results suggested that the ratio of CCR to RHA of 50–50% by weight was the suitable ratio to obtain the highest pozzolanic reaction.

In CCR-RHA paste, the $\text{Ca}(\text{OH})_2$ from CCR reacted with SiO_2 from RHA to form CSH while Portland cement reacted with water and formed CSH directly. The long setting times of calcium carbide residue-rice husk ash paste was because the pozzolanic reaction between pozzolan and lime was usually much slower than the hydration of cement.

2.2.3.6 Properties of Hardened Concrete Containing RHA

i Drying Shrinkage

Zhang and Malhotra (1996) studied the drying shrinkage strain of concretes made with 10% RHA and 10% silica fume (SF). Tests were conducted up the age of 448 days. The results indicated that RHA concrete had a drying shrinkage of 638×10^{-6} after 448 days, which was similar to the strains for the control and silica fume concretes.

ii Porosity and Water Absorption Capacity

Saraswathy and Song (2007) investigated the effect of partial replacement of cement with rice husk ash (RHA) on the porosity and water absorption of concrete. Cement was replaced with 0, 5, 10, 15, 20, 25, and 30% RHA. Proportion of control (without RHA) mix was 1:1.5:3 with w/c ratio of 0.53. Porosity and water absorption test was carried out as per ASTM C642-97. For determination of effective porosity and coefficient of water absorption, specimens of size 83 mm diameter and 50 mm thick were cast with and without rice husk ash and cured for 28 days in distilled water. After the curing period was over the specimens were dried in an oven at $105 \pm 5^\circ\text{C}$ for 48 h in order to evaporate the moisture content present in the concrete. Table 2.7 shows the results of porosity and coefficient of water absorption.

Table 2.7 Porosity and Water absorption of rice husk ash replaced concrete (Saraswathy and Song, 2007)

RHA (%)	Porosity (%)	Water absorption Coefficients (m ² /s)
0	18.06	3.5571 x 10 ⁻¹⁰
5	18.18	6.7587 x 10 ⁻¹¹
10	13.82	1.0320 x 10 ⁻¹¹
15	13.80	1.0644 x 10 ⁻¹¹
20	13.54	1.2122 x 10 ⁻¹⁰
25	13.04	1.4548 x 10 ⁻¹⁰
30	11.89	1.3030 x 10 ⁻¹⁰

Based on the results, they concluded that: (i) porosity values decreased with the increase in RHA content because small RHA particles improved the particle packing density of the blended cement, leading to a reduced volume of larger pores; and (ii) coefficient of water absorption for rice husk ash replaced concrete at all replacement levels was found to be less when compared to control concrete.

iii Compressive and Tensile Properties

Ikpong and Okpala (1992) studied the variation in strength of medium workability concrete with the incorporation of RHA. Cement was partially replaced with 0, 20, 25 and 30% of RHA. 28 day designed strength of concretes were 20, 25, 30 and 40 N/mm². Compressive strength were tested up the age of 90 days. The compressive strength continued to increase with age for each of the mixes. The control mix (0% RHA) attained a higher strength than the OPC/RHA mixes at all age. At the age of 28 days all the OPC/RHA concretes, except the designed mix of 40 N/mm², had attained their 28 day design strengths. For the designed mix of 40 N/mm², the mix having 30% RHA content reached 98.5% of its design strength, while the one having 40% RHA content reached 86.5%. Furthermore even at the early age of 7 days most of the

OPC/RHA concretes had attained over 60% of their 28-day design strengths. The worst case which was the designed mix of 40 N/mm² containing 40% RHA, attained 46% of its 28-day design strength.

Zhang and Malhotra (1996) investigated the influence of 10% RHA inclusion as partial replacement of cement on the compressive strength of concrete and compared it with the compressive strength of concrete containing 10% silica fume (SF). Water-to-cementitious material ratio was maintained at 0.40. They concluded that RHA concrete, in general, achieved higher strength than control concrete mixture but lower than that of silica fume concrete. At 28 days, the RHA concrete had a compressive strength of 38.6 MPa compared with 36.4 MPa for the control concrete and 44.4 MPa for SF concrete. At 180 days, RHA concrete exhibited compressive strength of 48.3 MPa compared with 44.2 MPa for the control concrete and 50.2 MPa for SF concrete. Ismail and Waliuddin (1996) examined the effect of rice husk ash (RHA) passing #200 and #325 sieves as 10–30% partial replacement of cement on the compressive strength of high-strength concrete (HSC). Control mixture had mixture proportion of 1:1.07:1.90 with cement content of 571 kg/m³. Strength tests were conducted up to the age of 150 days, and results are given in Table 2.8. Based on the test results, they concluded that: (i) it is possible to get high strength concrete economically using RHA; (ii) optimum replacement of cement by RHA was around 10% to 20% with finely ground RHA; (iii) rate of hydration in concrete made with part replacement of cement by RHA was slow as compared to concrete with OPC. This fact was very dominant during the initial three days of age of concrete. This rate of slow hydration also affects the 150 day strength of concrete made by part replacement of cement by RHA in the mix; and (iv) the lower strength of concrete made with part replacement of cement by RHA was because of higher w/c ratios. Though the w/c ratio for samples Ab-10 and Ab-20 was 23 and 28% higher than w/c ratio in specimen-A, the 28 day strength was 98% and 97% of the concrete of Mix-A where no replacement was made.

Table 2.8 Compressive Strength of RHA concrete (Ismail and Waliuddin, 1996)

Mix type	RHA (%)		w/(C+RHA)	Compressive strength (MPa)			
	#200	#325		3-day	7-day	28-day	150-day
A	-	-	0.24	54.3	62.3	72.4	85.0
Aa-10	10	-	0.31	46.2	56.0	68.1	71.1
Aa-20	20	-	0.33	35.3	46.8	57.3	57.4
Aa-30	30	-	0.36	31.5	39.3	47.7	48.8
Ab-10	-	10	0.30	47.0	61.0	71.0	72.4
Ab-20	-	20	0.32	46.7	56.0	70.2	70.3
Ab-30	-	30	0.34	43.1	51.7	63.0	63.2s

Zhang and Malhotra (1996) studied the mechanical properties of concrete made with 10% RHA and 10% silica fume (SF). Tests were conducted at the age of 28 days, The results indicated that splitting tensile strength, flexural strength and modulus of elasticity of control and concrete incorporating RHA and SF were comparable. Wada et al. (1999) demonstrated that RHA mortar and concrete exhibited higher compressive strength than the control mortar and concrete. They have further reported excellent strength development at the early stages even without steam curing for RHA mortar and concrete.

Jaturapitakkul and Roongreung (2003) determined the compressive strength of mortar made with a cementing material (combination of rice husk ash and calcium carbide residue). Calcium Carbide Residue (CCR) is a by-product of acetylene production process. CCR consists mainly of calcium hydroxide, Ca(OH)_2 , and is obtained in a slurry form. Chemical composition of CCR was 51.94% CaO , 3.36% SiO_2 , 2.56% Al_2O_3 , 0.33% Fe_2O_3 , 0.46% MgO , 0.03% K_2O , 0.22% SO_3 and 41.72% LOI. The major compound in RHA was SiO_2 (78.22%). Control mixture was made using Portland cement Type I as cementitious material. The rest of the mortars were made with a mixture of CCR and RHA as cementitious material. The mortar contained 1.0 part of cementitious material to 2.75 parts of river sand by weight. The water

cementitious material ratio was kept at a constant of 0.65 in all mixtures. Superplasticizer was employed in order to maintain the flow of mortar between 110 ± 5 since the mixture of CCR-RHA mortars required more water to achieve the target flow. Compressive strengths of mortars were determined at the age of 1, 3, 7, 14, 28, 60, 90, and 180 days of curing. They concluded that: (i) mortar 50C50R gave the highest compressive strength. Its strength was 0.9, 10.0, 15.6, 18.6, and 19.1 MPa, respectively, at the age of 1, 7, 28, 90, and 180 days. Mortar 50C50R had lower compressive strength than that of the control mortar at the same testing age; (ii) difference of compressive strength of mortar 50C50R and control mortar at an early age was very high and reduced to about two times after 28 days. It is important to mention the mix proportion of the mortar 50C50R did not contain Portland cement and the strength of mortar was solely contributed from the pozzolanic reaction of the two materials; and (iii) mortar 50C50R contained 50% of CCR and 50% of RHA as cementitious material gave the highest compressive strength. Increasing or decreasing of CCR content from 50% of the total cementitious material resulted in lower compressive strength of mortar.

Sakr, (2006) examined the role of RHA inclusion as partial replacement of cement, on the mechanical properties of heavyweight concrete. The aggregates used were special natural heavy weight mineral ores, mainly ilmenite and baryte. They were used as the fine and coarse aggregates for the heavy weight concrete. Gravel and sand were used as the coarse and fine aggregate for the gravel concrete. The RHA had a specific surface area of $5.6 \times 10^6 \text{ mm}^2/\text{g}$, and the unit weight was $2.06 \times 10^3 \text{ kg/m}^3$. The chemical composition of the RHA was 87.0% SiO_2 , 1.75% Al_2O_3 , 2.5% Fe_2O_3 , 2.5% CaO , 2.3% MgO , and 2.5% K_2O . Percentage replacement of RHA was 0, 5, 10, 15, and 20% by weight of cement. Tests were conducted for compressive strength, indirect tensile strength, flexural strength, bond strength, and modulus of elasticity at the age of 28 days. They concluded that: (i) with the increase in RHA content up to 15%,

generally, all the mechanical properties increased. Beyond 15%, decrease in strength properties values was observed. This may be due to the high specific surface areas of RHA, which consume more water to have the same workability; (ii) ilmenite concrete had the highest compressive strength when compared to the corresponding barite concrete or gravel concrete. This increase may be attributed to the shape, specific surface area, and hardness of the ilmenite aggregates. It may also be due to the effect of titanium oxide (TiO_2) in ilmenite, which may be combined with water to form a gelatinous material that fills the voids in the cement matrix, consequently increasing bonding between the cement particles and thus increasing its compressive strength; and (iii) 15% of RHA mixed with barite concrete showed a higher tensile strength when compared to gravel concrete but a lower tensile strength than ilmenite concrete mixed with 15% of RHA. The ilmenite concrete exhibited the highest flexural strength, bond strength, and static modulus of elasticity in compression as compared to the baryte or gravel concrete.

Abalaka, (2012) showed that the split tensile strength of concrete cylinders containing 5% and 10% RHA using a charcoal fired incinerator were higher than that of the control at 28days. Also the tensile strength of concrete cylinders containing 5% and 10% RHA were greater than the control at 28days. Saraswathy and Song, (2007) investigated the effect of rice husk ash (RHA) on the compressive, splitting tensile and bond strength of concrete. They concluded that the inclusion of RHA (up to 25%) as partial replacement of cement did not affect the splitting tensile strength of concrete. However, after 25% replacement level, a slight decrease in split tensile strength was observed. Sakr, (2006) examined effect of rice husk ash on the mechanical properties of heavyweight concrete. He concluded that with the increase in rice husk ash content up to 15%, generally, all the mechanical properties increased including the tensile strength. Sensale, (2006) investigated the influence of rice husk ash (RHA) (10 and 20%) on the compressive and splitting tensile strength of concrete. It was

concluded that the results of splitting tensile strength revealed the significance of the filler and pozzolanic effect on the concrete. Hossain, (2003) stated that these filler materials impart technical advantage to the resulting concrete and also enable larger quantities of cement replacement to be achieved. The results were consistent with the compressive strength development at 28days. Strength increases in concrete containing RHA have also been reported in other investigations (Godwin et al., 2013; Ganesan, 2008; Nehdi et al., 2003; Waswa-Sabuni et al., 2002; De Sensale, 2006).

2.2.3.7 Durability Properties of Concrete Containing RHA

i Permeability

Coutinho, (2003) determined the rapid chloride permeability of concrete with partial replacement of RHA in various percentages (10, 15, and 20%) and when using controlled permeability formwork (CPF). Controlled permeability formwork (or CPF) is one of the few techniques developed recently for directly improving the concrete surface zone. This technique reduces the near-surface water/binder ratio and reduces the sensitivity of concrete to poor site curing. CPF consists of using a textile liner on the usual formwork, allowing air bubbles and surplus water to drain out but retaining the cement particles and so enabling the water–cement ratio of the outer layer to become very low and the concrete to hydrate to a very dense surface skin as the filter makes enough water available at the right time to activate optimum hydration. So CPF enhances durability by providing an outer concrete layer which is richer in cement particles, with a lower water/binder ratio, less porous and so much less permeable than when ordinary formwork is used. Resistance to chloride penetration was assessed with the AASHTO T277-83 test method up to the age of 100 days. Table 2.9 below shows the average of 3 test specimens for each series. It is evident from these results that inclusion of RHA significantly reduced the charge passed. Further more, when CPF was used, it greatly

reduced the permeability of concrete mixtures.

