STATISTICAL MODELS FOR PREDICTING COMPRESSIVE STRENGTH AND DENSITY OF SANDSTONE CONCRETE

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CERTIFICATION

This is to certify that this work, **"Statistical Models for Predicting Compressive Strength and Density of Sandstone Concrete"** was carried out by **Emeka Kingsley Ejiogu** (B.Eng) with Reg. No. **20134872798**, 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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DEDICATION

To the Almighty God for His infinite mercies, and in loving memory of my mother, Late Mrs. Helen Chioma Ejiogu.

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ABSTRACT

This thesis developed statistical models for predicting compressive strength and density of sandstone concrete. The materials used in the laboratory experiment are water, Ordinary Portland Cement, river sand and sandstone. Forty-four (44) mix ratios were developed from eleven (11) solid mix ratios and four water-cement ratios (0.45, 0.5, 0.55, and 0.6). A total of one hundred and thirty-two (132) concrete cubes were cast for compressive strength test, comprising of three cubes per mix ratio. The Saturated surface dry (SSD) densities of the concrete cubes were determined. Using Ibearugbulem's regression method, the first twenty-two (22) mix ratios were used to determine the coefficients of the regression while the remaining twenty-two (22) mix ratios were used to validate the models. The results show that, optimum compressive strength of the sandstone concrete obtained from experiment at 28 days is 24.18 N/mm^2 and corresponds to 0.6:1:2.5:3.5 mix ratio (for water, cement, sand and sandstone respectively) while the optimum value predicted by the statistical model is 22.55 N/mm^2 corresponding to the same mix ratio. The maximum SSD density of the sandstone concrete obtained from experiment at 28 days is 2592.59Kg/m^3 and corresponds to 0.6:1:1.75:4.25 mix ratio while the statistical model predicted maximum value of 2567.7Kg/m^3 for the same mix ratio. The compressive strength was found to increase as the water/cement ratio increases and fine/coarse aggregate ratio decreases. Also, high water/cement ratio and low fine/coarse aggregate ratio resulted in high densities. The results from the models compared favorably with the corresponding experimental results. Predictions from the models were tested with the statistical Fisher test and found to be reliable at 95% confidence level. With the models developed, any desired compressive strength or SSD density of sandstone concrete can be predicted if the mix proportions are known and vice versa.

Keywords: Sandstone Concrete, Statistical Models, Prediction, Compressive Strength, Saturated Surface Dry (SSD) Density.

CHAPTER ONE

INTRODUCTION

1.1 Background of Study

Concrete is presently the most common building material in Nigeria, hence careful consideration must be given to factors that affect its strength. Compressive strength of concrete is one of the most important and useful properties. As a construction material, concrete is employed to resist compressive stresses and its failure is usually a result of many factors including improper mix ratio and ignorance in the use of locally available coarse aggregates (Babatunde and Opawole, 2009).

Optimization of the concrete mixture is a process of search for a mixture for which the sum of the costs of the ingredients is lowest, yet satisfying the required performance of concrete, such as workability strength and durability (Shakhmenko and Birsh, 1998). The basic ingredients of concrete can be classified into two groups: cement paste and aggregates. Although the quality of cement paste is governed mainly by the water/cement ratio, to achieve a targeted quality of concrete depends on the characteristics of aggregates. According to Glavind and Munch-Petersen (2000), these characteristics mainly include surface area and voids in aggregates. While surface area is governed by the shape and maximum size of aggregates, the void content is affected mainly by the particle size distribution of aggregates and can be reduced by increasing the aggregate/cement ratio (He et al., 2012). Goltermann et al. (1997) suggested a packing model for the aggregate selection and combination to obtain aggregate mixes having the lowest void contents with maximum packing degree (the ratio between bulk density and the aggregate grain density). Thus, the packing degree according to them is a characteristic of the specific aggregate type or mix and it indicates the void volume and the amount of cement paste necessary in the concrete. This indicates that a concrete mixture can be optimized by adjusting the levels of the key mixture factors such as water to cement ratio, coarse aggregate to total aggregate ratio, and cement content or aggregate to cement ratio.

The type and source of the coarse aggregate has a considerable influence on the compressive strength of concrete. Other factors such as the type of fine aggregate, the maximum size of aggregate, the overall grading, and particle shape and surface texture, have little direct effect on the compressive strength (Teychenné et al., 1997). The combined effect of the aggregate characteristics and its varying proportions does have effect on compressive strength through its influence on the water requirement to produce a workable concrete yet, mixes with appropriate water content that shall fulfill stated demands in fresh, young and hardened phase have to be achieved.

Attempts have been made in the past to optimize concrete mixtures using the fully experimental, fully analytical, semi-experimental, or statistical methods. Fully experimental methods involve an extensive series of tests, sometimes conducted on a trial-and-error basis, and the optimization results are often applicable only to a narrow range of local materials (Ahmed, 2007). Analytical methods help in searching for an optimum concrete mixture based on detailed knowledge of specific weights of mixture components and on certain basic formulas, which result from previous experience without conducting expensive and time-consuming experimental works (Sonebi and Bassuoni, 2013). Semi-experimental methods are based on combining the experimental database or experimentally developed prediction models and various analytical tools such as artificial neural network, genetic algorithm, and mathematical programming (Lee et al., 2009; Yeh, 2009).

Statistical and empirical methods are an improvement over fully experimental methods, in which, instead of selecting one starting mix proportion and then adjusting by trial and error for achieving the optimum solution, a set of trial batches covering a chosen range of proportions for each mixture component is defined according to established statistical procedures. Trial batches are then carried out, test specimens are fabricated and tested, and experimental results are analyzed using standard statistical methods (Ezeh and Ibearugbulem, 2009). These methods include fitting empirical models to the data for each performance criterion. In these models, each response (resultant concrete property) such as strength, slump, or cost is expressed as an algebraic function of factors such as w/c, cement content, fine/coarse aggregate content. After a response can be characterized by an equation (model), several analyses are possible. A user could determine which mixture proportions would yield one or more desired properties. A user also could optimize any property subject to constraints on other properties.

Various models have been formulated and developed for optimizing concrete made with conventional aggregates such as granite, gravel, granite-gravel, etc. This shows there is over-reliance on more conventional materials to achieve target strength and also due to the ignorance in the use of locally sourced coarse aggregates such as sandstone, in concrete construction. Sandstone is locally sourced, manually crushed and graded, and can be found in most parts of Nigeria. The need to optimize the compressive strength and density of concrete made with locally sourced coarse aggregate while determining the combination of materials that would give the highest compressive strength and the effect of water-cement and fine-coarse aggregate ratio gave birth to the topic of this research work “statistical models for predicting compressive strength and density of sandstone concrete.

In this experimental research, the Ibearugbulem’s regression method was adopted in the optimization of compressive strength and density of sandstone concrete. Effort has been made to exhibit the application of the statistical approach proposed to obtain optimum proportioning of sandstone concrete mixtures using the data obtained through an experiment considering water-cement ratio, and fine aggregate to coarse aggregate ratio as design factors. The experimental data were analyzed statistically and mathematical polynomials regression was developed for concrete strength/density as a function of mixture variables. The models could predict the compressive strength and density of sandstone concrete when mix ratios are given and vice versa. The utility of the developed compressive strength model in optimizing the mixture was tested using statistical Fisher test.

1.2 Statement of Problem

Professional engineers in the construction industry constantly face the problem of concrete mix failures in most civil engineering projects; these problems surely are occasioned by many factors, among them are bad coarse aggregate, improper mix ratio and the inability to get building materials at affordable prices which may cause negligence in the selection of correct ingredients for mixing, to achieve a suitable mix, and obtain a technically sound execution of concrete works. According to Sofia (2008), the difficulty with production of concrete is to succeed with the concrete within reasonable economical frames and to reach repeatable properties through the whole production chain, i.e. sufficient robustness.

The construction industry relies heavily on conventional coarse aggregate materials such as granite for the production of concrete (Onwuka et al., 2013). This have in turn streamlined researches as most researchers often choose granite as coarse aggregate for experimental works. As a result, the average man, rural dwellers and even local contractors have little or no knowledge of sandstone, which is chiefly available and locally sourced, in production of concrete to meet desired strength.

Many researches have been carried out in area of optimization and prediction of compressive strength of concrete using various statistical methods (Onwuka et al., 20011; Osadebe, 2003; Osadebe & Ibearubulem, 2008; Ibearugbulem et al., 2013 and 2014) but none investigated the relationship between the density and compressive strength. There is no model for prediction of the density of concrete cubes using the mixture proportions as most of the researchers stopped at predicting concrete strength. Most of the developed models are limited to concrete made with granite or gravel.

This experimental research therefore, is geared towards investigating the effects of the aggregate (sandstone) properties, its proportion and water-cement ratio on compressive strength and density of sandstone concrete, and to develop statistical models for sandstone concrete that can prescribe compressive strength and density, when the desired mixture proportion is known and vice-versa thereby, determining the combination of the materials that would give the highest compressive strength.

1.3 Objectives of Study

The main objective of this study is to develop statistical models for predicting compressive strength and SSD density of sandstone concrete. The specific objectives are:

i. To determine the adequacy of using sandstone as coarse aggregate material for concrete production through physical and mechanical characterization tests.

ii. To determine the mixture proportion that will give the highest compressive strength and the effects of water/cement ratio and fine/coarse aggregate ratio on compressive strength of sandstone concrete.

iii. To determine the effects of water/cement ratio and fine/coarse aggregate ratio on compressive strength and density of sandstone concrete.

iv. To develop statistical models that can prescribe compressive strength and SSD density of sandstone concrete when the mix ratio is known.

v. To test the reliability of the formulated models in predicting the compressive strength and SSD density of sandstone concrete.

1.4 Justification of Study

This research will provide a resource material for civil engineering students, concrete production companies and practicing engineers. The findings will have an impact on the concrete industry because the use of sandstone in concrete production will not only reduce cost but also, may lead to more sustainable mixes for concrete construction. Models that make prescription of compressive strength and SSD density easy when a desired mix ratio is known will help save enormous time and effort invested in carrying out trial mixes.

1.5 Scope of Study

The materials used in this research work were limited to water, Ordinary Portland cement, river sand and sandstone. The tests are limited to physical and mechanical characterization of aggregates, the compressive strength of prototype concrete cubes. The strength is based on 28 days curing. Curing was by full immersion in water. Statistical regression method of analysis based on Ibearugbulem's method for predicting and optimizing concrete was employed.