Table 2.9 Rapid chloride permeability of RHA concrete (Coutinho, 2003)

Type of mixture	Average charge passed (coulombs)
Control	2349.3
Control + CPF	1916.3
10% RHA	435.0
10% RHA + CPF	384.7
15% RHA	322.0
15% RHA + CPF	245.0
20% RHA	260.0
20% RHA + CPF	202.0

Gastaldini et al., (2007) studied the influence of chemical activators (K_2SO_4 , Na_2SO_4 , Na_2SiO_3) on the chloride-ion permeability of concrete made with 20% of rice husk ash as partial replacement of cement. Water/binder ratios used were 0.35, 0.50 and 0.65 and binder/aggregate ratios were 1:3.75, 1:5.25 and 1:6.9. Potassium sulfate (K_2SO_4), sodium sulfate (Na_2SO_4) and sodium silicate (Na_2SiO_3) were used as chemical activators in concentrations of 1% by weight of cement. Chloride-ion penetration was measured according to ASTM C1202 (2000) at the age of 28 and 91 days. Based on the results they concluded that: (i) at 28 days, RHA concrete exhibited significant reduction in the total charge passed. This reduction amounted to 42%, 51% and 27% for w/b = 0.35, 0.50 and 0.65, respectively. At 91 days, the same w/b ratios showed reductions of 65%, 68% and 59%; (ii) concrete mixtures with activators showed lower total charge passed values when compared with the mixture without activator. The mixtures activated with K_2SO_4 showed the best results. At 28 days, the mixture activated with Na_2SiO_3 showed the lowest charge passed values. Overall, the best results at 91 days were seen in the sample activated with K_2SO_4 ; and (iii) at 91 days, all mixtures with chemical activators showed very low charge passed values (100–1000 C), even for w/b ratios as high as 0.65 which

can be rated as very low as per ASTM C1202.

ii Corrosion Resistance

Saraswathy and Song, (2007) investigated the corrosion performance of concrete made with 0, 5, 10, 15, 20, 25, and 30% RHA as partial replacement of cement. Proportion of control (with out RHA) mix was 1:1.5:3 with w/c ratio of 0.53. Corrosion performance was evaluated by impressed voltage test and open circuit potential measurements. For impressed voltage test, cylindrical concrete specimens of size 50 mm diameter and 100 mm height were cast with centrally embedded rebar of 12 mm diameter and 100 mm height. After 28 days of curing, the specimens were subjected to impressed voltage test. In this technique, the concrete specimens were immersed in 5% NaCl solution and embedded steel in concrete was made anode with respect to an external stainless steel electrode serving as cathode by applying a constant positive potential of 12 V to the system from a DC source. The variation of current was recorded with time. For each specimen, the time taken for initial crack and the corresponding maximum anodic current flow was recorded. For open circuit potential test, specimens of size 100 × 100 × 100 mm were cast. 12 mm diameter rebar of 120 mm length were embedded at a cover of 25 mm from one side of the specimen. After casting the specimens were subjected to water curing for 28 days. After 28 days of curing the cubes were taken out and dried for 24 h and subjected to alternate wetting and drying in 3% NaCl solution. One cycle consisted of seven days immersion in 3% NaCl solution and seven days drying in open atmosphere. The tests were continued over a period of 200 days. Open circuit potential measurements were monitored with reference to saturated calomel electrode (SCE) periodically with time as per ASTM C876. Impressed voltage results were expressed in terms of time to cracking in hours. Time of cracking were 42, 72, and 74 hours for concretes made with 0, 5, and 10% RHA. However, no cracking was observed for concretes with 15, 20, 25, and 30%

RHA ever after 144 hours of exposure. It can be observed from the results that there was no in concretes made with 15, 20, 25 and 30% rise husk even after 144 h of exposure. Whereas in ordinary Portland cement concrete, the specimen was cracked even after 42 h of exposure in 5% NaCl solution. The concrete specimens containing 5 and 10% rise husk also failed within 72 and 74 hours of exposure. This indicated that the replacement of rice husk ash refined the pores and thereby the permeability and corrosion gets reduced.

Open circuit potential measurements values (OCP) as per ASTM C876-97 were lesser than -275 mV vs. saturated calomel electrode (SCE), was considered to be passive in condition. They concluded that all the rice husk ash replaced concretes had shown less negative potential than -275 mV even up to 100 days of exposure indicating the passive condition of the reinforcement bars. Beyond 100 days of exposure all the systems showed a more negative potential than -275 mV vs SCE irrespective of the replacement ratio showing the active condition of reinforcement bars.

iii Carbonation

Gastaldini et al., (2007) examined the role of chemical activators on the carbonation of concrete containing 20% of rice husk ash as partial replacement of cement. Water/binder ratios used were 0.35, 0.50 and 0.65 and binder/aggregate ratios were 1:3.75, 1:5.25 and 1:6.9. Potassium sulfate (K_2SO_4), sodium sulfate (Na_2SO_4) and sodium silicate (Na_2SiO_3) were used as chemical activators in concentrations of 1% by weight of cement. Specimens of size 100×100 mm were cured for 28 days. Their top surface was sealed and they underwent a preconditioning cycle as required by RILEM TC116-PCD (1999). They were then placed in a controlled atmosphere chamber with 5% CO_2 , at $23 \pm 1^\circ C$ and RH $65 \pm 1\%$. The carbon dioxide penetration depth was measured at different exposure times, 4, 8 and 12 weeks by means of the phenolphthalein test carried out on the transversely split section of the cylinders

specimens using the RILEM CPC18 (1988) method. For all mixtures investigated, K_c increased with the increase in w/b ratio because of the increase in concrete porosity and the lower concentration of cement. For the same w/b ratio (0.35, 0.50 and 0.65), the lowest carbonation coefficients were seen in the mixture with RHA and 1% K_2SO_4 showed. The values obtained were lower than those in the reference concrete.

iv Sulfate Resistance

Chindaprasirt et al., (2008) determined the sulfate resistance of mortars made from ordinary Portland cement containing fly ash and ground rice husk ash (RHA). Class F fly ash and RHA were used at replacement dosages of 20 and 40% by weight of cement. The Blaine fineness of PC, FA, and RHA was 2900 cm^2/g , 2600 cm^2/g , and 14,000 cm^2/g , respectively. Mortars were made with sand-to-binder ratio of 2.75 and adjusted water contents to achieve similar flow of $110 \pm 5\%$. The test for sulfate-induced expansion was done following the procedures described in ASTM C1012 with 5% sodium sulfate solution. The expansion of the PC prism was much larger than those made with the blended cements. The accelerating expansion pattern of the PC mortar was observed from 120 days onward. There was no obvious accelerating expansion pattern shown by blended cement mortar prisms. FA20, FA40 and RHA20 mixes show a “linear” pattern of expansion after about 120 days in sulfate solution. RHA40 mix, however, shows a very small expansion even after immersion for 360 days. Figure 7.9 shows the pH levels of the sodium sulfate solution. The highest level of the pH of sulfate solution was approximately 12.5 for all the mortars and was observed within the first 7 days of immersion, indicating that a substantial amount of the calcium hydroxide was leached out and thus increased the pH of the solution. The fresh solution pH was 7.0–7.5. After one day of immersion, the pH level was found to be more than 12. Even Khatri et al., (1997) have reported that the pH of 12–12.5 was obtained within a few hours of immersion.

The pH of the solution increased slightly as the immersion period continued until the solution was replaced by a fresh solution. At 90 and 180 days, the pH levels of the sulfate solutions were significantly lower and different. The pH value of the solution with PC mortar was the highest of 11.0 followed by those of FA20, FA40, RHA20 and RHA40 with 10.7, 10.5, 10.1 and 9.5, respectively. The expansion of the mortar bar is sensitive to the pH level of the solution (Cao et al., 1997). At pH 12–12.5 only ettringite formation can take place and at pH of 8.0–11.5 gypsum formation and decalcification occur (Khatri et al., 1997). Shi and Stegemann, (2000) have reported that dissolution of $\text{Ca}(\text{OH})_2$ and calcium sulfoaluminates, and the decalcification of CSH with a high C/S ratio in hardened PC paste results in a very porous layer whereas the decalcification of the low C/S CSH results in a protective layer of silica gel. For FA and RHA blended cement, C/S ratio of CSH would have been lower as a result of the pozzolanic reaction. FA and RHA mortars thus showed better resistance to the sulfate attack in comparison to PC mortar with RHA, being more reactive and showing better resistance to sulfate attack.

2.2.3.8 Effects of RHA on Other Properties of Concrete

Mehta and Pirtz, (2000) investigated the use of RHA to reduce temperature in high strength mass concrete and concluded that RHA is very effective in reducing the temperature of mass concrete compared to OPC concrete. Nicole et al., (2000) also stated that RHA reduces effects of alkali-silica reactivity (ASR). Lertsatilthanakorn et al., (2004) revealed that RHA reduces heat gain through the walls of buildings. Sata et al., (2007) also revealed that RHA reduces the amount of super plasticizers in concrete. Chindaprasirt et al., (2007) further revealed that RHA reduces potential for efflorescence due to reduced calcium hydracids. Ganesan and Thankvel, (2008); Bhanumathidas and Mehta, (2005); and Feng et al., (2004) in their studies revealed that RHA increases the setting time of pastes and accelerates the early hydration of C_3S . Salas et al., (2009)

argues that RHA in concrete reduces the pore sizes, permeability, and diffusivity of chloride ions. Rafat, (2008) in his work concluded that incorporation of rice husk ash in concrete reduces the porosity and the $\text{Ca}(\text{OH})_2$ amount in the Interfacial Transition Zone.

2.2.4 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. This dust lying as a waste product is sourced in plywood factories in timber market villages and or sites. (Raheem et al., 2012) reports that there is need to convert this waste product into a useful by-product. The dust is usually used as domestic fuel. The resulting ash known as sawdust ash (SDA) is a form of pozzolana. 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 saw dust ratio it is not flammable. As a basic construction material sawdust concrete does indeed has its functions. Sawdust is in abundance in North Eastern India (Meghalaya) and other parts of the world.

2.2.4.1 Effects of Saw Dust Ash on Cement Composites

Elinwa et al., (2008) investigated the suitability of sawdust ash as a pozzolanic material and found that it could be used in binary combination with OPC to improve the properties of cement composites. Elinwa et al., (2005) also found out that sawdust ash can be used in combination with metakaolin as a ternary blend with 3% added to act as an admixture in concrete. Further studies by Elinwa and Abdukadir, (2011) established SDA as reducing porosity as well as being effective in reducing corrosion of reinforcement in concrete. Sumaila and Job, (1999) assessed the properties of OPC-SDA cement concrete at percentage replacement of OPC with SDA of 0 to 30% and reported that all the tested samples developed over 60% of their 28-day strength at 7 days. They recommended the use of SDA to partially replace OPC up to a maximum

percentage replacement of 5-10% by volume. Adeagbo, (1999) investigated the effect of water-cement ratio on the properties of sawdust cement sandcrete and found that 42-57% of sawdust was still adequate in producing sawdust cement sandcrete using suitable water-cement ratio. Udoeyo and Dashibil, (2002) found that SDA concrete with SDA used as partial replacement of OPC attained its 28 day design strength at 10% replacement. Their results indicated that SDA concrete could attain the same strength level as conventional concrete at longer curing ages.

Mageswari and Vidivelli, (2009) investigated the use of sawdust ash as fine aggregate replacement in concrete by replacing sand with 5 to 30% of SDA in making concrete cubes and cylinders and testing for compressive, tensile, and flexural strengths up to 180 days of curing. Their results indicated the similarity in properties of concrete with 100% sand as fine aggregate and those obtained by replacing sand with SDA at 10-20%.