CHAPTER TWO

LITERATURE REVIEW

2.1 Concrete as a Structural Material

According to Mehta and Monteiro (2013), concrete is a composite material that consists essentially of a binding medium within which are embedded particles or fragments of aggregate. Often additives and reinforcements (such as rebar) are included in the mixture to achieve the desired physical properties of the finished material. When these ingredients are mixed together, they form a fluid mass that is easily molded into shape. Over time, the cement forms a hard matrix which binds the rest of the ingredients together into a durable atone-like material with many uses (Zongjin, 2011).

Concrete is neither as strong nor as tough as steel but is the most widely used engineering material. According to Mehta and Monteiro (2013), it is estimated that the present consumption of concrete in the world is of the other of 11 billion metric tonnes every year). Shilstone et al. (2002) discussed specifications and/or methods that will contribute to production of the desired levels of engineering, construction, and durability properties required for economy and long-term serviceability of the concrete project. John and Ban (2003) confine the factors that govern the strength of concrete mixes as; aggregate properties, cement paste properties, properties of the transition zone between cement paste and the aggregate and relative proportion of constituent materials.

Neville (2005); Mehta and Monteiro (2013) identified the following primary reasons for the preference for concrete as an engineering material:

i. Resistance to water: Concrete possesses excellent resistance to water. Unlike wood and ordinary steel, the ability of concrete to withstand the action of water without serious deterioration makes it an ideal material for building structures to control, store, and transport water. The use of plain concrete for dams, canal linings, and pavements is now a common sight almost everywhere in the world.

ii. Ability to be cast to any desired shape and configuration: The secondly reason for the widespread use of concrete is the ease with which structural concrete elements can be formed into a variety of shapes and sizes. This is because freshly made concrete is of plastic consistency, which enables the material to flow into prefabricated formwork. After a number of hours when concrete has solidified and hardened to a strong mass, the formwork can be removed for reuse.

iii. Economical: Concrete is popular with engineers because it is usually the cheapest and most readily available material on the job. The principal components for making concrete, namely aggregate, water, and Portland cement are relatively inexpensive and are commonly available in most part of the world.

iv. Maintenance: Concrete does not corrode, needs no surface treatment, and its strength increases with time; therefore, concrete structures require much less maintenance. Steel structures, on the other hand, are susceptible to rather heavy corrosion in offshore environments, require costly surface treatment and other methods of protection, and entail considerable maintenance and repair costs.

v. Fire resistance: The fire resistance of concrete is perhaps the most important single aspect of offshore safety and, at the same time, the area in which the advantages of concrete are most evident. Since an adequate cover on reinforcement or tendons is required for structural integrity in reinforced and prestressed concrete structures, the protection against failure due to excessive heat is provided at thee same time.

vi. Resistance to cyclic loading: The fatigue strength of steel structures is greatly influenced by local stress fields in welded joints, corrosion pitting, and sudden changes in geometry, such as from thin web to thick frame connections. In most code of practice, the allowable concrete stresses are limited to about 50% of the ultimate strength: thus the fatigue strength of concrete is generally not a problem.

Other advantages include: high compression taking member, energy efficient because of its thermal properties, on-site fabrication, and aesthetic properties. All of these advantages combine to make concrete very versatile and adaptable. However, ordinary concrete made with natural aggregate has low strength-weight ratio compared to steel. This places concrete at an economic disadvantage when designing structural members for tall buildings.

2.2 Types of Concrete

According to Mehta and Monteiro (2013), concrete can be classified into three broad categories. Concrete containing natural sand and gravel or crushed-rock aggregates, generally weighing about 2400 Kg/m^3 is called *normal-weight concrete* and it is the most commonly used concrete for structural purposes. For applications where a higher strength-to-weight ratio is desired, it is possible to reduce the unit weight of concrete by using natural or pyro-processed aggregates with lower bulk density. The term *lightweight concrete* is used for concrete that weighs less than about 1800 Kg/m^3. Concrete produced from high-density aggregates and generally weighs more than 3200 Kg/m^3 are called *heavyweight concrete*. They are used for radiation shielding.

They went ahead to divide concrete into three general categories based on compressive strength:

- Low-strength concrete: less than 20 MPa
- Moderate-strength concrete: 20 to 40 MPa
- High-strength concrete: more than 40 MPa.

Moderate-strength concrete, also referred to as ordinary or normal concrete, is used for most structural work. High-strength concrete is used for special applications. Typical proportions of materials for producing low-strength, moderate-strength, and high-strength concrete mixtures with normal weight aggregate are shown in Table 2.1

Table 2.1 Typical Proportions of Materials in Concrete Mixtures of Different Strength.

Material	Low-strength (Kg/m^3)	Moderate-strength (Kg/m^3)	High-strength (Kg/m^3)
Cement	255	356	510
Water	178	178	178
Fine aggregate	801	848	890
Coarse aggregate	1169	1032	872

Cement paste proportion			
percent by mass	18	22.1	28.1
percent by volume	26	29.3	34.3
Water/cement by mass	0.70	0.50	0.35
Strength, MPa	18	30	60

Source: Mehta and Monteiro (2013).

Conventional Portland-cement concrete mixtures suffer from certain deficiencies. Attempts to overcome these deficiencies have resulted in the development of special concrete types that are described in the following sections.

2.2.1 Structural Light Weight Concrete

This is a structural lightweight concrete in every respect except that, for reasons of overall cost economy, the concrete is made with a cellular lightweight aggregate so that its unit weight is approximately two-third of the unit weight of concrete made with typical natural aggregate (Mehta and Monteiro, 2013). Because the density of concrete, not the strength, is the primary objective, the specifications limit the maximum permissible unit weight of concrete. Also, because highly porous aggregates tend to reduce the concrete strength greatly, the specifications require a minimum of 28-day compressive strength to ensure that the concrete is of structural quality.

ACI 213R-99 (2002), defines structural lightweight aggregate concrete as those having a 28-day compressive strength in excess of 17 MPa and a 28-day, air-dried unit weight not exceeding 1850 Kg/m3. The concrete may consist entirely of lightweight aggregates or, a combination of lightweight and normal weirgt aggregates. In consideration to workability and other properties, it is common practice to use normal-weight sand as fine aggregate, and limit the nominal size of lightweight aggregate to a maximum of 19 mm.

Dean Frank (2003) in his own study said that before lightweight aggregates are used in a concrete mixture, it will be wise to wet them for a period of not less than 24 hours to the time of mixing. This according to him will enable the aggregate particle not to segregate during handling. To this effect he concluded by saying that it is a wrong practice to batch lightweight aggregates directly in a concrete mixture. If this is done the aggregate particles will continue absorbing the free water from the concrete mixture to a point it will cause the mixture to segregate or stiffen before placement.

2.2.2 High Strength Concrete

Although high-strength concrete is often considered a relatively new material, its development has been gradual over many years. As the development has continued, the definition of high-strength concrete has changed as it is solely on the basis of compressive strength at a given age. In the 1950s, concrete with a compressive strength of 35 MPa was considered high strength. More recently, compressive strengths approaching 138 MPa have been used in cast-in-place buildings. In recent years, the applications of high-strength concrete have increased, and high-strength concrete has now been used in many parts of the world. The growth has been possible as a result of recent developments in material technology and a demand for higher strength concrete.

According to ACI Committee on High Strength Concrete,*" Concrete that has a specified compressive strength for design of 55MPa or more is known as High Strength Concrete*. However, in regions where the upper limit on commercially available material is currently 35 MPa concrete, 60 MPa concrete is considered as High Strength Concrete.

Selection of material, concrete mix proportioning, batching, mixing, transporting, placing, and control procedures are applicable across a wide range of concrete strengths. Ordinary Portland cement of any type meeting ASTM C 150 (2002), Standard Specification can be used to obtain concrete mixtures with compressive strengths up to 55 MPa. To obtain a higher strength while maintaining good workability, it is necessary to use chemical and mineral admixtures in combination with cement (Mehta and Monteiro, 2013).

High-strength, lightweight aggregate concrete with compressive strengths up to 50 MPa, can be commercially produced with high-quality lightweight aggregates.

2.2.3 Self Consolidating Concrete

Self-consolidating concrete (SCC) can be defined as a flowing concrete that can be cast into place without the use of vibrators to form a product free of honeycombs and bug holes (Mehta and Monteiro, 2013).The constructability of highly congested reinforced concrete elements requires the fresh concrete mixtures to be very fluid. Slump values on the order of 200 to 250 mm, has been achieved without the use of too much water, made possible by the advent of superplasticizers. The introduction of “modern” self-leveling concrete or self-consolidating concrete (SCC) is associated with the drive towards better quality concrete pursued in Japan around 1983, where the lack of uniform and complete compaction had been identified as the primary factor responsible for poor performance of concrete structures” (Dehn et al., 2000).

Self-compacting concrete can be obtained in such a way, by adding chemical and mineral admixtures, so that its splitting tensile and compressive strengths are higher than those of normal vibrated concrete. Yet, the use of flowing concrete mixtures presents the risk of bleeding, segregation, and settlement, which, by weakening the interfacial transition zone between the cement paste and aggregate (including the reinforcement steel), would have an adverse effect on the mechanical properties as well as on the durability of concrete.

Self-consolidating concrete was developed to solve the problem of how to produce concrete mixtures possessing a high workability, that is, high in fluidity and high cohesiveness simultaneously. Fresh concrete properties of self-compacting concrete containing saw-dust ash were investigated by Elinwa et al. (2008). A suitable acceptance test method for self-compactability has been developed by Ouchi et al (2000). If the concrete flows through the apparatus, the concrete is considered as self-compactable for the structure. If the concrete is stopped by the apparatus, the concrete is considered as having insufficient self-compactability and the mix-proportion has to be adjusted.

Druta (2003) pointed out that self-compacting concrete has two big advantages. One relates to the construction time, which in most of the cases is shorter than the time when normal concrete is used, due to the fact that no time is wasted with the compaction through vibration. The second advantage is related to the placing. As long as SCC does not require compaction, it can be considered environmentally friendly, because if no vibration is applied no noise is made.

2.2.4 High Performance Concrete

High performance concrete according to ACI is defined as a concrete meeting special combination of performance and uniformity requirements that cannot always be achieved routinely using conventional constituents and normal mixing, placing, and curing practices. This definition was developed by ACI's Technical Activities Committee task group and approved in1998. This is a different approach to the suggestion by Mehta and Aitcin (1990). They suggested the term "high-performance concrete" (HPC) for concrete mixtures that possess the following three properties: high-workability, high-strength, and high durability. This has resulted in a situation that many of the concrete mixtures that are being marketed today as HPC may not prove to be durable under demanding environmental conditions.

According to the commentary that followed the ACI definition, a high performance concrete is a concrete in which certain characteristics are developed for a particular application and environment. Examples of characteristics that may be considered critical for particular application are:

i. Ease of placement
ii. Compaction without segregation
iii. Early strength
iv. Long-term strength and mechanical properties
v. Permeability
vi. Density
vii. Heat of hydration
viii. Toughness
ix. Volume stability
x. Long life in severe environment.

High performance concrete (HPC) is known as high technology construction material, proving to be very effective, reliable, and having long-term durability in natural environment. The study of HPC has been an extremely active research area in recent years. Broadly speaking, HPC is defined as “concrete made with appropriate materials combined according to a selected mix design and properly mixed, transported, placed, consolidated, and cured so that the resulting concrete will give excellent performance in the structure in which it will be exposed, and with the loads to which it will be subjected for its design life” (Forster, 1994).

In his study of on “high performance concrete using sandstone aggregates”, Paramasivam (2006) stated that, “with the development of HPC, the strength and durability of concrete has largely improved. Although HPC offers many advantages, due to the restriction of the manufacturing process and the availability of quality raw materials, the development of HPC is still limited”. He used a suitable mix to develop HSC using crushed sandstone coarse and fine aggregates. Three types of curing conditions were employed to investigate the effect of curing on strength and durability. It was found that, a combination of silica fume and fly ash as partial replacement of cement with crush sandstone aggregates offers synergistic effect on workability, strength, and durability. He concludes that sandstone aggregates can be used in HSC production”.

According to Neville (1997), the aggregate with low modulus of elasticity is more beneficial with respect to HPC. In high rise buildings and bridges, the stiffness of the structure is of interest to the structural engineers. On certain projects the minimum modulus of elasticity is affected significantly by the properties of aggregate and mix proportion (Baalbakki et al., 1991; Kosmatka et al., 2002).

2.2.5 Fiber Reinforced Concrete

Because conventional Portland-cement concrete mixtures suffer from certain deficiencies, attempts to overcome these deficiencies have resulted in the development of special concrete types. The deficiency of poor impact resistance has been substantially improved by using the concept of microlevel reinforcement. *Fiber-reinforced concrete mixtures* containing steel, glass, or polypropylene fibers are being employed successfully in the structures where resistance to impact is important; however, for most structural and non-structural purposes, steel fiber is most commonly used of all the fibers.

Originally, it was assumed that tensile as well as flexural strengths of concrete can be substantially increased by introducing closely spaced fibers that would obstruct the propagation of micro-cracks, therefore delaying the onset of tension cracks and increasing the tensile strength of the material. But years of experimental research studies showed that with the volumes and sizes of fibers that could conveniently be incorporated into conventional mortars or concretes, the fiber-reinforced products did not offer a substantial improvement in strength over corresponding mixtures without fibers (Mehta and Monteiro, 2013).

According to Ward-Waller (2004), steel was developed in the 19th century as a stronger and more ductile alternative to iron. It has been used to reinforced concrete since nearly its advent as a modern construction material, and is manufactured in the form of bars, plates, wire, and mesh. Other fibrous concrete has been used in structures for a long time but the use is still low in comparison to conventional reinforcement bars. However, the advantage steel provides is the ability to bond with cement mortar matrix of the concrete (Karl and Tim, 2013. With regard to typical use of steel-fiber-reinforced concrete, ACI 544.1R-96 aptly summarizes the position thus:

"Generally, for flexural structural components, steel fibers should be used in conjunction with properly designed continuous reinforcement. Steel fibers can reliably confine cracking and improve resistance to material deterioration as a result of fatigue, impact, and shrinkage or thermal loads. In applications where the presence of continuous tensile reinforcement is not essential to the safety and integrity of the structure, such as floors on grade, pavements, overlays, and shotcreting linings, the improvements in flexural strength, impact resistance, toughness, and fatigue performance associated with fibers can be used to reduce section thickness, improve performance, or both."

Finally, Mehta and Monteiro (2013) opined that the cost of mixing, transporting, and placing concrete does not change by fiber incorporation, the difference in cost between in-place fibrous concrete and plain concrete will not be large. Also, compared to plain concrete, since the thickness of fibrous concrete slabs designed for a given load can be substantially reduced, the overall difference in the first cost may turn out to be negligible. Considering the service life, therefore, fibrous concrete would appear to be cost-effective.

2.2.6 Other Breeds of Concrete

Concrete mixtures containing polymers have been developed which show very low permeability and excellent chemical resistance where imperviousness is desired for durability to strong chemical solutions. Overlays composed of such concrete mixtures are suitable for protection of reinforced steel from corrosion in industrial floors and bridge decks. It may also contain pozzolans and other admixtures commonly used with conventional concrete. Zongjin (2011) classified concretes containing polymers into three categories: polymer concrete (PC) formed by polymerizing a mixture of a monomer and aggregate without other bonding material; latex-modified concrete (LMC) is usually made by replacing a part of the mixing water with latex (polymer emulsion); and polymer impregnated concrete (PIC) produced by infiltrating a hardened Portland cement concrete with a monomer and subsequent polymerizing the monomer in-situ.

To counteract the problem concrete cracking due to restrained shrinkage on drying, which has been recognized in the design and construction practice of relatively thin structural elements such as floor pavement slabs, *shrinkage-compensating concrete* containing expansive cements or cements additives were developed about 35 years ago. *Heavyweight concrete* made with high-density minerals is about 50 percent heavier than normal concrete containing conventional aggregate. The concrete unit weights are in the range 3360 to 3840 Kg/m^3. This type of concrete is being used for radiation shielding in nuclear power plants, medical units, and atomic research and testing facilities when limitations of usable space require reduction in the thickness of the shield. Other materials can be employed for this purpose, but concrete is usually the most economical and has several other advantages.

Mass concrete for dams and other structures has been around for some time, but methods selected to control the temperature rise have had a considerably influence on the construction technology during the last 40 years. As defined by ACI Committee 116R-00, mass concrete are concrete in a massive structure, for example, a beam, columns, pier, lock, or dam where its volume is of such magnitude as to require special means of coping with the generation of heat and subsequent volume change. Precooling of concrete materials has virtually eliminated the need for expensive postcooling operations and has made faster construction schedules possible. Dams are also built now with roller-compacted concrete, using ordinary earth-moving equipment, at speeds and cost that were unimaginable only 25 years ago.

2.3 Constituent Materials of Concrete

In its simplest form, concrete is a mixture of paste and aggregates. The paste, composed of Portland cement and water, coats the surface of the fine and coarse aggregates. Through a chemical reaction called hydration, the paste hardens and gains strength to form the rock-like mass known as concrete. Normal weight concrete materials include cement, water, sand and coarse aggregate as can be seen in Figure 2.1. Sand (fine aggregate) in this case is river sand while coarse aggregate is sandstone.

2.3.1. Mixing Water

Water is always available in its natural states, as rainwater, river water, fresh seawater, and borehole water. Water is an important ingredient because,

- It actively participates in the reaction with cement, a process known as cement hydration
- It ensures workability

The time-honored rule of thumb for water quality is, "If you can drink it you can make concrete with it", and a large fraction of concrete is made using municipal water supply.

However, good quality concrete can be made with water that would pass normal standards for drinking water. ASTM 1602 (2004) gave specifications for mixing water used in production of hydraulic cement concrete.

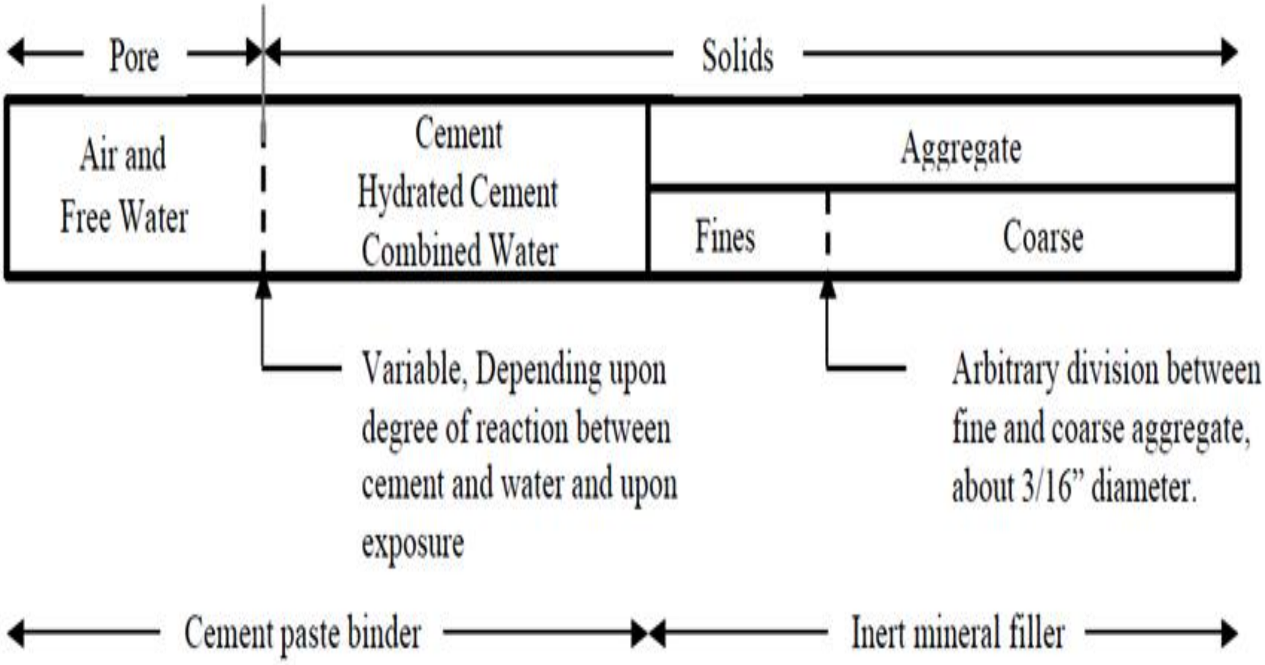


Figure 2.1 Ingredients of Concrete
Source: Saad (2005).