Onwuka et al., (2013) optimized the compressive strength of OPC-SDA cement concrete and obtained 20.44N/mm^2 as the optimum 28-day compressive strength corresponding to 5% replacement of OPC with SDA. Marthong, (2012a) also investigated the strength of mortar cubes, concrete cubes, and beam specimen made with OPC-SDA blended cement and found that the inclusion of SDA caused a little expansion due to low calcium content but early strength development was about 50-60% of their 28-day strength. The study suggested the use of SDA as partial replacement of cement up to a maximum of 10% by volume in all grades of cement. Marthong, (2012b) further investigated the size effect phenomenon of OPC-SDA cement concrete with 10% SDA as partial replacement of OPC using cylinders of different sizes. The results showed that OPC-SDA cement concrete had more size effect than 100% OPC concrete, the size effect being more pronounced at 28 days than at 90 days of hydration.

Recent Studies by Ettu et al. (2013a), Ettu et al. (2013b), Ettu et al. (2013c), and Ettu et al. (2013d) have also 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.

2.2.5 Pulverised Fly Ash (PFA)

Pulverized fly Ash is a residue from the process of combustion in the boilers of coal fired power stations. It is extracted as a fine powder from the fuel gases and hence its other common name fly ash. The ash extracted from the bottom of power station boilers, furnace bottom ash, is less suitable as a pozzolana.

PFA is used to partially replace Portland cement (by up to 60% by mass). Fly ash and Rice husk ash cement mortar are of low PH levels which makes them less susceptible to sulfate attack (Chindaprasirt et al., 2007).

PFA is available, in large quantities, in countries or regions using coal fired electricity generating stations. These include most of Europe, North America, the Indian sub-continent, China and Southern Africa.

There are two types of PFA, depending up on the type of coal used. These are high lime and low lime, with the former having lime content above 10% and therefore possessing some cementing properties on its own. Low lime PFA has a lime content below 10%. Both types of PFA can be used as a pozzolana. In a high volume fly ash concrete mixtures, up to 20% reduction in water requirement can be achieved (Jiang and Malhotra, 2000). The incorporation of pozzolan such as fly ash into concrete have also been investigated by (Poon et al., 1997; Naik and Ramme, 1990; Gopalakrishna et al., 2001),

2.3 Empirical Model

Experimental data are used to derive an empirical model linking the outputs and inputs. The data thus obtained are then processed by the methods of classical

regression or correlation analysis. Nalimov, (1965), Akhnazarova, (1972), Himmelblom, (1970), Drapa, (1966), Nalimov, (1960), Stepanov, (1976).

Applying regression or correlation analysis to experimental data, the designer can establish relationship between various variables and determine the conditions of an optimum. A typical mathematical model is

$$Y = Q(X_1 X_2 \dots \dots \dots X_k) \dots \dots \dots 2.3$$

Where y is the dependent variable or the response and x_k are the independent variables or factors. They occupy what is known as the factor space and graphical representation of the response function is called response surface.

2.4 Regression Analysis

Regression analysis is a process used to estimate a function which predicts value of response variable in terms of values of other independent variables.

Xin and Xiao, (2009). If the regression function is determined only through a set of parameters, the type of regression is called the parametric regression.

Samprit and Ali, (2006), also defined regression analysis as a conceptually simple method for investigating functional relationships among variables. The relationship is expressed in the form of an equation or a model connecting the response or dependent and one or more explanatory or predictor variables.

To perform regression analysis, an investigator often assembles data on underlying variables of interest and employs regression model to estimate the quantitative casual effect of the independent variables to the response variable.

The investigator also typically assesses the statistical significance of the estimated relationship between the independent variables and dependent variable, that is, the degree of confidence on how the true relationship is close to the estimated statistical relationship.

In regression analysis, the dependent variable is modeled as a function of independent variables, corresponding regression parameters (coefficients), and a random error term which represents variation in the dependent variable

unexplained by the function of the dependent variables and coefficients. In linear regression the dependent variable is modeled as a linear function of a set of regression parameters and a random error. The parameters need to be estimated so that the model gives the best fit to the data.

The two main types of regression are the simple and multiple regression. The former has to do with only one predictor variable while the later has to do with two or more predictor variables.

CHAPTER THREE

MATERIALS AND METHODS

3.1 Materials

3.1.1 Cement

A single brand of Ordinary Portland Cement produced by Dangote cement company was used throughout this research work. The OPC has a specific gravity of 3.06 and bulk density of 1660kg/m³.

3.1.2 Pozzolanic Ashes

The Rice Husk used in this project was sourced from Afikpo Local Government Area in Ebonyi State. The Saw Dust used in this project was sourced from Naze timber market in Owerri, Imo State. The characterization of the ashes are presented in chapter 4 Table 4.1.

3.1.3 Aggregates

i Fine Aggregate

The fine aggregate used for this research project was good quality white sharp sand free from debris and organic materials obtained from Ihiagwa bank of Otamiri River in Owerri, Imo state with a specific gravity of 2.64, fineness modulus of 3.35 and bulk density of 1710kg/m³. The specific gravity is lower than the value reported by Ettu et al., (2013) which is 2.80. However, the bulk density and the fineness modulus are higher than the value reported by Ettu et al., (2013) which are 1590kg/m³ and 2.90 respectively. Its coefficient of uniformity, C_u and curvature, C_c are greater than 6 and lies between 1 and 3 respectively which shows that it is well graded according to Arora,(2009).The grading of the sand was carried out to BS812:103 (BS812: Part 1 1975). It was found that the river sand had 62% passing sieve size 600 μ m and hence falls within zone III of the grading limits of fine aggregate according to Shetty, (2008). See Appendix A for Grading Curve of river sand.

ii Coarse Aggregate

Coarse aggregate used for this research project was crushed sandstone rock (Ajali formation) bought from an aggregate depot along Aba-Owerri road before wethral junction Imo State. It has a specific gravity of 2.76, bulk density of 1490kg/m^3 and fineness modulus of 5.34. It falls within the 20mm nominal size grading limit according to Shetty, (2008). See Appendix B for Grading Curve of crushed sandstone.

3.1.4 Water

Clean water used for this study was gotten from a tap in Federal University of Technology Owerri, Imo state (FUTO) and it was free of impurities.

3.3 Methods

The tests carried out during this project were done in accordance to specification of the appropriate BS codes, (BS 812 – 103.1 1985: method for determination of particle size distribution e.t.c).

3.2.1 Production of Pozzolanic Ashes

Rice husk was obtained from rice milling factories in Afikpo local government area, Ebonyi State and Saw dust from wood mills in Naze timber market in Owerri, Imo State, all in South Eastern Nigeria. These materials were air-dried and calcined into ashes by open burning in a metal container available at FUTO. In this method, the sun dried rice husks and saw dust are burnt by placing them individually in heaps inside the metal container and allowed to burn in the open air for several hours, at a temperature of 650°C measured with a type K thermocouple. This method has been reported to produce ash with high quality and rich in amorphous silica. After burning, the RHA and SDA were allowed to cool; thereafter the RHA and SDA were sieved and large particles retained on the $600\mu\text{m}$ sieve were discarded while those passing the sieve were used for this work. No grinding or any special treatment to improve the quality of the ashes

and enhance their pozzolanicity was applied because I wanted to utilize simple process that could be easily replicated by local community dwellers.

3.2.2 Determination of Particle Size Distribution For Coarse And Fine Aggregate

This operation is used to divide a sample of aggregate into various fraction; each consisting of particles of the same size.

Apparatus

- i. Set of test sieves; 9.5mm, 4.75mm, 2.36mm, 1.18mm, 600 μ m, 150mm (for fine aggregate), 33mm, 18mm, 9.5mm, 4.75mm (for coarse aggregate).
- ii. Weighing balance
- iii. Wire brush
- iv. Oven

Procedure

Sufficient representative sample was collect from the stockpile and was reduced by quartering to obtain the quantity for test. The sample was washed with water several times until the water becomes clear; it was dried in the oven for 24hours. On the following day the sample was removed from the oven and allowed to cool; sieving commenced as soon as sample had cooled using 500g; the sieves were arranged in descending order starting with sieve 9.5mm down for the fine aggregate and sieve 9.5mm down for coarse aggregate. The sample was introduced into the arranged sieves with a receiver at the bottom and covered. It was shaken very well using hand, the sample retained in each sieves were then collected and weighed.

3.2.3 Determination of Particle Size Distribution For RHA and SDA

Aim: To know its suitability for concrete production.

Apparatus:

- i. Weighing balance
- ii. Hydrometer

This test was conducted in FUGRO NIGERIA LIMITED, in Eleme, Portharcourt, Rivers State, Nigeria. Particle size distribution (PSD) of the ashes was determined using the hydrometer method followed by sieving recommended for sediment and soil samples containing less than 35% fine particles i.e clays and silts. The test method is based on the BS1377 (Part 2; 1990) which is in accordance with the DUTCH RAW and the American ASTM D422.

3.2.4 Determination of Specific Gravity Of Aggregate (Fine and Coarse) and Pozzolanic Ashes (RHA and SDA)

Specific gravity of aggregate is used in design calculations of concrete mixes. It is defined as the ratio of the density of a material to the density of distilled water. For this project the specific gravity of aggregate were determined on saturated and surface dry (SSD) basis.

Apparatus

- i. Weighing balance
- ii. Pycnometer bottle
- iii. Sieves ($75\mu\text{m}$ for aggregate and 4.75mm for coarse aggregate)
- iv. Absorbent cloth
- v. Distilled water

Procedure

Sufficient representative sample was collected from the stockpile and quartered. The sample was sieved using sieve $75\mu\text{m}$ and 4.75mm for fine and coarse aggregate respectively. The aggregates were collected and washed thoroughly until the water was clear. After washing, the samples soaked in distilled water for 24hours, the following day the water was decanted and the samples were

spread on a metal plate to surface dry. Once the sample was dry, it was divided into four portions for the test. The weight of the pycnometer bottle was decanted and the bottle was dried with hand towel, then one portion of the sample was introduced into the empty bottle and weighed. The bottle with the sample was filled with water and weighed. The specific gravity is calculated from the results and the sequence is repeated for the second portion.

3.2.5 Pozzolanicity Test of Ashes

A form of pozzolanicity test was carried out for each of the ashes. It consists of mixing 20g of the ash with 100ml volume of Calcium hydroxide solution $[\text{Ca}(\text{OH})_2]$ in a 50ml burette, and titrating samples of the mixture against 0.1M of H_2SO_4 solution at time intervals of 30mins, 90mins, and 120mins respectively using Methyl orange as indicator at normal temperature. The mixture was stirred using a Labnet Orbit shaker (model 1000). For each of the ashes the titre values was found to reduce with time, confirming the ash as a pozzolana that fixed more and more of the calcium hydroxide, thereby reducing the alkalinity of the mixture.

3.2.6 Chemical Analysis and X-ray Diffraction of Ashes

The RHA and SDA were taken to FUGRO NIGERIA LIMITED, in Eleme, Portharcourt, Rivers State, Nigeria, for both the chemical analysis and X-ray diffraction test. The chemical analysis was done to determine the different oxide composition of the ash samples. The analysis was done using X-RAY FLUORESCENT Analyser (Model QX 1279). The X-ray Diffraction (XRD) analysis was performed to determine the silica phase of the RHA and SDA samples. They were scanned by an X-ray Diffractometer-6000 using cuka radiation at 60kv/80mA, Cps=1k, width=2.5, speed $2^\circ/\text{min}$ and scanning with angle of 2Θ from $3-70^\circ$.

3.2.7 Concrete Mix Design

The aim of mix design is to determine the proportion of each of the constituents of the concrete in the mix. In this research study, From the values obtained from particle size and specific gravity tests, concrete mix design was done using SCALE149 (STRUCTURAL CALCULATIONS ENSEMBLE) Software using a specified characteristic concrete strength of 25N/mm^2 at 28 days, 5% defectives, standard deviation of 8N/mm^2 and slump ranges 10-30mm. See Appendix C for printed concrete mix design form.

3.2.8 Batching of Constituent Materials

Batching of the constituent materials was by weight and cement, fine aggregate and coarse aggregate ratio of 1:2:3.5 and a water/cement ratio of 0.60 were used as shown in (Appendix C: printed concrete mix design form). Ordinary Portland cement was replaced with rice husk ash (RHA) and saw dust ash (SDA) in percentages of 0, 5, 10 & 15. OPC was also replaced by mixing RHA and SDA in a ratio 70:30 for each of the percentage replacements. Table 3.1 below shows the summary of mix proportions for each three concrete cylinders used for the concrete production.