Impurities in water affect the following concrete qualities:

i. Setting time
ii. Drying shrinkage
iii. Resistance to efflorescence
iv. Durability.

Impurities that make water unsuitable for use are discussed below:

Suspended Solids: Suspended clay/silt increase water demand, increase drying shrinkage, cause efflorescence. Algae/suspended organic matter retard setting, reduced strength, interfere with cement hydration, entrain excessive amount of water.

Dissolved Solids: Hazards depend upon the nature of the dissolved materials. For example – soluble carbonates and bicarbonates, soluble inorganic salts of zinc, copper lead etc., acidic water, organic acids, alkaline water (NaOH/ KOH).

Dissolved Organic Material: Dissolved organic materials which are mainly tannic and humid acids (make water colored) may retard the hydration of cements or entrain excessive amounts of air.

2.3.2. Cement

Cement, in the general sense of the word, can be described as a material with adhesive and cohesive properties, which make it capable of bonding mineral fragments into a compact whole. This definition embraces a large variety of cementing materials (Mehta and Monteiro, 2013).

For constructional purposes, the meaning of the term cement is restricted to the bonding materials used with stones, sand, bricks, building blocks etc. Cement plays the central role in the concrete mix not by contributing volume but being primarily responsible for its strength. Among various cementing materials, the cements of interest in the making of concrete have the property of setting and hardening under water by virtue of chemical reactions with it and hence are known as hydraulic cements.

But the best known and most versatile type of artificial cement is the Ordinary Portland cement because of its resemblance when hardened to Portland stone found near Dorset, England. In the most general sense, Portland cement is produced by heating sources of lime, iron, silica, and alumina to clinkering temperature (2,500 to 2,800 degrees Fahrenheit) in a rotating kiln, then grinding the clinker to a fine powder. The heating that occurs in the kiln transforms the raw materials into new chemical compounds. Therefore, the chemical composition of the cement is defined by the mass percentages and composition of the raw sources of lime, iron, silica, and alumina as well as the temperature and duration of heating (Obla et al., 2003). It is this variation in raw materials source and the plant-specific characteristics, as well as the finishing processes (i.e. grinding and possible blending with gypsum, limestone, or supplementary cementing materials), that define the cement produced.

2.3.2.1 Standards

To ensure a level of consistency between cement-producing plants, certain chemical and physical limits are placed on cements. These chemical limits are defined by a variety of standards and specifications. For instance, Portland cements and blended hydraulic cements for concrete in the U.S. conform to the American Society for Testing and Materials (ASTM) C150 (Standard Specification for Portland cement), ASTM C595 (Standard Specification for Blended Hydraulic Cement) or ASTM C1157 (Performance Specification for Hydraulic Cements).

Some state agencies refer to very similar specifications: AASHTO M 85 for Portland cement and M 240 for blended cements. These specifications refer to standard test methods to assure that the testing is performed in the same manner. For example, ASTM C109 (Standard Test Method for Compressive Strength for Hydraulic Cement Mortars using 2-inch Cube Specimens), describes in detail how to fabricate and test mortar cubes for compressive strength testing in a standardized fashion.

2.3.2.2 Nomenclature Differences

Three separate standards may apply depending on the category of cement. For Portland cement types, ASTM C150 describes: Normal (I), Moderate Sulphate Resistance (II), Moderate Heat of Hydration (and Moderate Sulphate Resistance) (II (MH)), High Early Strength (III), Low Heat Hydration (IV) and High Sulphate Resistance (V).

2.3.2.3 Chemical Performance Requirements

Chemical tests verify the content and composition of cement, while physical testing demonstrates physical criteria. In ASTMC150/M 85 and ASTM C595/M 240, both chemical and physical properties are limited. In ASTM C1157, the limits are almost entirely physical requirements.

Chemical testing includes oxide analyses (SiO_2, CaO, Al_2O_3, Fe_2O_3, etc.) to allow the cement phase composition to be calculated. Type II cements are limited in ASTM C150/M 85 to a maximum of 8% by mass of tricalcium aluminate (a cement phase, often abbreviated C3A), which impacts a cement’s sulphate resistance. Certain oxides are also themselves limited by specifications: For example, the magnesia (MgO) content which is limited to 6 percent maximum by weight for Portland cements, because it can impact soundness at higher levels.

2.3.2.4 Typical physical requirements for cements

These are air content, fineness, expansion, strength, heat of hydration, and setting time. Most of these physical tests are carried out using mortar or paste created from the cement. This testing confirms that cement has the ability to perform well in concrete; however, the performance of concrete in the field is determined by all of the concrete ingredients, their quantity, as well as the environment, and the handling and placing procedures used.

Although the process for cement manufacture is relatively similar across the globe, the reference to cement specifications can be different depending on the jurisdiction. In addition, test methods can vary as well, so that compressive strength requirements (for example) in Europe do not ‘translate’ directly to those in Africa.

2.3.3 Aggregates

Aggregates are inert granular materials such as sand, gravel, or crushed stone that, along with water and Portland cement, are an essential ingredient in concrete. For a good concrete mix, aggregates need to be clean, hard, strong particles free of absorbed chemicals or coatings of clay and other fine materials that could cause the deterioration of concrete.

Aggregates account for 60 to 75 % of the total volume of concrete; its properties have a definite influence on behaviour of hardened concrete. Since aggregate is relatively inexpensive and does not enter into complex chemical reactions with water; it has been customary, therefore, to treat it as inert filler in concrete. However, due to increasing awareness of the role played by aggregate in determining many important properties of concrete, the traditional view of aggregate as inert filler is being questioned (Mehta and Monteiro, 2013).

A study by Abdullahi (2012) and Aginam et al. (2013) examined the effects of coarse aggregate types on the compressive strength of concrete using three different types of coarse aggregate for their comparative research. Abdullahi worked with quartzite, granite and river gravel and concluded that crushed quartzite concrete has the highest strength at 28-day curing. While Aginam et al. stated that concrete containing crushed granite has the highest compressive strength at 28-day curing having worked with crushed granite, washed gravel and unwashed gravel.

2.3.3.1 Classification of Aggregates

Strength of concrete and mix design is essentially independent of the composition of aggregates. No particular rock or mineralogical type in itself is required for aggregate. In the absence of special requirements, most kinds of rocks and most of the artificial materials can produce acceptable aggregates that conform to BS and ASTM specifications. Thus, classification by mineralogy or rock type has almost no practical engineering significance.

I) Classification on the basis of specific gravity and origin:

a. Normal weight aggregate: most natural mineral aggregates have bulk density of 1520 to 1680 Kg/m3 and produce normal weight concrete with approximately 2400Kg/m^3. They are further divided into:

i. Natural aggregate (e.g. sand, gravel, crushed rock such as granite, quartz, basalt, sandstone etc.).
ii. Artificial aggregate (e.g. broken brick, Air cooled slag etc.)

b. Lightweight Aggregate: for special needs, aggregates with lighter or heavier densities can be used to make correspondingly light-weight and heavy-weight concretes. Generally, the aggregates with bulk densities less than 1120 Kg/m^3 are called lightweight aggregate. They include: sintered clay, shale slate, diatomaceous shale, perlite, vermiculite or slag, others are, natural pumice, scoria, volcanic cinders, tuff, palm karnel shells, periwinkle shells, and diatomite (Ediongsenyene, 2014).

c. Heavyweight Aggregate: those weighing more than 2080 Kg/m^3 are called heavyweight aggregates.

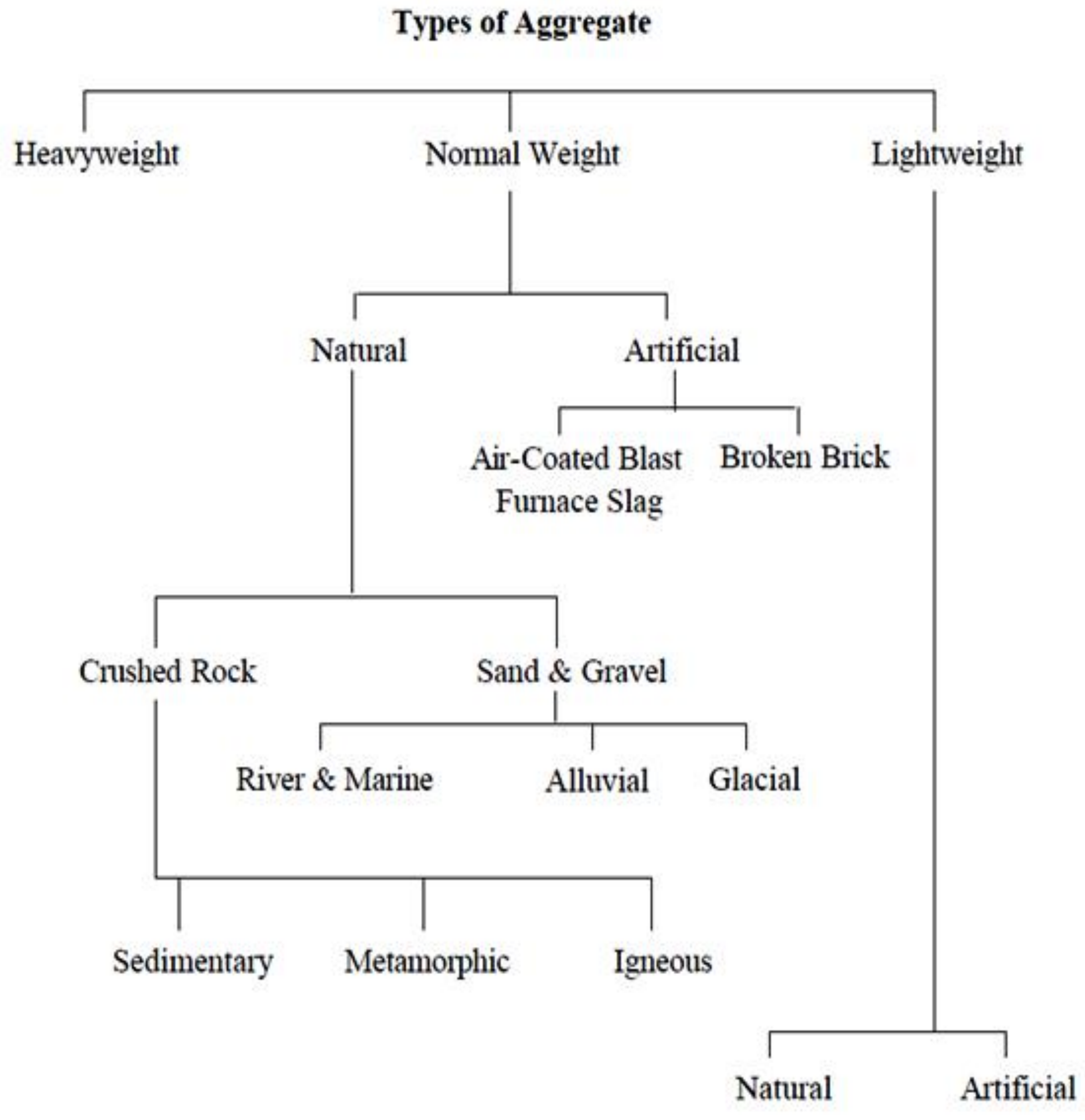


Figure 2.2 Classifications of Aggregate

Source: Saad (2005).

II) Classification based on aggregate size:

a. Fine Aggregate: Often fine aggregates are called sand and are not larger than 5 mm or 3/16 in. The coarse aggregates comprise the materials in size greater than this size. However, the division is at No. 4 sieve, which is actually 3/16 inch or 4.16 mm in size, i.e. the same as mentioned above. Fine aggregates generally consist of natural sand or crushed stone with most particles passing through a 3/8-inch sieve.

b. Coarse Aggregate: Coarse aggregates are any particles greater than 4.75 mm (0.19 inch) retained on No. 4 sieve, but generally range between 3/8 and 1.5 inches in diameter. Gravels constitute the majority of coarse aggregate used in concrete with crushed stone making up most of the remainder.

Natural gravel and sand are usually dug or dredged from a pit, river, lake, or seabed. Crushed aggregate is produced by crushing quarry rock, boulders, cobbles, or large-size gravel. Recycled concrete is a viable source of aggregate and has been satisfactorily used in granular sub-bases, soil-cement, and in new concrete (Mehta and Monteiro, 2013). The various types and classifications of aggregate can be seen in Figure 2.2.

Aggregates strongly influence concrete's freshly mixed and hardened properties, mixture proportions, and economy. Consequently, selection of aggregates is an important process.

2.3.3.2 Properties of Aggregate

Although some variation in aggregate properties is expected, the following characteristics are considered.

Physical Properties:

i. Grading
ii. Particle shape and surface texture
iii. Abrasion and skid resistance
iv. Density and specific gravity
v. Aggregate voids
vi. Absorption, Surface moisture and permeability
vii. Strength and elasticity
viii. Hardness
ix. Undesirable physical components

Chemical Properties:
i. composition
ii. Reactions with asphalt and cement

2.3.3.3 Impact of Physical Properties of Aggregate on Concrete Strength

Although aggregate is considered inert filler, it is a necessary component that defines the concrete's thermal and elastic properties and dimensional stability. Physical and mineralogical properties of aggregate must be known before mixing concrete to obtain a desirable mixture. These properties along with water/cement ratio determine the strength, workability and durability of the concrete.

Grading: This refers to the determination of the particle-size distribution for aggregate. As reviewed by Neptune (2008), grading limits and maximum aggregate size are specified because these properties affect the amount of aggregate used as well as cement and water requirements, workability, pumpability, and durability of concrete. The grading or size distribution of aggregate is an important characteristic because it determines the paste requirement for workable concrete. When the particles are of uniform size the spacing is the greatest but when a range of sizes is used the void spaces are filled, the less workable the concrete becomes, and therefore, a compromise between workability and economy is necessary.

In general, if the water-cement ratio is chosen correctly, a wide range in grading can be used without a major effect on strength. When gap-graded aggregate are specified, certain particle sizes of aggregate are omitted from the size continuum. Gap-graded aggregate are used to obtain uniform textures in exposed aggregate concrete. Close control of mix proportions is necessary to avoid segregation.

Particle shape and surface texture: Particle shape and surface texture influence the properties of freshly mixed concrete more than the properties of hardened concrete (Masad, 2002). Concrete is more workable when smooth and rounded aggregate is used instead of rough angular or elongated aggregate. Crushed stone produces much more angular and elongated aggregate, which have a higher surface to volume ratio better bond characteristics but require more cement paste to produce a workable mixture. 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 (Quiroga and Fowler, 2004).

Generally, flat and elongated particles are avoided or are limited to about 15 percent by weight of the total aggregate. Surface texture is the pattern and the relative roughness or smoothness of the aggregate particle. Surface texture plays a big role in developing the bond between an aggregate particle and a cementing material. A rough surface texture gives the cementing material something to grip, producing a stronger bond, and thus creating a stronger hot mix asphalt or Portland cement concrete strength (IMCP, 2006; Mehta and Aitcin, 1990).

Abrasion and Skid Resistance: Abrasion and skid resistance of an aggregate are essential when the aggregate is to be used in concrete constantly subject to abrasion as in heavy-duty floors or pavements. Beshr et al. (2003) investigated the effect of coarse aggregate quality on the mechanical properties of high strength concrete. He observed that the strength of concrete at the interfacial zone essentially depends on the integrity of the cement paste and the nature of the coarse aggregate. Different minerals in the aggregate wear and polish at different rates. Harder aggregate can be selected in highly abrasive conditions to minimize wear.

Density and Specific Gravity*:* Density is the weight per unit of volume of a substance. Specific gravity is the ratio of the density of the substance to the density of water. The density and the specific gravity of an aggregate particle are dependent upon the density and specific gravity of the minerals making up the particle and upon the porosity of the particle. The density of the aggregate is required in mixture proportioning to establish weight- volume relationships.

Aggregate Voids: There are aggregate particle voids, and there are voids between aggregate particles. The required amount of the concrete paste is dependent upon the amount of void space that must be filled and the total surface area that must be covered.

As solid as aggregate may be to the naked eye, most aggregate particles have voids, which are natural pores that are filled with air or water. These voids or pores influence the specific gravity and absorption of the aggregate materials. The void content between particles affects the amount of cement paste required for the mix (Mindess et al., 2003). Angular aggregates increase the void content. Larger sizes of well-graded aggregate and improved grading decrease the void content. The voids between the particles influence concrete mix design.

Absorption, Surface Moisture and Permeability: Absorption and surface moisture of aggregate are measured when selecting aggregate because the internal structure of aggregate is made up of solid material and voids that may or may not contain water. The amount of water in the concrete mixture must be adjusted to include the moisture conditions of the aggregate. Absorption relates to the particle's ability to take in a liquid. Porosity is a ratio of the volume of the pores to the total volume of the particle. Permeability refers to the particle's ability to allow liquids to pass through. If the rock pores are not connected, a rock may have high porosity and low permeability (Mehta and Monteiro, 2013).

Strength and Elasticity: Strength is a measure of the ability of an aggregate particle to stand up to pulling or crushing forces. Elasticity measures the "stretch" in a particle. High strength and elasticity are desirable in aggregate base and surface courses. These qualities minimize the rate of disintegration and maximize the stability of the compacted material. The best results for Portland cement concrete may be obtained by compromising between high and low strength, and elasticity. This permits volumetric changes to take place more uniformly throughout the concrete (Kuo and Freeman, 2000). If strength of aggregate is lower than strength of mortar, cracking occurs in the aggregate in the first place. For this reason, load bearing capacity of the concrete is considerably affected by aggregate properties. In other words, aggregates form the carrier system of regular concrete (Neville, 2000).

Hardness: The hardness of the minerals that make up the aggregate particles and the firmness with which the individual grains are cemented or interlocked control the resistance of the aggregate to abrasion and degradation. Soft aggregate particles are composed of minerals with a low degree of hardness. Weak particles have poor cementation. Neither type is acceptable. The Mohs Hardness Scale is frequently used for determination of mineral hardness (Ali et al., 2012).

Undesirable Physical Components*:* Particles with undesirable physical characteristics include but are not limited to the following: non-durable soft or structurally weak particles, clay lumps or clay balls**,** flat or elongated particles**,** organic matter contaminants**,** lightweight chert.

2.3.3.4 Impact of Chemical Properties of Aggregates on Concrete

The chemical properties of aggregates have to do with the molecular structure of the minerals in the aggregate particles.

a) Composition

The word dolomite is the mineral calcium magnesium carbonate Ca Mg $(Co_3)2$. Elemental magnesium content of 10.3 percent or above is required for dolomite aggregates. Some aggregates have minerals that are subject to oxidation, hydration, and carbonation. These properties are not particularly harmful, except when the aggregates are used in Portland cement concrete. As might be expected, iron sulfides, ferric and ferrous oxides, free lime, and free magnesia in industrial products and wastes are some of the common substances. Any of these substances may cause distress in the Portland cement concrete and give the concrete an unsightly appearance.

b) Reactions with Asphalt and Cement

There are several types of substances found in mineral aggregates which may have a negative effect on the cementing and overall performance qualities of asphalt and cement. Most are rarely significant but various organic substances may retard hardening, reduce strength development or cause excessive air entrainment in Portland cement concrete (Obla et al., 2003).. These organic substances include, but are not limited to, mica, iron oxide, lightweight chert, shale, coal, and lignite.

2.4 Sandstone as Coarse Aggregate

Sandstones are sedimentary rocks formed by the cementation of sediment by material cements and they show a great deal of variation in mineral composition, degree of sorting and roundness. Sedimentary rocks are formed from the erosion, transportation, sorting, deposition and lithification of sediments derived from physical, biological and chemical weathering of pre-existing igneous, metamorphic and sedimentary rocks. Sediments are the collective name for loose, solid particles that originate from the weathering and erosion of pre-existing rocks and the chemical precipitation from solution including secretion by organisms in water. Sandstones are sedimentary rocks with sand grains of 1/16 to 2mm in diameter and they possess quality reservoir characteristics and mineralogy (Ikhane et al., 2012).