Total number of concrete cylinders casted for the control = 9

Total number of concrete cylinders casted for the partial replacement = 243

Total number of concrete cylinders = 252

Dimension of concrete cylinder = 150*300mm or 0.15*0.3m

Where 150mm is the diameter and 300mm is the height.

Table 3.1: Summary of mix proportions for RHA, SDA, & RHA-SDA Concrete

Material	Unit	% Replacement			
		0%	5%	10%	15%
OPC	kg	4.56	4.33	4.1	3.88
RHA	kg	0	0.23	0.46	0.68
SDA	kg	0	0.23	0.46	0.68
RHA-SDA	kg	0	0.23	0.46	0.68
Water	kg	2.72	2.72	2.72	2.72
Fine aggregate	kg	9.92	9.92	9.92	9.92
Coarse aggregate	kg	14.8	14.8	14.8	14.8
w/c	-	0.6	0.6	0.6	0.6

3.2.9 Mixing of Concrete

The coarse and fine aggregate were first mixed and spread on the hard clean floor of the laboratory, the cement then spread uniformly over the mixed sand and crushed stone. The materials were shoveled over and over again from one end to another and cutting with shovel until the mix appeared uniform. Water was then added gradually so that neither water by itself nor cement could escape. The mixing operation was then repeated as for the dry state until the mixture appeared uniform in color and consistency.

3.2.10 Slump Test on Fresh Concrete

Slump Test was carried out to check the effect of RHA, SDA, and RHA-SDA on the workability of fresh concrete. The test was carried out in accordance with the requirements of BS1881: Part 102(1983) for slump test.

Apparatus

- i. Slump cone
- ii. Steel Tamping rod

- iii. Steel measuring ruler
- iv. Mason trowel

Procedure

The internal surface of the slump cone is thoroughly cleaned and freed from superfluous moisture and adherence of any old set concrete before commencing the test. The mould is placed on a smooth, horizontal, rigid and non-absorbant surface. A portion of the mixed concrete was placed in the cone in three layers. Each layer is tapped 25 times by the tamping rod. Care was taken to distribute the strokes over the cross section. This is to ensure the proper compaction of the concrete inside the cone. After the top layer has been rodded, the concrete is struck off level with a mason's trowel and tamping rod. The cone was gently and carefully lifted up in a vertical direction. The fresh concrete bulged sideways with a resultant reduction in the height of the concrete cone. The new height was measured; the difference between the height of the cone and the concrete cone after removal was determined.

3.2.11 Casting of Concrete Cylinders

Engine oil was applied inside the mould with the help of a brush in order to lubricate the mould and allow for easy demoulding. The thoroughly mixed concrete was filled into the cylindrical moulds in three layers with the aid of a mason's trowel. The first, second and third layers were given a compaction of 35 blows with the rammer at its own weight, spread uniformly over the cross section of the mould. The top of each mould was smoothed and leveled and the outside surfaces cleaned.

3.2.12 Curing of the Concrete Cylinders

The method adopted for the curing of the concrete cylinders was complete immersion in water. After twenty four hours of casting each set of the cylindrical concrete specimen produced were demoulded, then transported with the aid of wheel barrow to temporary rectangular steel curing tanks in workshop

III in Federal University of Technology Owerri (FUTO). The cylinders were immersed and left inside water for 28, 90 and 150days.

3.2.13 Tensile Strength Test on concrete cylinders

The concrete cylinders were crushed in accordance with BS1881-117:1983. Three concrete cylinders of each mix were tested by crushing in a Universal Testing Machine (See Appendix D for Universal Testing Machine) in the Civil Engineering Department Laboratory of FUTO after 28, 90, and 150days of curing respectively. First, to complete was 28 days set, followed by 90days and 150days.

The cylinders were placed, each, on the lower compression platen within the compression space of the machine. The upper platen consisting of two adjustable platens, was brought down gently to rest on the concrete cylinder. The cylinder was then loaded by activating the loading valve and setting it on approximately constant loading rate. The load was steadily applied until fracture occurred. The concrete cylinders were centrally located before crushing to ensure best results. Average of the crushing loads were taken for each set of cylinders and the Split Tensile Strength evaluated using Split Tensile Strength,

$$\sigma_T = \frac{2P}{\pi DL} \text{ (N/mm}^2\text{)} \dots\dots\dots 3.1$$

Where,

P = Average Compressive Load at failure (N)

L = Length of Cylinder = 300mm

D = Diameter of Cylinder = 150mm

3.2.14 Model Development

The models developed in this study were done using Excel Spreadsheet Regression Analysis. The model for the Split tensile strength estimation was

done in the standard linear – interactive manner according to Cindy and Robert, (2007). Relationship between various variables were established.

3.2.15 Adequacy Test For the Model

A statistical adequacy test for the mathematical model was done using statistical student's t-test at 95% accuracy level. The following two hypotheses were tested using statistical student's t-test.

- i. Null Hypothesis:** There is no significant difference between the laboratory concrete cylinder split tensile strengths and predicted split tensile strength results at 95% accuracy level.

- ii. Alternative Hypothesis:** There is a significant difference between the laboratory concrete cylinder split tensile strengths and predicted split tensile strength results at 95% accuracy level.

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 Results

4.1.1 Characterization of RHA and SDA

The physical properties of the Rice husk ash (RHA) and sawdust ash (SDA) are as shown in Table 4.1 below.

Table 4,1: Physical properties of RHA and SDA

Physical properties	Rice husk ash	Sawdust ash
Colour	grey	grey
Specific gravity	1.92	1.84
Bulk density	780kg/m ³	750kg/m ³
Fineness modulus	1.48	1.86
Grading curve zone	Zone 2	Zone 2

Oxide Composition of RHA and SDA are as shown in Table 4.2 below.

Table 4.2: Oxide Composition of RHA and SDA by XRF

S/No	Oxide	RHA %composition	SDA % composition
1	Silica (SiO ₂)	85.79	47.12
2	Aluminum oxide (Al ₂ O ₃)	1.45	2.16
3	Calcium oxide (CaO)	1.38	23.43
4	Manganese oxide (MnO)	0.18	0.24
5	Zink oxide (ZnO)	0.03	0.03
6	Ferrous oxide (Fe ₂ O ₃)	0.43	0.75
7	Magnesium oxide (MgO)	2.45	7.50
8	Potsassium oxide (K ₂ O)	1.82	10.08
9	Sodium oxide(S O ₃)	0.79	2.59
10	Loss on Ignition	5.40	7.59

4.1.2 Tensile Strengths

4.1.2.1 Tensile Strength of RHA-OPC Concrete

Table 4.3 below shows the tensile strength of concrete containing RHA at 28, 90, and 150days of curing for different percentage replacements. See appendix E for detailed tensile strength of RHA concrete.

Table 4.3 Tensile Strength of Concrete Containing RHA

Curing age / % Replacement		Split Tensile strength (N/mm ²)
28Days	0%	0.88
	5%	0.86
	10%	0.78
	15%	0.70
90Days	0%	1.34
	5%	1.42
	10%	1.37
	15%	1.25
150Days	0%	1.61
	5%	1.85
	10%	1.74
	15%	1.59

4.1.2.2 Tensile Strength of SDA-OPC Concrete

Table 4.4 below shows the tensile strength of concrete containing SDA at 28, 90, and 150days of curing for different percentage replacements. See appendix F for detailed tensile strength of SDA concrete.

Table 4.4 Tensile Strength of Concrete Containing SDA

Curing age / % Replacement		Split Tensile strength (N/mm ²) SDA
28Days	0%	0.88
	5%	0.77
	10%	0.58
	15%	0.62
90Days	0%	1.34
	5%	1.28
	10%	1.15
	15%	0.97
150Days	0%	1.61
	5%	1.65
	10%	1.50
	15%	1.47

4.1.2.3 Tensile Strength of RHA-SDA-OPC Concrete

Table 4.5 below shows the tensile strength of concrete containing RHA-SDA at 28, 90, and 150days of curing for different percentage replacements. See appendix G for detailed tensile strength of RHA-SDA concrete.

Table 4.5 Tensile Strength of Concrete Containing RHA-SDA

Curing age / % Replacement		Split Tensile strength (N/mm ²) RHA-SDA
28Days	0%	0.88
	5%	0.82
	10%	0.64
	15%	0.50
90Days	0%	1.34
	5%	1.35
	10%	1.33
	15%	1.18
150Days	0%	1.61
	5%	1.66
	10%	1.69
	15%	1.49

4.1.3 Models

The models developed are presented below where Y represents tensile strength, X₁ represents curing age in days and X₂ represents Percentage replacement.

4.1.3.1 Model for Tensile Strength of RHA-OPC Concrete

The model equation is:

$$Y = 0.68378 + 0.007323X_1 - 0.0074X_2$$

Table 4.6 below shows the predicted values for the twelve observations for RHA-Concrete.

Table 4.6 Predicted tensile strength of concrete containing RHA

Observation	Predicted Split Tensile Strength of RHA (N/mm ²)	Residuals
1	0.888836	-0.00884
2	0.851836	0.008163
3	0.814836	-0.03484
4	0.777836	-0.07784
5	1.342882	-0.00288
6	1.305882	0.114118
7	1.268882	0.101118
8	1.231882	0.018118
9	1.782281	-0.17228
10	1.745281	0.104719
11	1.708281	0.031719
12	1.671281	-0.08128

4.1.3.2 Model for Tensile Strength of SDA-OPC Concrete

The model equation is:

$$Y = 0.61579 + 0.00693X_1 - 0.01853X_2$$

Table 4.7 below shows the predicted values for the twelve observations for SDA-Concrete.

Table 4.7 Predicted tensile strength of concrete containing SDA

Observation	Predicted Split Tensile Strength of SDA (N/mm ²)	Residuals
1	0.865621	0.014379
2	0.772954	-0.00295
3	0.680287	-0.10029
4	0.587621	0.032379
5	1.295286	0.044713
6	1.202620	0.07738
7	1.109953	0.040047
8	1.017286	-0.04729
9	1.711091	-0.10109
10	1.618425	0.031575
11	1.525758	-0.02576
12	1.433091	0.036908

4.1.3.3 Model for Tensile Strength of RHA-SDA-OPC Concrete

The model equation is:

$$Y = 0.653102 + 0.007409X_1 - 0.01433X_2$$

Table 4.8 below shows the predicted values for the twelve observations for RHA-SDA-Concrete.

Table 4.8 Predicted tensile strength of concrete containing RHA-SDA

Observation	Predicted Split Tensile Strength of RHA-SDA (N/mm ²)	Residuals
1	0.860562	0.019437
2	0.788895	0.031104
3	0.717229	-0.07723
4	0.645562	-0.14556
5	1.319939	0.02006
6	1.248272	0.101727
7	1.176606	0.153394
8	1.104939	0.07506
9	1.764497	-0.1545
10	1.692831	-0.03283
11	1.621164	0.068835
12	1.549497	-0.0595

4.2 Discussions

4.2.1 Characterization of Ashes

4.2.1.1 Physical Properties of RHA and SDA

The Rice husk ash (RHA) sample were grey in colour. The RHA had a specific gravity of 1.86, a bulk density of 780kg/m³ and a fineness modulus of 1.48. The specific gravity value is less than the value (2.05 and 2.10) reported by Kartini et al., 2010 and Bui et al., (2005). However, it is higher than the value (1.85) reported by Ettu et al., (2013). The bulk density and fineness modulus values are higher than the values (770kg/m³ and 1.45) reported by Ettu et al., (2013). The RHA falls within zone 2 of the grading curve according to Elinwa and

Ejeh, (2004). See Appendix H for Grading Curve of RHA. The Sawdust ash (SDA) had the same colour of grey like the RHA. The SDA had a specific gravity of 1.84, and a bulk density of 750kg/m^3 and a fineness modulus of 1.86. These values are all lower than the values reported by Ettu et al., (2013) which are 2.05, 820kg/m^3 , and 1.89 for the specific gravity, bulk density and fineness modulus respectively. The SDA falls within zone 2 of the grading curve according to Elinwa and Ejeh, (2004). See Appendix I for Grading Curve of SDA.