As the name implies, sandstone contains sand-sized grains of rock fragments and individual minerals broken down from other, older rocks. The original rocks that eventually make up the small grains of sandstone are weathered when they break down. The pieces coming off of a source rock can then be eroded, or carried away from the source area. The longer the chunks of a source rock are carried by wind, water, or ice, the more likely they are to be broken down into very small fragments. Certain minerals found within rocks are also more likely to survive significantly long rock fragment travels. Because quartz is such a hardy mineral, unlikely to undergo chemical changes during erosion, it is the mineral found in the greatest amounts in many sands. According to Hisam et al. (2002), during the crushing process at the quarry, about 22% of the particle has the size 3mm to dust which is known as crushed sandstone sand.

In places where there is scarcity of suitable materials for high performance concrete, the economic considerations may necessitate experimenting with suitable locally occurring substitute materials. Some aggregates however, can cause detrimental effect on durability performance of the concrete. Sandstone is a reactive material, which has detrimental effect on concrete durability. One of the major concerns with such aggregate is the alkali-silica reaction. This has raised the concern now when the concrete industry is forced to develop durable concrete with local available sources (Paramasivam, 2006).

2.4.1 Classification of Sandstone

Classification of sandstone based on the scheme of Krynine (1984), has revealed a lot of inconsistencies. This is because most of the key parameters: quartz (Q), feldspar (F), rock fragment (R) and matrix element (M), are not properly defined. However, Pettijohn et al. (1972) and Pettijohn (1975) have successfully used the Na_2O/ K_2O ratios in isolating graywackes from arkoses.

Based on the variation in composition and cementing material the sandstone has been categorized unto three major types, which are as follows:

i. Quartz Sandstone

Quartz sandstone formed as a result of the extreme weathering and sorting of a sediment until everything that can be removed has been removed. The high content of quartz is a result of removal of feldspar and lithic. This is done by the complete chemical weathering and then the final removal of the clay takes placed in high energy environments. Tidal sand bars that accumulate large bodies of quartz sand is yet another situation that leads to formation of sandstone. Sand-sized quartz grains could come from the weathering of source area rocks such as granite, gneiss, or other sandstones which contain quartz. Quartz sandstone implies a long time in the depositional basin and is dominated by quartz grains.

ii. Arkose Sandstone

Arkose sandstone is derived from disintegration of granite or gneiss, and characterized by high feldspar content. This is thus the quartz sandstone containing over a quarter feldspar with iron oxide cement. Micas may also be present. Bedding is sometimes present, but fossils are rare. It effervesces slightly in dilute hydrochloric acid thus indicating calcite cement. Its colour is usually in the shades of buff, brownish-gray or pink. Arkose sandstone finds its applications in building stone, and millstones for grinding corn. Arkose implies a short time in the depositional basin (because feldspar typically weathers quickly to clay). Also implies rapid erosion, arid climate, tectonic activity, steep slopes. It is dominated by feldspar grains (usually potassium feldspar)

iii. Litharenite or Lithic Sandstone

Litharenite or lithic sandstone is commonly known by the name of graywacke. It is predominantly composed of dark sand-sized rock fragments, with some mica, quartz, and feldspar grains in a clay-rich matrix. A litharenite is composed of sand-sized rock fragments. Litharenite implies rapid erosion, temperate or arid (not humid) climate. It is dominated by rock fragment grains.

2.4.1.1 Physical Properties of Sandstone

In their study of Ajali sandstone units in Ohafia area, south-eastern Nigeria, Ibe and Akaolisa (2010) identified some physical properties of sandstone. The colour varies from red, green, yellow, gray and white. The variation is result of the binding material and its percentage constituent. The capacity of water absorption is not more than 1.0%. Hardness lies between 6 to 7 on Mohr's Scale and density 2.32 to 2.62 Kg/m^3. The porosity varies from low to very low while the compressive Strength varies from 365 to 460 Kg/m2

2.4.1.2 Chemical Properties of Sandstone

The chemical constitution of sandstone is the same as that of sand; the rock is thus composed essentially of quartz. The natural cementing material that binds the sand together as rock is usually composed of silica, calcium carbonate, or iron oxide. Chemically sandstone is very resistant Mono-Mineralic rock, with silica as the principal (Ibe and Akaolisa, 2010). The percentage of each constituent is as follows:

i. SiO_2 - 93 to 94%

ii. Iron (Fe_2O_3) - 1.5% to 1.6%

iii. Alumina (Al_2O_3) - 1.4 to 1.5%

iv. Soda (Na_2O) & Potash (Kro) - 1.0% to 1.2%
v. Lime (CaO) - 0.8% to 0.9%
vi. Magnesia (MgO) - 0.2 to 0.25%
vii. Loss on Ignition (LOI) - 1.0% to 1.2%
They are highly resistant to acids, alkalis and thermal impact.

2.4.2 Alkali-Silica Reaction of Sandstone

Alkali-silica reaction (ASR) occurs either in mortar or concrete. ASR is a deleterious chemical reaction between hydroxyl (OH^-) ions associated with alkalis (sodium and potassium) present in cement or other sources and certain reactive siliceous components that may be present in aggregates, producing the gel. When the alkali-silica gel absorbs moisture, it expands and eventually produces cracks in aggregate particles as well as in the concrete structures. According to Mehta and Monteiro, 2013, the following three concrete conditions must be satisfied for expansive ASR to occur:

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

It is the form of silica that determines whether a siliceous aggregate is reactive or not. Certain reactive aggregates do not exhibit maximum expansion unless the aggregate is present in critical range. The proportion of reactive aggregate particles that produces maximum expansion for a given alkali content and water-binder ratio (w/b) in concrete is known as the pessimum proportion.

Not all aggregates that are susceptible to ASR, however, show the pessimum effect. Aggregates without this pessimum effect exhibit increasing expansion as a function of the amount of reactive particles present in the aggregate. Fine aggregates are more susceptible to ASR because of its higher surface area.

2.4.3 Petrography of Sandstone Aggregate

Petrography is a comparatively quick way to predict aggregate reactivity based on microscopic examination of aggregate samples. Mineral properties in aggregate determine aggregate reactivity. A petrographic and geochemical study of sandstones in the Afowo Formation of Dahomey basin southwestern Nigeria was carried out to infer the various elemental compositions, mineralogical composition, degree of sorting and degree of roundness of the sandstones in order to classify them. The mineral compositions obtained from the petrographical analysis are Quartz, Aluminum oxides, and high percentage of Iron oxides acting as the cementing material (Ikhane et al, 2012). A petrographical, mineralogical and textural description of the different sources of Kota Kinabalu quarried sandstone aggregate has the following characteristics (Felix, 1989). Thin-section studies showed that the sandstone consisted of 70% quartz, 8% chert, 15% feldspar and less than 10% rock fragments including sedimentary, metamorphic and igneous rock. In terms of chemical composition, sandstone contains mainly silica 82% and aluminum 9% and all other compositions are marginal.

The interstitial matrix consists of silt-sized quartz, mica, and probably also sub-microscopic clay minerals at grain interfaces. Internal porosity of sandstone is clearly enhanced adjacent to mica. In general, the sandstone is held together by phyllosilicate minerals (clay and altered rock fragments) due to local compaction and rarely by chemical cement, and it is relatively soft and friable. Coarser granularity and better crystallinity suggest that sandstone is less kiable to silica dissolution.

Petrographic examination of sandstone aggregate from a different source by Gogte (1973) as cited by Paramasivam (2006) indicated that the mortar-bar expansions with sandstones appear to vary widely between 0.032 and 0.25%. one containing 20% chert shows an expansion of 0.25% while another with 7% chert shows expansion up to 0.11%. Some of the sandstones containing a few grains of strongly undulatory quartz also show negligible expansions.

The amount of gel forming depends on the reactive silica and therefore, up to a point, an increase in the amount of reactive silica produces an increase in expansion. However, above a certain proportion of reactive silica to alkali so much alkali is absorbed that the concentration of hydroxide in solution is insufficient to maintain the same degree of attack. Thus, the expansion decreases again. Exceeding certain percentage (about 45%) is known as pessimum proportion that may occur in many aggregates. This excess silica acts as a deterrent to the reaction.

Hobbs (1989) indicated that the degree of cracking induced in concrete subjected to ASR decreases as the reactive silica content of the aggregate increases. Also, it is pointed out that at very high silica contents no cracking or expansion is observed. The percentage at which this limit of the expansion occurs is higher at lower water-cement ratios and at higher cements (Neville, 2005; Ferraris, 1995).

2.5 Physical Properties of Concrete

Form engineers point of view, important properties of hardened concrete can be listed as follows:

- Workability
- Strength
- Durability
- Shrinkage
- sustainability

The literature is discussed about how each concrete property is affected by mixture composition. The five mixture characteristics covered include: - cement content, water-to-cement ratio (w/c), aggregates, chemical admixtures and supplementary cementitious materials. Stronger concretes are stiffer, more nearly watertight, and more resistant to weathering and certain destructive agencies. On the other hand, however, stronger concretes usually exhibit higher drying shrinkage and lower extensibility, hence are more liable to cracking. A structure must be adequately designed and properly constructed of concrete which is strong enough to carry the design loads and which is economical not merely in terms of cost but also in terms of its ultimate service.

The term w/c was used instead of the water-to-binder ratio (w/b) throughout this report because no supplementary cementitious materials were used in this study.

Concrete durability is commonly specified by defining minimum cement content, minimum strength, and maximum free w/c (Arachchige, 2008). The w/c is the main factor affecting concrete strength where lower w/c provides higher strength. However, it is also perceived that concrete strength is controlled by the cement content. Based on this perception, a common specified design parameter is the minimum cement content which may exceed the amount required for the desired strength and durability.

2.5.1 Workability

American Concrete Institute (ACI) 116R defines workability as “that property of freshly mixed concrete or mortar that determines the ease and homogeneity with which it can be mixed, placed, compacted and finished to a homogenous condition”. The concrete must be easily capable of satisfying the following requirements: -

i. It must be easily mixed and transported.

ii. It must be uniform throughout a given batch and between batches.

iii. It should have flow properties such that it is capable of filling completely the form for which it was designed.

iv. It must have the ability to be compacted fully without an excessive amount of energy being applied.

v. It must not segregate or bleed during placing and consolidation.

vi. It must be capable of being finished properly.