4.2.1.2 Oxide Composition of RHA and SDA

The result showed that The RHA have combined percentages of ($\text{SiO}_2 + \text{Al}_2\text{O}_3 + \text{Fe}_2\text{O}_3$) of 87.67% which is more than 70% indicating that it is a good pozzolanic material in accordance with the requirement in ASTM C618-1987. The SDA have combined percentages of ($\text{SiO}_2 + \text{Al}_2\text{O}_3 + \text{Fe}_2\text{O}_3$) of 50.03% which is less than 70% but its loss on ignition of 7.59 is less than 12 in accordance with the requirement in ASTM C618-1987. Calcium hydroxide is responsible for the formation of C_3S ($3\text{CaO}.\text{SiO}_2$) in cement and all other clinker minerals, responsible for the strength in the late or early part of the concrete (Taylor, 1990). The percentage calcium oxide in RHA was 1.38% much lower than that in SDA (23.43%). Similarly, low magnesia content was obtained in both the RHA and SDA falling within the same range of 0.18-0.24%. High magnesia content is not required in cement because it causes unsoundness (Taylor, 1990). On the other hand, the percentage iron III oxide in RHA and SDA are 0.43% and 0.75%. Iron III oxide is responsible for imparting colour on cement product (Lea, 1970).

Potassium oxides are the main alkalis associated with cement. There was high percentage of potassium oxide in the SDA than in RHA. High alkali is undesirable as it results in alkali-silicate reaction and attacks reinforced concrete as reported by Taylor, (1990).

4.2.1.3 Pozzolanicity Test

The pozzolanicity test confirmed both ashes as pozzolans since they fixed some quantities of lime $[\text{Ca}(\text{OH})_2]$ over time.

4.2.1.4 X-ray Diffraction of RHA and SDA

The XRD of the rice husk ash and saw dust ash, (See Appendix J and K for the XRD pattern of the RHA and SDA) indicates that the ashes contain certain amount of crystalline silica in the form of cristobalite and tridymite. This indicated that, if the temperature at which the ashes were produced had been increased between 700°C - 1000°C , more crystallization of the amorphous silica would have taken place. Further analysis show that the RHA contains more amorphous silica than the SDA.

4.2.1.5 Workability of RHA and SDA Concrete

The results of the slump indicating the workability of RHA/, SDA, and RHA-SDA Concrete are shown in Appendix L (Table of slump values). The table indicates that the slump values decreased more for the RHA and RHA-SDA concrete than the SDA concrete. From the result, it was noticed that the concrete became less workable as the RHA, SDA and RHA-SDA percentage increases meaning that more water is required to make the mixes more workable. The high demand for water in the RHA and RHA-SDA concrete as the ashes increases is due to increased amount of silica in the mixture. This is typical of pozzolan cement concrete as the silica-lime reaction requires more water in addition to the water needed during hydration of cement (Hague and Kayali, 1998; Waswa-Sabuni et al. 2002; Adesanya and Raheem, 2009a).

4.2.2 Tensile Strength

4.2.2.1 Tensile strength of RHA-OPC concrete

Table 4.2 indicates that split tensile strength generally increases with curing period and decreases with increased amount of the pozzolanic ashes. The result

at 28 day showed a decrease in strength from 0.88N/mm^2 for control to 0.86N/mm^2 , 0.78N/mm^2 , and 0.70N/mm^2 for 5, 10 and 15% RHA replacement respectively. These results indicate that concrete containing rice husk ash (RHA) gain strength slowly at early curing age. This is in line with previous findings that concrete containing pozzolanic materials gained strength slowly at early curing ages as being a result of the low rate of pozzolanic reaction at those early ages (Hossain, 2005; Adesanya and Raheem, 2009a; Ettu et al., 2013e; Ettu et al., 2013f; Ikpong and Okpala, 1992).

The result at 90days indicated that pozzolanic reaction had commenced as evident from the higher percentage increase in split tensile strength by 5%, and 10% RHA concrete over that of the control. The control concrete gave a strength of 1.34N/mm^2 at 90 day while the 5% and 10% RHA concrete had a strength of 1.42N/mm^2 and 1.37N/mm^2 respectively. The percentage increase were 8%, and 3%, for 5%, and 10% RHA replacements. This increase in split tensile strength can be attributed to the high silica content. The silica from the RHA reacts with calcium hydroxide liberated as a by-product during the hydration of OPC to form additional calcium-silicate-hydrate (C-S-H) that increases the binder efficiency and the corresponding strength values at later days of curing.

At 150 days, there was a significant improvement in split tensile strength development beyond 90 days as shown in Table 4.2. The increase in strength from 28 to 150 days for the control was 0.46N/mm^2 and 0.73N/mm^2 . Higher increases in strength than at 90 days were observed confirming that cement continues to hydrate. However, the significant increases in 5%, 10% and 15% of the split tensile strength of RHA suggests that some pozzolanic activity are still going on at such later curing ages (Ettu et al., 2013). The strength gain can be attributed to the cementitious products formed as a result of hydration of cement and those formed when lime reacts with the pozzolan incorporated (Balendran

and Martin-Buades, 2000; Adesanya and Raheem, 2009a; Ramasamy, 2012). An optimum value of 1.85N/mm^2 at 150 days was obtained for concrete with 5% RHA. Similarly, an optimum value of 1.74N/mm^2 at 150 days was obtained for concrete with 10% RHA. Also an optimum value of 1.74N/mm^2 at 150 days was obtained for concrete with 15% RHA.

4.2.2.2 Tensile strength of SDA-OPC concrete

It can be seen from Table 4.3 indicates that split tensile strength generally increases with curing period, this is in accordance with Mehta and Monteiro, (2006), that so long as the hydration of anhydrous cement particles goes on, concrete strength increases with increase in the moist curing period. This is so because the strength of concrete depends on the amount of gel (the C-S-H which is the essential cementing compound) in the cement paste at any time, and this itself is a function of age. The result at 28 and 90 day showed a decrease in strength from 0.88N/mm^2 for control to 0.77N/mm^2 , 0.62N/mm^2 , and 0.58N/mm^2 for 5, 15 and 10% SDA at 28 day and 1.34N/mm^2 for control to 1.28N/mm^2 , 1.15N/mm^2 , and 0.97N/mm^2 for 5, 10 and 15% SDA at 90 day respectively. The SDA concrete gave a split tensile strength lower than the control for all replacement levels at 28 and 90 day. This decrease in split tensile strength can be attributed to the low silica content of SDA of 47.12% see Table 4.1 above. It is observed that 15% SDA replacement gave a strength higher than 10% at 28 day curing age, this could be attributed to proper compaction of the 15% concrete cylinders during casting.

At 150 days, 5% SDA gave a strength of 1.65N/mm^2 which is greater than the control strength of 1.61N/mm^2 . The increase in strength confirms that cement continues to hydrate and that pozzolanic reaction continued to take place. The strength gain can be attributed to the cementitious products formed as a result of hydration of cement and those formed when lime reacts with the SDA pozzolan incorporated (Balendran and Martin-Buades, 2000; Adesanya and Raheem,

2009a). An optimum value of 1.65N/mm^2 at 150 days was obtained for concrete with 5% SDA. Similarly, an optimum value of 1.50N/mm^2 at 150 days was obtained for concrete with 10% SDA. Also an optimum value of 1.47N/mm^2 at 150 days was obtained for concrete with 15% SDA.

4.2.2.3 Tensile strength of RHA-SDA-OPC concrete

Results in Table 4.4 indicates that split tensile strength followed the same trend of increasing with increased curing age. The result at 28 day showed a strength of 0.88N/mm^2 for control while that for 5%, 10% and 15% RHA-SDA concrete were 0.82N/mm^2 , 0.64N/mm^2 , and 0.50N/mm^2 respectively. It is clear from the result that the 5% RHA-SDA concrete gave a strength comparable with the control at 28 day curing age. There is 5% increase in strength for the 5% ternary blend i.e RHA-SDA-OPC concrete compared to binary blend of SDA-OPC concrete from 0.77N/mm^2 to 0.82N/mm^2 at 28 day curing age. This can be attributed to the more silica content present in the ternary blend which is as a result of the addition of RHA which has a high silica content of 87.67% as evidenced in the XRF analysis in the ratio of 70:30 in the blend. The strength values for all replacement levels are lower than the control at 28 day which is still in line with previous findings that concrete containing pozzolanic materials gained strength slowly at early curing ages as being a result of the low rate of pozzolanic reaction at those early ages (Ettu et al., 2013e; Ettu et al., 2013f; Hossain, 2005; Adesanya and Raheem, 2009a; Ikpong and Okpala, 1992).

The 90 days result showed the 5% replacement strength of 1.35N/mm^2 been higher than the control strength of 1.34N/mm^2 . The 10% replacement strength of 1.33N/mm^2 is also much comparable with the control. This indicates that pozzolanic reaction had commenced. The silica from the RHA and SDA reacted with calcium hydroxide liberated as a by-product during the hydration of OPC to form additional calcium-silicate-hydrate (C-S-H) that increased the binder efficiency and the corresponding strength values.

At 150 days, the split tensile strength development of the 5% and 10% RHA-SDA concrete of 1.66N/mm^2 and 1.69N/mm^2 was far greater than the control strength of 1.61N/mm^2 . Though the 15% RHA-SDA concrete strength of 1.49N/mm^2 is below the control, it is however comparable. The strength gain can be attributed to the cementitious products formed as a result of hydration of cement and those formed when lime reacts with the RHA-SDA pozzolan incorporated (Balendran and Martin-Buades, 2000; Adesanya and Raheem, 2009a).

4.2.3 Models

4.2.3.1 Model for Tensile Strength of RHA-OPC concrete

If $t \text{ Stat} < - t \text{ Critical two tail}$ or $t \text{ Stat} > t \text{ Critical two-tail}$, reject the null hypothesis, that is accept the alternative hypothesis, otherwise accept the null hypothesis. The values in the condition stated above are obtained from appendix M.

From appendix M, $t \text{ Stat} = 0 < t \text{ Critical two-tail} = 2.073873$, hence null hypothesis is accepted and alternative hypothesis rejected. Hence, the model is adequate.

4.2.3.2 Model for Tensile Strength of SDA-OPC concrete

If $t \text{ Stat} < - t \text{ Critical two tail}$ or $t \text{ Stat} > t \text{ Critical two-tail}$, reject the null hypothesis, that is accept the alternative hypothesis, otherwise accept the null hypothesis. The values in the condition stated above are obtained from appendix N.

From appendix N, $t \text{ Stat} = -(1.4 \times 10^{-15}) < t \text{ Critical two-tail} = 2.073873$, hence null hypothesis is accepted and alternative hypothesis rejected. Hence, the model is adequate.

4.2.3.3 Model for Tensile Strength of RHA-SDA-OPC concrete

If $t \text{ Stat} < - t \text{ Critical two tail}$ or $t \text{ Stat} > t \text{ Critical two-tail}$, reject the null hypothesis, that is accept the alternative hypothesis, otherwise accept the null hypothesis. The values in the condition stated above are obtained from appendix O.

From appendix O, $t \text{ Stat} = 0 < t \text{ Critical two-tail} = 2.073873$, hence null hypothesis is accepted and alternative hypothesis rejected. Hence, the model is adequate.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

From the findings of this work, the following conclusions were drawn;

- i. RHA-OPC concrete had higher water demand, followed by RHA-SDA-OPC concrete.
- ii. Ternary blend of RHA-SDA-OPC concrete enjoy maximum benefit of strength gain compared to SDA-OPC concrete.
- iii. The Split Tensile strength of RHA-OPC, SDA-OPC and RHA-SDA-OPC Concrete generally decreases as the percentage replacement of OPC increases.
- iv. Tensile strength model for a two component RHA-OPC concrete, SDA-OPC concrete and three component RHA-SDA-OPC concrete was developed.

5.2 Recommendations

From the results of this investigation, the following recommendations are made;

- i. Concrete produced with 5% to 15% replacement of OPC with RHA could be used in general reinforced concrete works such as in tall buildings, bridges, etc. and in unreinforced concrete structures such as pavement slabs, airfield runway and dams.
- ii. Concrete produced with 5% to 15% replacement of OPC with SDA could be used for general concrete works where strength is of less importance such as in mass concrete and floor screed.
- iii. Concrete produced with 5% to 10% replacement of OPC with RHA-SDA could be used for general reinforced concrete works, 15% for minor works in concrete.
- iv. Tensile strength models developed could be used by concrete industries to predict strength of concrete.