A concrete satisfying these conditions is said to be workable. Workability is often defined in terms of the amount of mechanical work, required to produce full compaction of the concrete without segregation. Workability can be identified by three main parameters (Kosmatka et al., 2002; Chen and Duan, 2000):

i. Cohesiveness: the resistance to segregation,

ii. Consistency: the ease of flow, and
iii. Plasticity: the ease of molding.

Although workability is typically quantified by the slump tests, these tests are of limited value because they do not fully characterize concrete flow (Ferraris and Gaidis, 1992). For mixtures with different proportions, slump tests should not be used for comparison, but they indicate uniformity for similar mixtures (Kosmatka et al., 2002). However, mixtures with same slump value may not behave the same way during placement (Ferraris and Gaidis, 1992).Workability is commonly assessed by engineers using the slump test (ASTM C143). A number of factors can influence the workability of a mixture:

Water Content

Water content is the most important factor for workability and increasing the water content in concrete will increase workability (Mindess et al., 2003). However, excessive water content should be avoided to prevent segregation and bleeding (Mindess et al., 2003). For given cement content, increasing water content will also increase w/c and that increased w/c will increase workability (Kosmatka et al., 2002).

Cement Content

Workability is affected by paste volume, because the paste lubricates the aggregates (Dhir et al., 2004). For a given water content, decreasing the cement content increases stiffness of concrete with having poor workability (Lamond and Pielert, 2006). Concrete with high cement content shows high cohesiveness and becomes sticky (Lamond and Pielert, 2006; Kosmatka et al., 2002; Mehta and Monteiro, 1993). To prevent an adverse effect, appropriate cement content should be used to achieve the desired workability.

Aggregates

Aggregates constitute 60 % to 75 % of the total volume of concrete; therefore their selection is very important in the mix design process. Gradation, shape, porosity, and surface texture of aggregates affect the workability of concrete (Kosmatka et al., 2002). Aggregates should be well-graded to achieve the desired workability because fine aggregates have a high water requirement due to their high specific surface area and inadequate amount of fine aggregate causes mixtures to become stiff and segregate (IMCP, 2006; Mindess et al., 2003; Shilstone, 2002). Aggregate shape and texture affect workability through their effect on cement paste requirements. Spherical, well rounded with smooth surfaced aggregates increase workability whereas angular, elongated, rough surfaced aggregates decrease workability and cause segregation (Mindess et al., 2003).

Chemical Admixtures

Water-reducing agents make water available in concrete by neutralizing the surface charge of cement particles which causes flocculation and blocks water particles in those (Mindess et al., 2003). Therefore, for given water-content, the addition of a water-reducing admixture will increase workability (IMCP, 2006; Mindess et al., 2003; Kosmatka et al., 2002; Mehta and Monteiro, 2013).

Supplementary Cementitious Materials

Supplementary cementitious materials, especially fly ash, slag, calcined clay, metakaolin and shale generally improve the workability of concrete because their fine spherical morphology reduce interparticle friction (IMCP, 2006; Mindess et al., 2003; Kosmatka et al., 2002; Wong et al., 2001). However, silica fume increases the water requirement and stickiness of a concrete mixture because of its high surface area (IMCP, 2006; Obla et al., 2003; Kosmatka et al., 2002; Ferraris et al., 2001).

2.5.2 Strength

Strength can be defined as the ability force. Kosmatka et al. (2002) define strength as “the measured maximum resistance of a concrete specimen to axial loading”. With regard to concrete for structural purpose, it is taken, unless stated otherwise, as unit force (stress) required to cause rupture. Rupture may be caused by applied tensile stress (failure in cohesion), by applied shearing (sliding) stress, or by compressive (crushing) stress. Although other parameters such as durability and shrinkage may be more critical to assess concrete quality, strength is still commonly used for this purpose, particularly in structural applications (IMCP, 2006).

Strength is affected by the following factors:

- Water-to-cement ratio (w/c): Increasing w/c will decrease strength (IMCP, 2006; Mindess et al., 2003; Kosmatka et al., 2002; Mehta and Monteiro, 2013).
- Degree of hydration: Increasing the degree of hydration will increase strength (IMCP, 2006; Mindess et al., 2003; Kosmatka et al., 2002; Mehta and Monteiro, 1993).
- Age: Strength increases as concrete age increases, initially rapidly and slowing over time (IMCP, 2006; Mindess et al., 2003; Kosmatka et al., 2002; Mehta and Monteiro, 2013).
- Supplementary cementitious materials: Increasing the supplementary cementitious materials content will change strength (IMCP, 2006; Mindess et al., 2003; Kosmatka et al., 2002; Mehta and Monteiro, 2013).
- Entrained air: Increasing the entrained air will decrease strength (IMCP, 2006; Mindess et al., 2003; Kosmatka et al., 2002; Mehta and Monteiro, 2013).
- Admixture type and dosage: Water-reducing agents may have an indirect influence on strength (Mindess et al., 2003; Kosmatka et al., 2002; Mehta and Monteiro, 2013). The effects of other admixtures on strength are beyond the scope of this study.
- Aggregates: Rough and angular aggregates will increase strength (IMCP, 2006; Mindess et al., 2003; Mehta and Monteiro, 2013).
- Type of cement: Increasing the cement fineness will increase the early strength (Mindess et al., 2003; Kosmatka et al., 2002; Mehta and Monteiro, 2013).
- Cement content: For a given w/c, strength is reportedly independent of cement content (Wassermann et al., 2009; Dhir et al., 2004; Schulze, 1999).

2.6.3.1 Compressive strength

Except for highway pavements, most concrete structures are designed under an assumption, that the concrete resists compressive stresses but not tensile stresses; hence, for purposes of structural design the compressive strength is the criterion for quality, and working stresses are prescribed by codes in terms of percentages of compressive strength as determined by standard tests. A future consideration is that compression tests are relatively easy to make.

The usual test employs a cubic specimen of height equal to twice the diameter, moistured at 21ºC for 28 days and then subjected to slow (static) loading at a specified rate until rupture occurs; usually loading is completed within 2 or 3 minutes. Values of strength obtained in this way usually range from 2000-6000 psi (Ezgi, 2010).

2.6.3.2 Tensile strength

As previously stated, concrete is not expected to resist direct tensile forces because of its relatively low tensile strength and brittle nature. However, tension is of importance with regard to cracking, which is a tensile failure; most cracking (aside from that due to settlement of parts of the structure) is due to restraint of contraction induced by drying shrinkage or lowering of temperature (Mindess et al., 2003).

2.6.3.3 Flexural strength

When concrete is subject to bending, tensile and compressive stresses and in many cases direct shearing stresses are developed. The most common plain concrete structure subjected to flexural is a highway pavement, and the strength of concrete for pavements is commonly evaluated by means of bending tests on beam specimens. Flexural strength is expressed in terms of “modulus of rupture”, which is the maximum tensile (or compressive) stress at rupture.

2.6.3.4 Shear strength

Shear is the action of two equal and opposite forces applied in planes a short distance apart. Shear stress cannot exist without accompanying tensile and compressive stresses. Pure shear can be applied only through torsion of a cylindrical specimen, in which case the stresses are equal in primary shear, secondary tension and secondary compression. Since concrete is weaker in tension than in shear, failure in torsion invariably occurs in diagonal tension.

2.6.3.5 Impact strength

It has been suggested that the impact strength varies from 0.5 to 0.75 of the compressive cube strength. A measure of impact strength is the number of blows that concrete can withstand till there is “no rebound” of impacting device. Factors affecting impact strength are: testing method, nature of cement aggregate bond and curing conditions.

2.5.3 Density

The density of a concrete is a measure of its unit weight. A normal weight concrete has a density of about 2400Kg/m^3. The unit weight of concrete varies depending on the amount of and density of aggregate, the amount of entrained air (entrapped air), and the water-cement content. Maximum unit weight prescribed for lightweight aggregate concrete is 1850kglm^3 and minimum compressive strength at 28 day is 17 N/mm^2. Heavyweight concrete are produced from high densities and generally weighs more than 3200Kg/m^3. Porous aggregates are likely to form light but weak concrete. Introduction of sand as fine aggregate increases the unit weight of the corresponding concrete. Entraining of air, which improves the workability of the resulting concrete, compensates this increase in unit weight.

Many research works have shown that increase in unit weight of concrete brings about increase in the compressive strength. A reduced density of normal concrete always mean a higher water content which means lower strength concrete.

Müller (2001) showed that higher unit weight results into higher compressive strength. He used the same mix ratio and water/cement ratio in his investigation. A total of six lightweight aggregates were used. The result of compressive strength from the concrete made with these aggregates is:

Expanded Clay 1 (density = 1067 kg /m^3) has compressive strength of 38.4 N/mm^3.
Expanded Clay 2 (density = 1650kg/m^3) has compressive strength of 53.1N/mm^2.
Expanded shale (density = 1259kg/m^3) has compressive strength of 30.4N/mm^2.
Glass waste (density = 260kg/m^3) has compressive strength of 20.90N/mm^3.

Masonry construction and demolition waste, CD1 (density = 530kg/m^3) has compressive strength of 28.6 N/mm^2.
CD 2 (density =1800kg/m^3) has compressive strength of 39.5 N/mm^2.