5.3 Contributions to Knowledge

- i. This study has brought out clearly the variation in the split tensile strength of concrete when OPC is replaced with RHA, SDA, and RHA-SDA.
- ii. The models developed could be used by any concrete industry and researchers to predict the Tensile strength of concrete at any given curing period and percentage replacement of OPC with RHA, SDA, and RHA-SDA.

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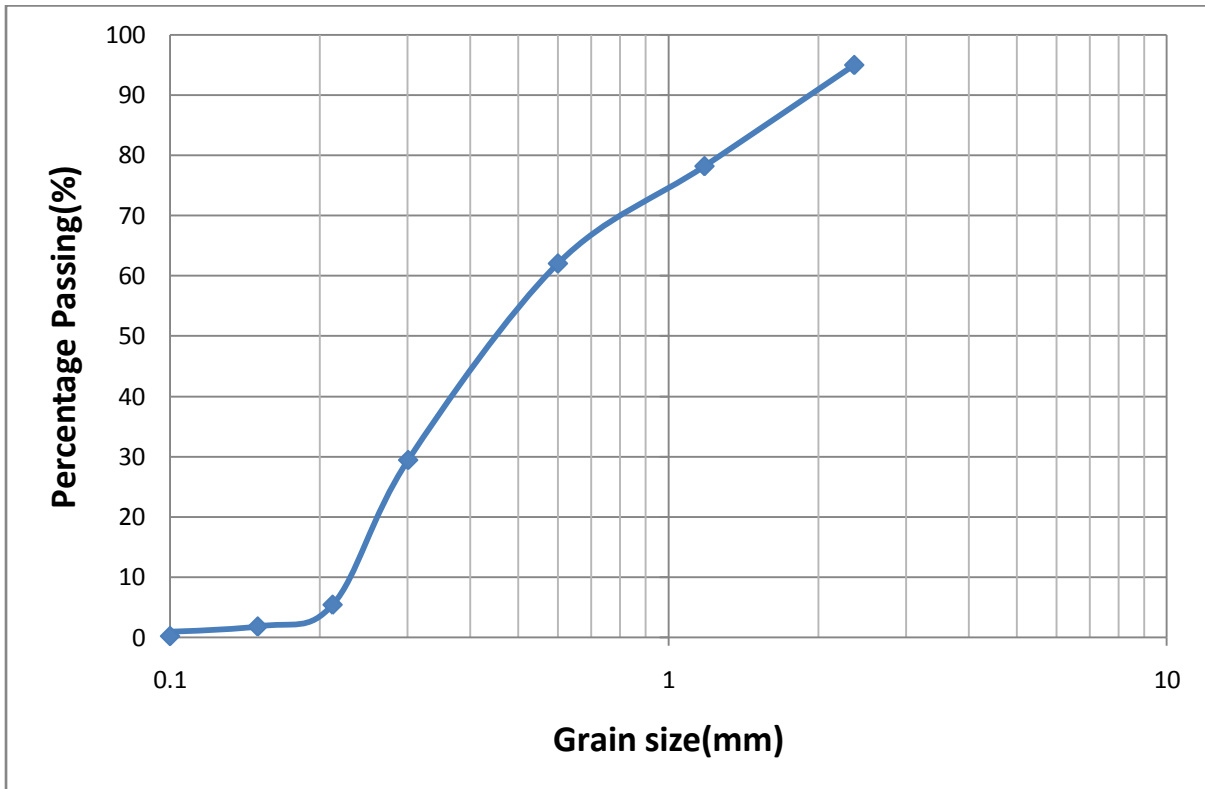
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APPENDICES

Appendix A

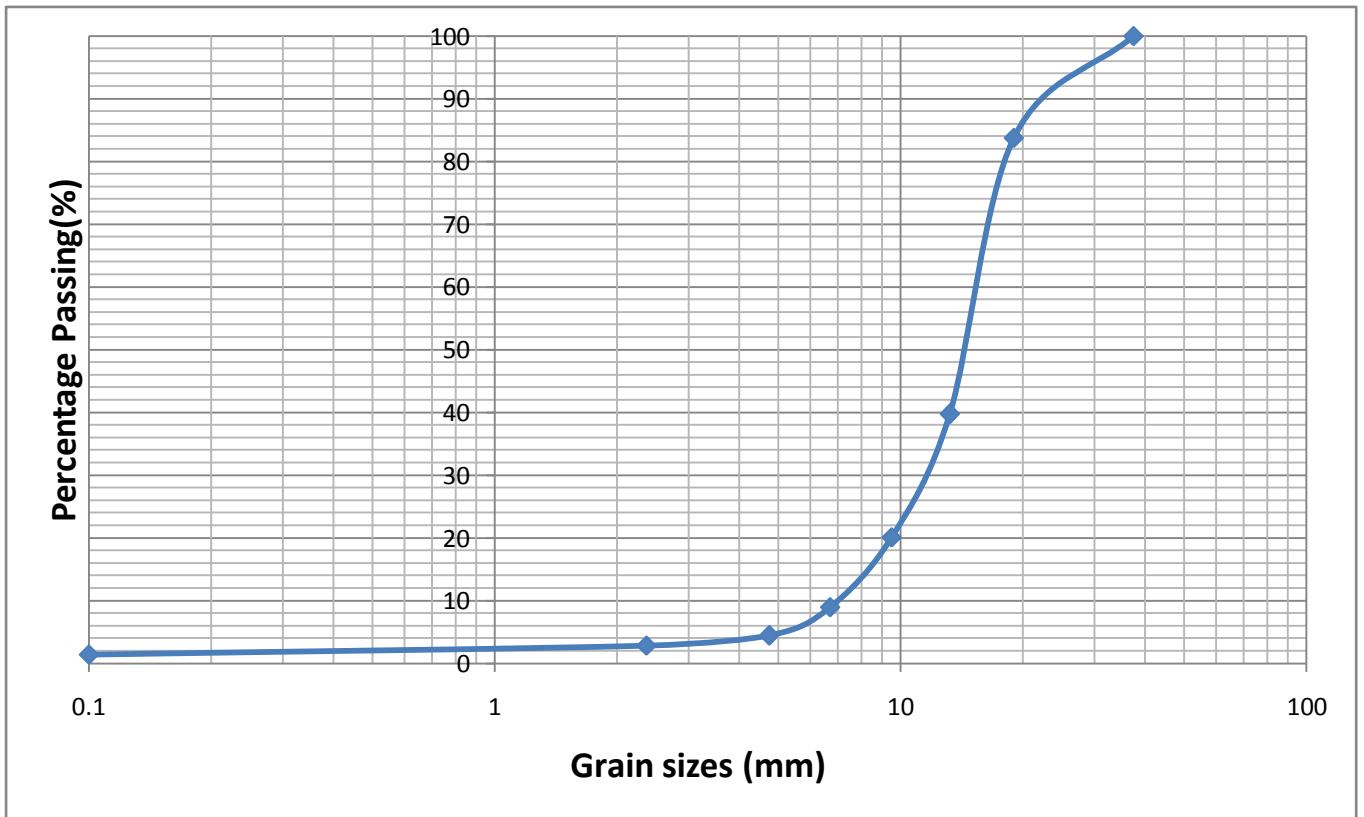
Grading curve of river sand



Sieve analysis of fine aggregate (river sand)

Sieve sizes(m m)	Mass Retained (gm)	Cumulative mass retained(g)	Percent 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

Appendix B
Grading curve of crushed sandstone



Sieve analysis of coarse aggregate (crushed sandstone)

Sieve Sizes	Mass Retained(gm)	cumulative mass retained(gm)	% mass retained	cumulative mass retained(%)	(%) passing
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
0	75	4929	1.5	98.58	1.42

Appendix C
Printed concrete mix design form

Page: 1
Made by: Ezenkwa
Date:
Ref No: 10

Office: 5786

Concrete mix design form

Mix title: Design of Normal Concrete Mixes

Stage	Item	Reference or calculation	Values			
1	1.1 Char. strength	Specified	25 N/mm ² at 28 days			
	1.2 Standard Deviation	defectives Fig 3	5 % 8 N/mm ²			
	1.3 Margin	C1 (k=1.64)	1.64 x 8 = 13 N/mm ²			
	1.4 Target strength	C2	25 & 13 = 38 N/mm ²			
	1.5 Cement type	Specified	OPC			
	1.6 Aggregate type: coarse		Crushed			
	Aggregate type: fine		Uncrushed			
	1.7 Free-water/cement ratio	Table2, Fig 4	0.6			
2	2.1 Slump or Vebe time	Specified	Slump 10-30 mm or Vebe 6-12 s			
	2.2 Maximum aggregate size	Specified	20 mm			
	2.3 Free-water content	Table 3	170 kg/m ³ 2/3 (160) + 1/3 (190)			
3	3.1 Cement content	C3	170 / 0.6 = 285 kg/m ³			
4	4.1 Relative density of aggregate (SSD)		2.7 known			
	4.2 Concrete density	Fig 5	2460 kg/m ³			
	4.3 Total Agg. content	C4	2460 - 285 - 170 = 2005 kg/m ³			
5	5.1 Grading of fine aggregate	Percentage passing 600 micron sieve	62			
	5.2 Proportion of fine aggregate	Fig 6	31			
	5.3 Fine agg. content	C5	2005 x 0.31 = 620 kg/m ³			
	5.4 Coarse agg. content	C5	2005 - 620 = 1385 kg/m ³			
Quantities		Cement (kg)	Water (kg)	Fine Agg. (kg)	Coarse Aggregate (kg)	
					10 mm	20 mm
per m ³ (to near 5 kg)		285	170	620	460	925
per trial mix		4.56	2.72	9.92	7.36	14.8
of 0.016 m ³ (3 conc. cylinders)						

No149

Appendix D

Universal Compressive Strength Testing Machine



Appendix E

Tensile strength of RHA concrete

Curing Age/ % Replacement		Mass m (kg)			Average mass m (kg)	Area (A) (mm ²)	Failure Load P (KN)			Average Failure Load P (KN)	Volume V (m ³)	Density m/v (kg/m ³)	Split Tensile strength (N/mm ²)
		M ₁	M ₂	M ₃			P ₁	P ₂	P ₃				
28Day	0%	13.0	12.8	13.2	13.00	141428.57	62.5	60.0	65.0	62.50	0.00530	2450.98	0.88
	5%	13.2	12.0	12.7	12.63	141428.57	58.0	61.2	63.3	60.83	0.00530	2383.02	0.86
	10%	12.2	12.2	12.50	12.30	141428.57	59.33	50.0	55.5	54.94	0.00530	2320.75	0.78
	15%	12.6	12.6	13.20	12.80	141428.57	45.60	49.40	52.10	49.03	0.00530	2415.09	0.70
90Day	0%	12.9	12.88	13.2	12.99	141428.57	98.00	88.50	97.00	94.50	0.00530	2450.48	1.34
	5%	13.1	12.62	12.7	12.81	141428.57	101.90	100.20	98.90	100.33	0.00530	2416.53	1.42
	10%	12.2	12.80	13.1	12.70	141428.57	97.66	94.10	99.04	96.93	0.00530	2395.77	1.37
	15%	13.2	12.6	12.5	12.77	141428.57	84.15	90.70	89.30	88.05	0.00530	2408.98	1.25
150Day	0%	13.0	12.92	12.80	12.91	141428.57	120.01	112.2	110.17	114.13	0.00530	2416.85	1.61
	5%	13.3	13.01	13.1	13.14	141428.57	136.00	125.0	132.40	131.13	0.00530	2479.25	1.85
	10%	13.2	12.60	12.85	12.90	141428.57	122.40	127.0	120.15	123.18	0.00530	2433.96	1.74
	15%	13.0	12.90	13.2	12.70	141428.57	112.80	110.2	114.64	112.55	0.00530	2396.23	1.59

Appendix F

Tensile strength of SDA concrete

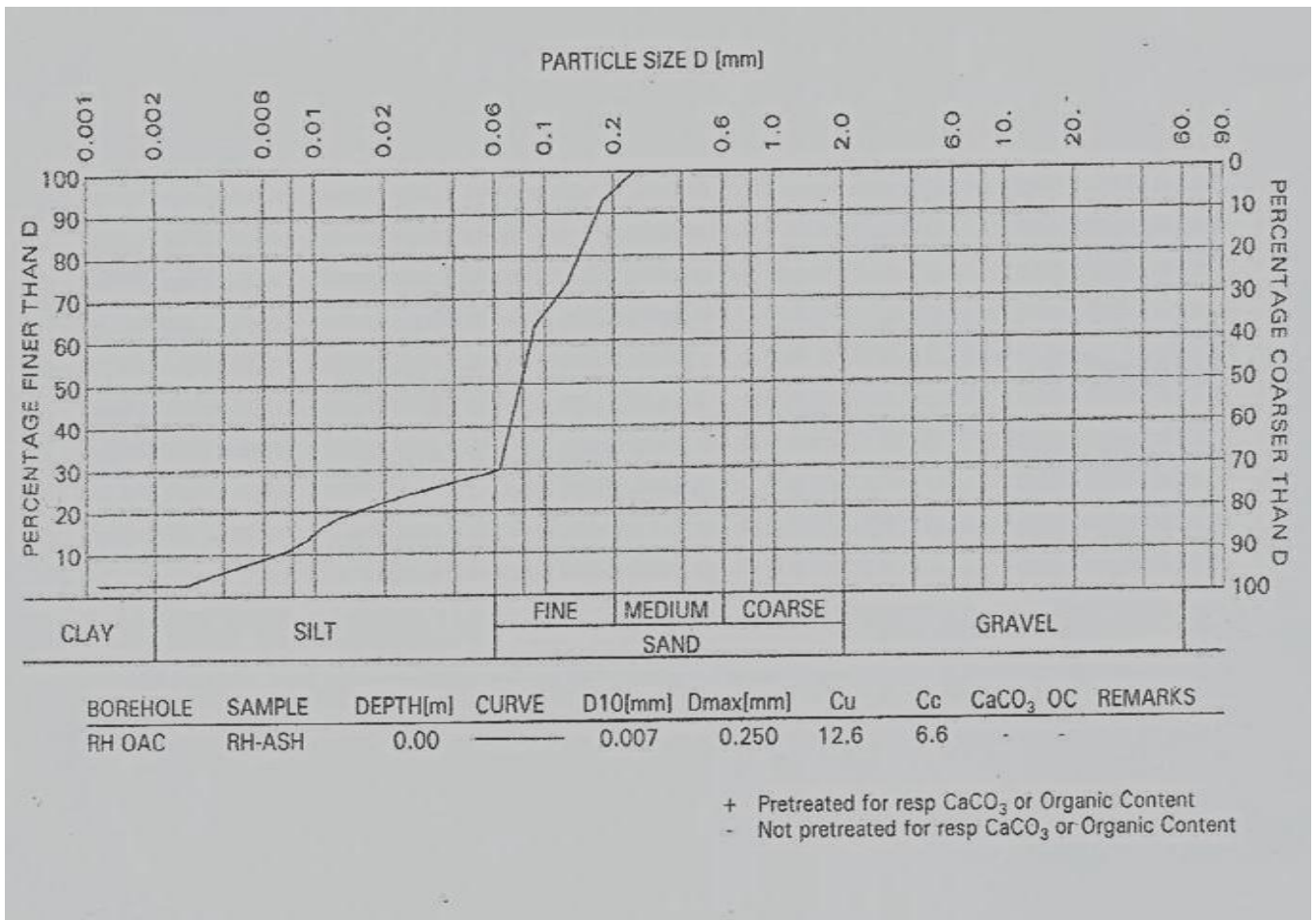
Curing Age/% Replacement		Mass m (kg)			Average mass m (kg)	Area (A) (mm ²)	Failure Load P (KN)			Average Failure Load P (KN)	Volume V (m ³)	Density m/v (kg/m ³)	Split Tensile strength (N/mm ²)
		M ₁	M ₂	M ₃			P ₁	P ₂	P ₃				
28Day	0%	13.0	12.8	13.2	13.00	141428.57	62.5	60.0	65.0	62.50	0.00530	2450.98	0.88
	5%	13.1	12.0	12.8	12.63	141428.57	50.08	57.31	56.2	54.53	0.00530	2381.22	0.77
	10%	12.1	12.1	12.66	12.27	141428.57	43.32	40.40	39.5	41.07	0.00530	2313.35	0.58
	15%	12.7	12.6	13.1	12.80	141428.57	41.10	44.27	46.30	43.89	0.00530	2413.27	0.62
90Day	0%	12.9	12.88	13.2	12.99	141428.57	98.00	88.50	97.00	94.50	0.00530	2450.48	1.34
	5%	12.2	12.6	12.0	12.27	141428.57	91.86	87.03	93.12	90.67	0.00530	2314.66	1.28
	10%	13.0	12.9	12.4	12.77	141428.57	84.09	79.40	80.50	81.33	0.00530	2408.98	1.15
	15%	12.8	12.9	12.72	12.81	141428.57	69.21	70.06	65.55	68.27	0.00530	2416.53	0.97
150Day	0%	13.3	13.01	13.1	13.14	141428.57	120.01	112.2	110.17	114.13	0.00530	2479.25	1.61
	5%	12.8	12.70	13.2	12.90	141428.57	113.58	119.62	117.04	116.75	0.00530	2422.64	1.65
	10%	12.9	12.30	12.8	12.67	141428.57	105.50	108.00	103.02	106.17	0.00530	2390.57	1.50
	15%	13.2	13.0	12.4	12.87	141428.57	104.20	104.00	102.60	103.60	0.00530	2428.30	1.47

Appendix G

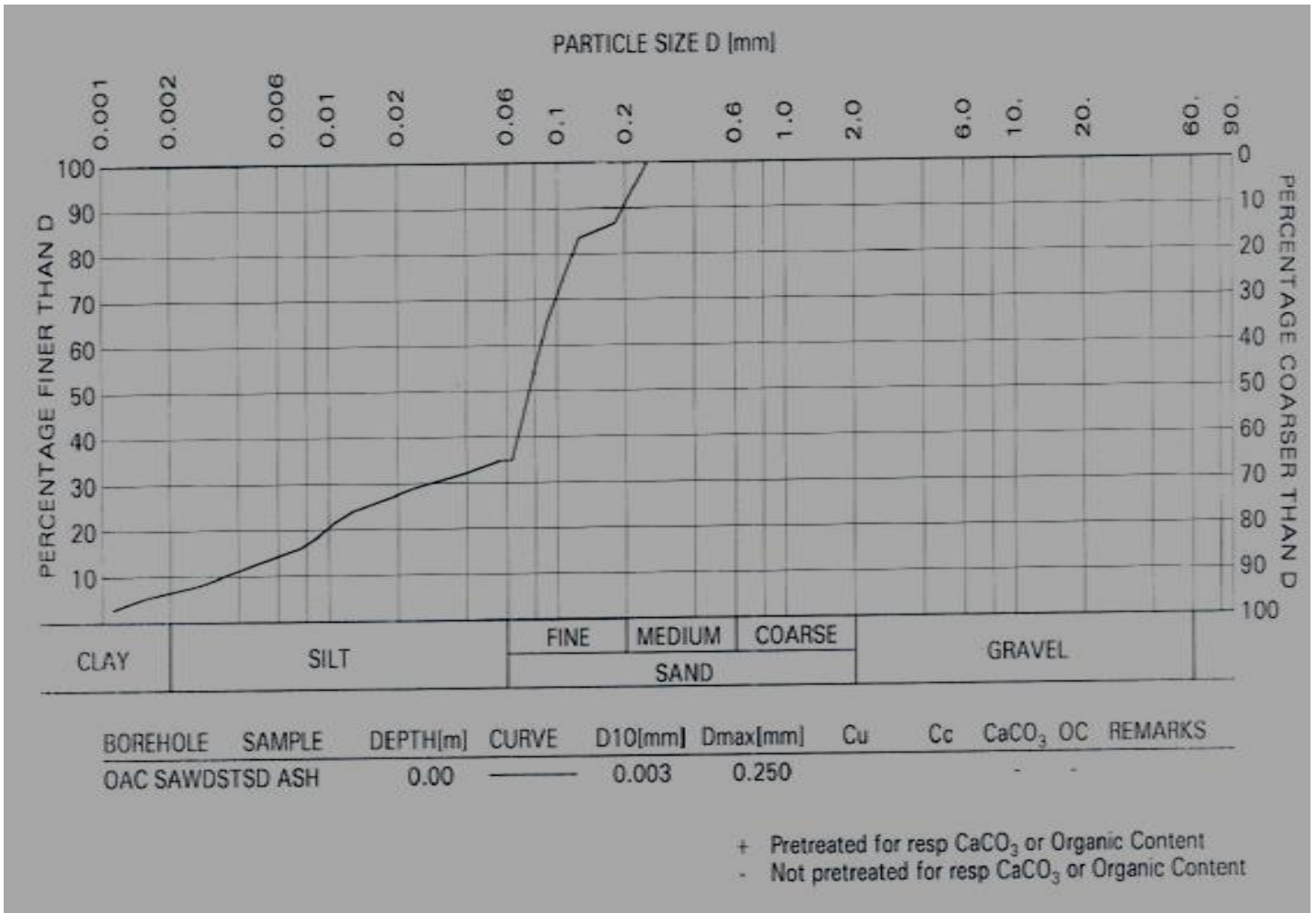
Tensile strength of RHA/SDA concrete

Curing Age/% Replacement		Mass m (kg)			Average mass m (kg)	Area (A) (mm ²)	Failure Load P (KN)			Average Failure Load, P (KN)	Volume V (m ³)	Density m/v (kg/m ³)	Split Tensile strength (N/mm ²)
		M ₁	M ₂	M ₃			P ₁	P ₂	P ₃				
28Day	0%	13.0	12.8	13.2	13.00	141428.57	62.5	60.0	65.0	62.50	0.00530	2450.98	0.88
	5%	13.3	12.7	12.9	12.97	141428.57	58.7	55.4	60.2	58.10	0.00530	2445.32	0.82
	10%	12.4	13.1	12.3	12.67	141428.57	44.0	44.9	46.5	45.13	0.00530	2388.76	0.64
	15%	12.9	13.0	13.1	13.00	141428.57	36.20	35.9	33.94	35.34	0.00530	2450.98	0.50
90Day	0%	12.9	12.88	13.2	12.99	141428.57	98.00	88.50	97.00	94.50	0.00530	2450.48	1.34
	5%	12.4	12.8	12.6	12.60	141428.57	95.90	94.45	96.77	95.71	0.00530	2376.91	1.35
	10%	13.1	12.8	13.2	13.03	141428.57	92.70	93.56	96.28	94.18	0.00530	2458.03	1.33
	15%	12.0	12.3	12.7	12.33	141428.57	84.23	83.00	83.50	83.58	0.00530	2325.98	1.18
150Day	0%	13.3	13.01	13.1	13.14	141428.57	120.01	112.2	110.17	114.13	0.00530	2479.25	1.61
	5%	12.7	13.0	12.8	12.83	141428.57	116.30	118.8	119.60	117.57	0.00530	2420.75	1.66
	10%	12.8	12.9	12.7	12.80	141428.57	115.00	122.70	120.44	119.38	0.00530	2415.09	1.69
	15%	12.8	12.75	12.8	12.78	141428.57	103.50	104.00	107.4	105.00	0.00530	2411.32	1.49

Appendix H Grading Curve of RHA

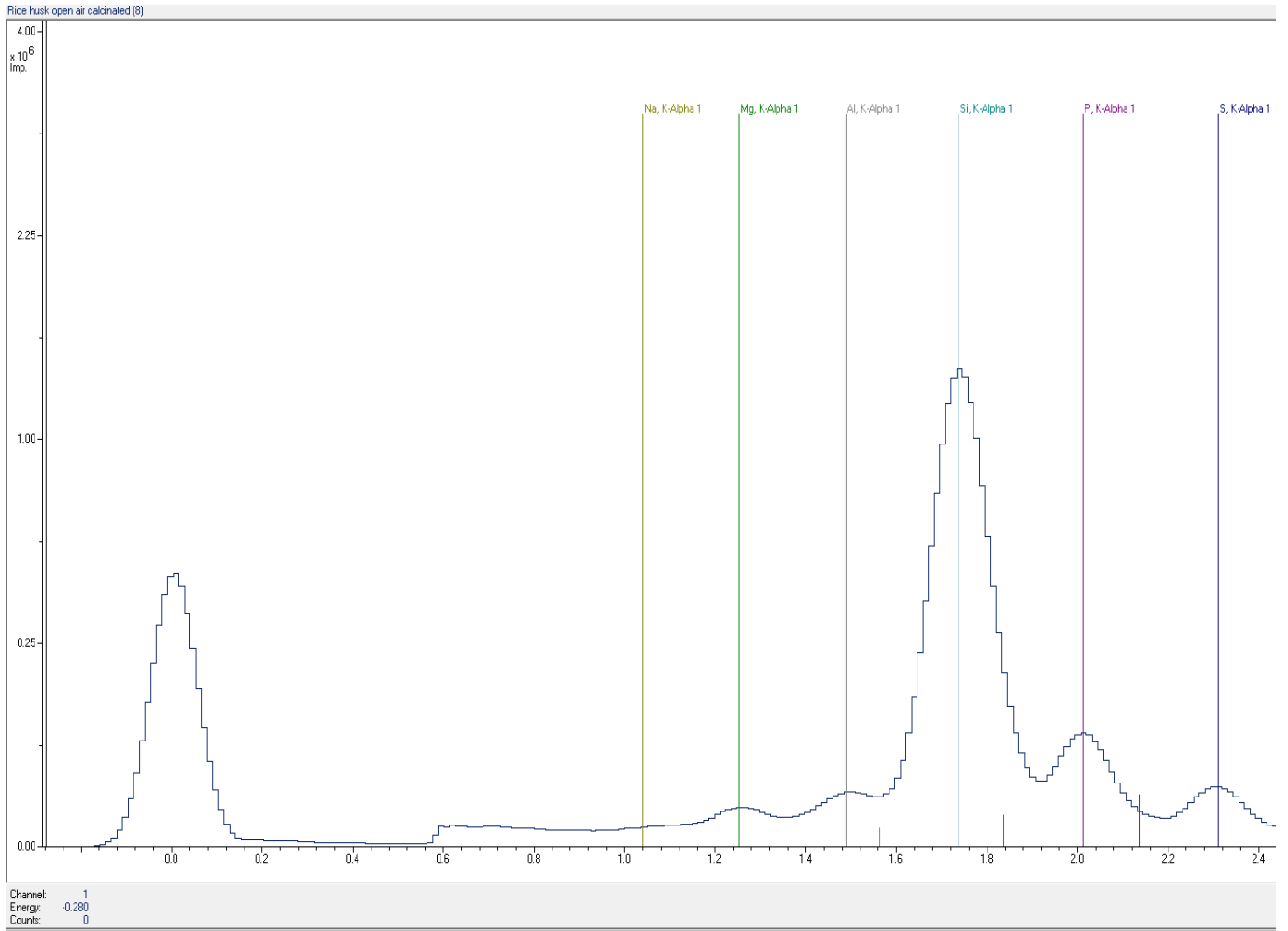


Appendix I
Grading curve of SDA



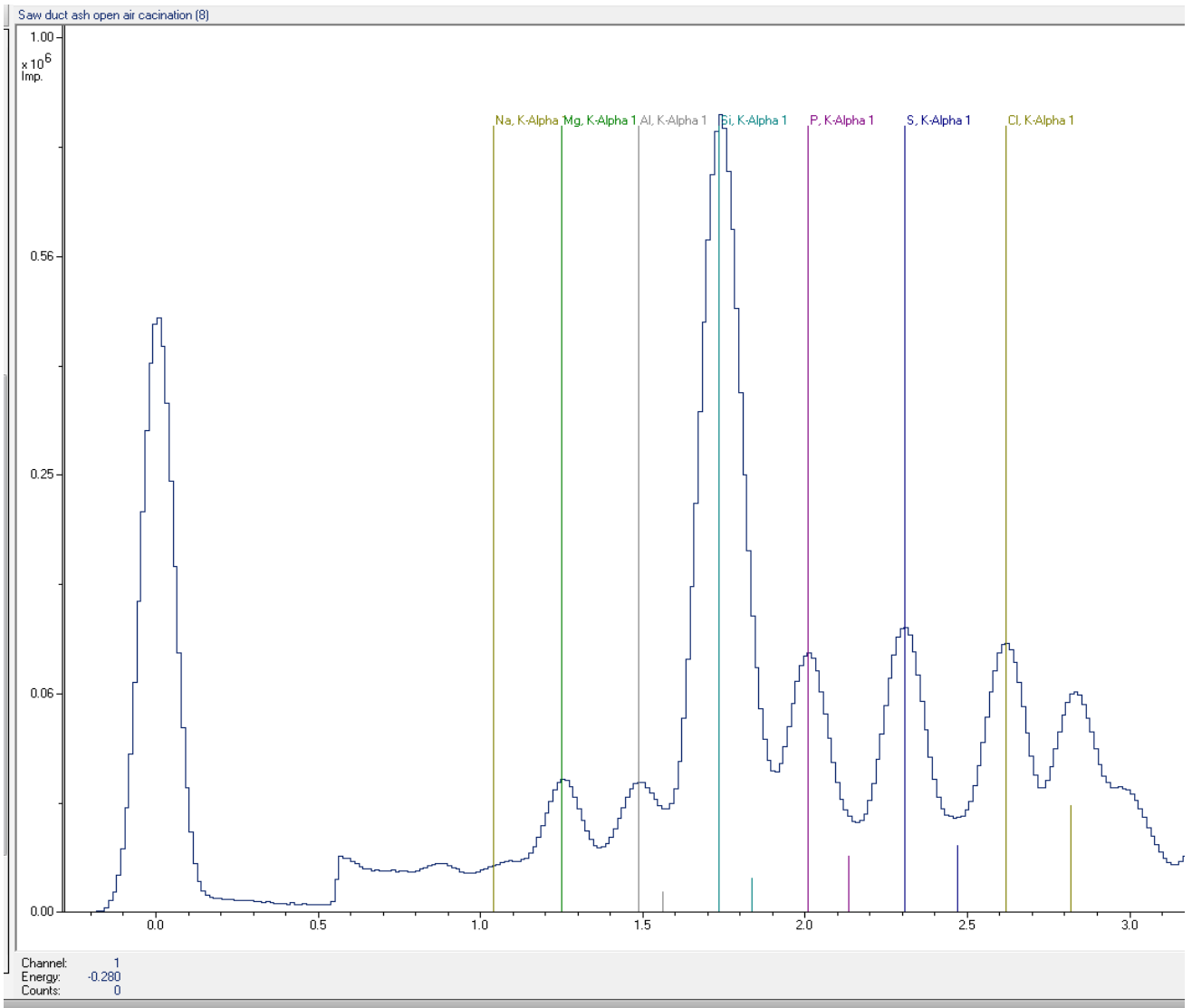
Appendix J

XRD Pattern of RHA



Appendix K

XRD Pattern of SDA



Appendix L

Slump Values of freshly mixed RHA, SDA and RHA-SDA concrete

DESCRIPTION		RHA			SDA			RHA/SDA		
		0%	5%	10%	15%	5%	10%	15%	5%	10%
Height of slump cone (mm)	300	300	300	300	300	300	300	300	300	300
Height of concrete after removal (mm)	281	285	287	287	284	282	283	284	285	286
Slump (mm)	19	15	13	13	16	18	17	16	15	14

Appendix M

Statistical F and T-test for RHA-OPC concrete

F-Test Two-Sample for Variances

	<i>0.88</i>	<i>0.888837</i>
Mean	1.319091	1.318288
Variance	0.151969	0.144811
Observations	11	11
df	10	10
F	1.049434	
P(F<=f) one-tail	0.470343	
F Critical one-tail	2.978237	

t-Test: Two-Sample Assuming Unequal Variances

	<i>Variable 1</i>	<i>Variable 2</i>
Mean	1.2825	1.2825
Variance	0.15422	0.147015
Observations	12	12
Hypothesized Mean Difference	0	
df	22	
t Stat	0	
P(T<=t) one-tail	0.5	
t Critical one-tail	1.717144	
P(T<=t) two-tail	1	
t Critical two-tail	2.073873	

Appendix N

Statistical F and T-test for SDA-OPC concrete

F-Test Two-Sample for Variances

	<i>0.88</i>	<i>0.865621</i>
Mean	1.176364	1.177671
Variance	0.151445	0.146932
Observations	11	11
df	10	10
F	1.030721	
P(F<=f) one-tail	0.481391	
F Critical one-tail	2.978237	

t-Test: Two-Sample Assuming Unequal Variances

	<i>Variable 1</i>	<i>Variable 2</i>
Mean	1.151667	1.151667
Variance	0.14499	0.141689
Observations	12	12
Hypothesized Mean Difference	0	
df	22	
t Stat	-1.4E-15	
P(T<=t) one-tail	0.5	
t Critical one-tail	1.71714	
P(T<=t) two-tail	1	
t Critical two-tail	2.07387	

Appendix O

Statistical F and T-test for RHA-SDA-OPC concrete

F-Test Two-Sample for Variances

	<i>0.88</i>	<i>0.860563</i>
Mean	1.237273	1.23904
Variance	0.169602	0.158008
Observations	11	11
df	10	10
F	1.073376	
P(F<=f) one-tail	0.456527	
F Critical one-tail	2.978237	

t-Test: Two-Sample Assuming Unequal Variances

	<i>Variable 1</i>	<i>Variable 2</i>
Mean	1.2075	1.2075
Variance	0.16482	0.155581
Observations	12	12
Hypothesized Mean Difference	0	
df	22	
t Stat	0	
P(T<=t) one-tail	0.5	
t Critical one-tail	1.71714	
P(T<=t) two-tail	1	
t Critical two-tail	2.07387	
	3	

Appendix P

Regression statistics of RHA-OPC Concrete

SUMMARY OUTPUT

<i>Regression Statistics</i>	
Multiple R	0.976359482
R Square	0.953277839
Adjusted R Square	0.942895136
Standard Error	0.093844222
Observations	12

ANOVA

	<i>df</i>	<i>SS</i>	<i>MS</i>	<i>F</i>	<i>Significance F</i>
Regression	2	1.617164	0.808582	91.81404	1.03E-06
Residual	9	0.079261	0.008807		
Total	11	1.696425			

	<i>Coefficients</i>	<i>Standard Error</i>	<i>t Stat</i>	<i>P-value</i>	<i>Lower 95%</i>	<i>Upper 95%</i>	<i>Lower 95.0%</i>	<i>Upper 95.0%</i>
Intercept	0.683783769	0.066451	10.29012	2.82E-06	0.533462	0.834105	0.533462	0.834105
Curing Age (Days)	0.007323316	0.000544	13.46463	2.87E-07	0.006093	0.008554	0.006093	0.008554
Replacement Level (%)	-0.0074	0.004846	-1.527	0.161106	-0.01836	0.003563	-0.01836	0.003563

Appendix Q

Regression statistics of SDA-OPC Concrete

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.988526261
R Square	0.977184169
Adjusted R Square	0.972113985
Standard Error	0.063587638
Observations	12

ANOVA

	df	SS	MS	F	Significance F
Regression	2	1.558576	0.779288	192.7315	4.09E-08
Residual	9	0.03639	0.004043		
Total	11	1.594967			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	0.671578825	0.045026	14.91534	1.18E-07	0.569723	0.773435	0.569723	0.77343479
Curing Age (Days)	0.006930088	0.000369	18.80444	1.56E-08	0.006096	0.007764	0.006096	0.00776377
Replacement Level (%)	-0.018533333	0.003284	-5.64412	0.000316	-0.02596	-0.01111	-0.02596	-0.0111052

Appendix R

Regression statistics of RHA-SDA-OPC Concrete

SUMMARY
OUTPUT

<i>Regression Statistics</i>	
Multiple R	0.971565667
R Square	0.943939846
Adjusted R Square	0.931482034
Standard Error	0.106269292
Observations	12

ANOVA

	<i>df</i>	<i>SS</i>	<i>MS</i>	<i>F</i>	<i>Significance F</i>
Regression	2	1.711387	0.855693	75.77092	2.34E-06
Residual	9	0.101638	0.011293		
Total	11	1.813025			

	<i>Coefficients</i>	<i>Standard Error</i>	<i>t Stat</i>	<i>P-value</i>	<i>Lower 95%</i>	<i>Upper 95%</i>	<i>Lower 95.0%</i>	<i>Upper 95.0%</i>
Intercept	0.653101935	0.075249	8.679249	1.15E-05	0.482878	0.823326	0.482878	0.82332624
Curing Age (Days)	0.007409307	0.000616	12.02996	7.54E-07	0.006016	0.008803	0.006016	0.00880258
Replacement Level (%)	-	0.005488	-2.61189	0.028183	-0.02675	-0.00192	-0.02675	-0.0019192