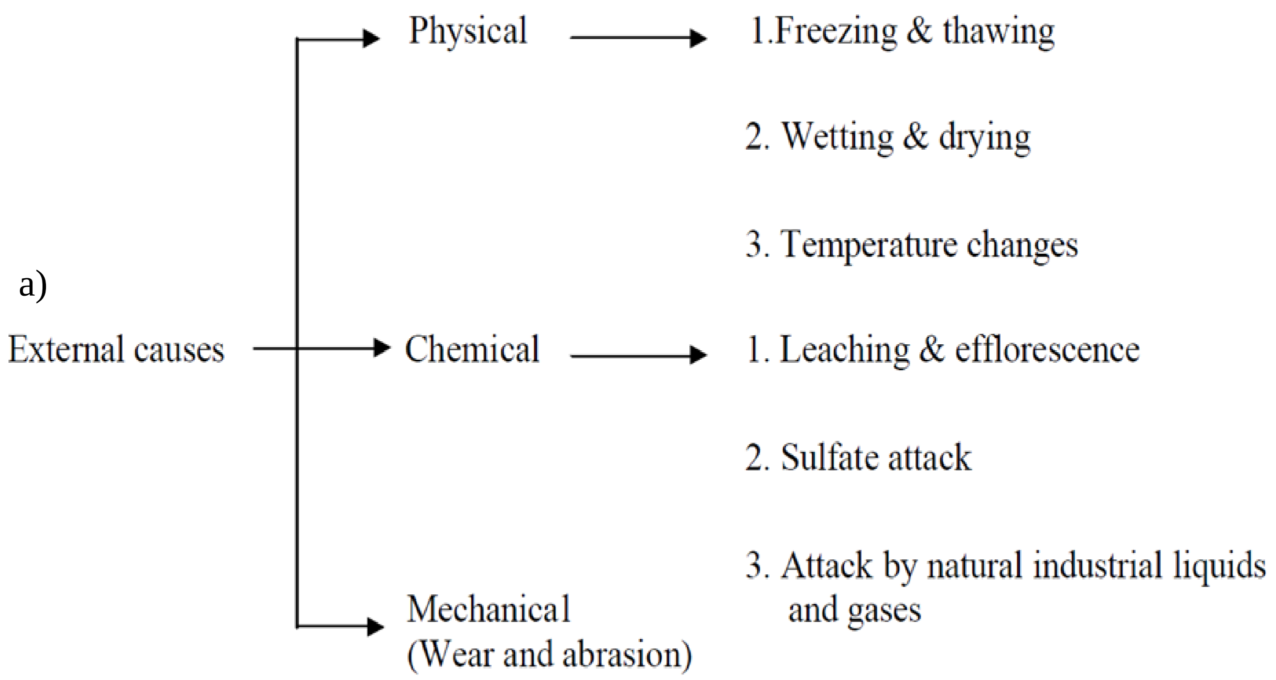
The above result confirmed the fact that the higher the density the stronger the concrete. However, it will be observed that CD2 that has the highest density did not become the strongest. This means that factors other than density might have caused it.
In their own work, Ansari et al. (2000) compared the compressive strength of concrete made with the conventional fine and coarse aggregate designated Class A and concrete made with Class A with 50% of the coarse aggregate replaced with recycled aggregate designated Class A with recycled Conc. They used the same mix ratio and water cement ratio. The compressive strengths at 28 days for the two are almost the same. Compressive strength of the former was 29.39 N/mm^3 while that of the later is 29.75 N/mm^2. Ironically their densities are almost the same. Thus, as sandstones are suspected to be normal weight and natural aggregate, the concrete made with them will be heavy with moderate compressive strength.

2.5.4 Durability

ACI Committee 201 (2008) defines durability of concrete as "the ability to resist weathering action, chemical attack, abrasion, or any other process of deterioration and retain its original form, quality, and serviceability when exposed to its environment".
Air-entrained concrete seem to be more durable than others are. They resist freeze-thaw better than concrete without air-entrainment. The hydraulic pressure caused by expulsion of water from the aggregate can be accommodated by the entrained air present in the cement mortar, thus preventing damage to the concrete. Because lightweight aggregates are porous, they entrap air within them. The air they entrapped is similar to entrained air in concrete. As such lightweight aggregates concrete has more durability potential than normal weight aggregate concrete. The ability of concrete to resist chemical attacks is a welcomed development (Aitcin, 2003).

Ferraris et al. (1997) studied the effect of stress relaxation, self-desiccation and water absorption on the alkali – silica– reaction in low water/cement ratio mortar. The aggregates experimented on were sprat (with more than 0.1% expansion at 14 days), siliceous dolomitic limestone and non reactive lime stone (with 0.01% expansion at 14 days). Their result showed that the mix with the former aggregate expanded up to a maximum of 0.43% at 30 days while that of later expanded up to a maximum of 0.02% at 30 days. The mix of the former gained more mass than the mix of the later. It can be seen that the non-reactive aggregate produced more durable mix.

Concrete which can withstand the conditions for which it has been design, without deterioration, over a period of years is said to be durable. The absence of durability may be due to two causes: –



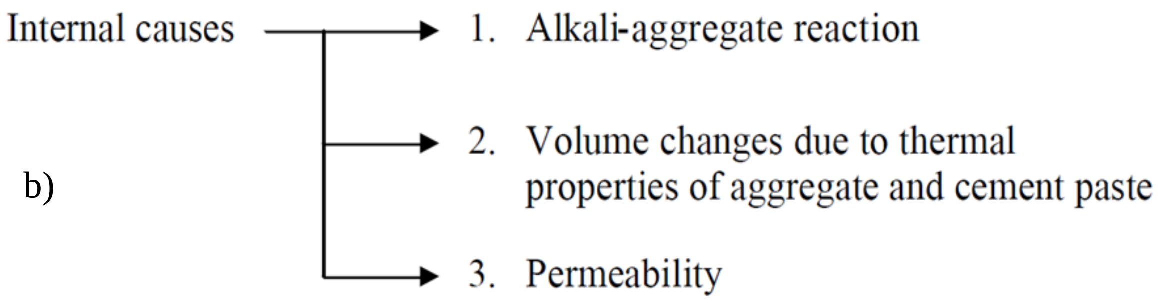


Figure 2.3 Causes of durability

Source: Mindess et al. (2003).

Environmental conditions, concrete components, mix design, placement and curing determine the required degree of ultimate durability and life of different concretes (Kosmatka et al., 2002).

The following factors influence the concrete durability:

- Water content: Decreasing water content will increase durability (IMCP, 2006; Mindess et al., 2003; Kosmatka et al., 2002; Mehta and Monteiro, 2013).
- Water-to-cement ratio (w/c): Decreasing w/c will increase durability (IMCP, 2006; Dhir et al., 2004; Mindess et al., 2003; Kosmatka et al., 2002; Mehta and Monteiro, 2013).
- Cement content: For a given w/c, increasing cement content may decrease durability (Wassermann et al., 2009; Dhir et al., 2004).
- Supplementary cementitious materials: Increasing the amount of supplementary cementitious materials will generally increase durability (IMCP, 2006; Obla et al., 2003; Mindess et al., 2003; Kosmatka et al., 2002; Mehta and Monteiro, 2013).
- Degree of hydration: Increasing the degree of hydration will increase durability (IMCP, 2006; Mindess et al., 2003; Kosmatka et al., 2002; Mehta and Monteiro, 1993).
- Age: Durability increases as concrete age increases (IMCP, 2006; Obla et al., 2003; Mindess et al., 2003; Kosmatka et al., 2002; Mehta and Monteiro, 2013).
- Aggregates: Use of hard, dense and strong aggregate that is free of reactive silica will improve durability (IMCP, 2006; Mindess et al., 2003; Kosmatka et al., 2002; Mehta and Monteiro, 2013).
- Air void system: Having good air void system increases durability when concrete is subjected to the freeze-thaw conditions (IMCP, 2006; Mindess et al., 2003; Kosmatka et al., 2002; Mehta and Monteiro 2013).
- Admixture type and dosage: A water-reducing agent may indirectly increase durability by reducing w/c and providing a more uniform pore structure (Mindess et al., 2003; Mehta and Monteiro, 2013).

- Consolidation and curing: Adequate consolidation and proper curing will increase durability (IMCP, 2006; Mindess et al., 2003; Kosmatka et al., 2002; Mehta and Monteiro, 2013).

2.6 Concrete Mixture Optimization

Shakhmenko and Birsh (1998), defined optimization of the concrete mixture as a process of search for a mixture for which the sum of the costs of the ingredients is lowest, yet satisfying the required performance of concrete, such as workability strength and durability. The basic ingredients of concrete can be classified into two groups: cement paste and aggregates. While the quality of cement paste is governed mainly by the water/cement ratio, the quantity of cement paste required to achieve a targeted quality of concrete depends on the characteristics of aggregates. These characteristics mainly include surface area and voids in aggregates. The void content is affected mainly by the particle size distribution of aggregates and can be reduced by increasing the aggregate/cement ratio (He et al., 2012; Senthil and Manu, 2003). The packing degree according to Goltermann (1997) is a characteristic of the specific aggregate type or mix and it indicates the void volume and the amount of cement paste necessary in the concrete.

Various methods are employed in optimizing the concrete mixture design using either the fully experimental methods or fully analytical methods or semi-experimental (half-analytical) methods or statistical methods. Ahmed (2007) opined that, fully experimental methods involve an extensive series of tests, sometimes conducted on a trial-and-error basis, and the optimization results are often applicable only to a narrow range of local materials. In order to reduce the number of trial mixtures required to obtain an optimal mixture, efforts have been made towards developing analytical methods rationalizing the initial mixture proportioning into a more logical and systematic process (Yeh, 2007).

Analytical methods tries to achieve an optimum concrete mixture based on detailed knowledge of specific weights of mixture components and on certain basic formulas, which result from previous experience without conducting expensive and time-consuming experimental works. Half-analytical) methods are based on combining the experimental database or experimentally developed prediction models and various analytical tools such as artificial neural network, genetic algorithm, and mathematical programming (Lee et al., 2009; Yeh, 2009; Sonebi and Bassuoni, 2013).

In statistical methods, also termed as statistical experiment design methods, instead of selecting one starting mix proportion and then adjusting by trial and error for achieving the optimum solution, a set of trial batches covering a chosen range of proportions for each mixture component is defined according to established statistical procedures. Trial batches are then carried out, test specimens are fabricated and tested, and experimental results are analyzed using standard statistical methods (Simon, 2003; Ezeh and Ibearugbulem, 2009). These methods include fitting empirical models to the data for each performance criterion. Simultaneous optimization to meet several constraints is also possible. One could determine the lowest cost mixture with strength greater than a specified value, air content within a given range, and slump within a given range (Ahmad and Alghamdi, 2014). Fully analytical methods are less expensive and less time consuming but they have the disadvantage of being less precise because of the variations in the materials characteristics of the aggregates and cements. Fully experimental or semi-experimental (i.e., half-analytical) methods are reliable and accurate; however, they involve comprehensive laboratory works. Statistical methods also require a certain amount of experimental works but they have an additional advantage in a sense that the expected properties (responses) can be characterized by an uncertainty (variability). This has important implications for specifications and for production of the cost-effective concrete mixture observed by Simon (2003).

2.6.1 Concrete Mix Design

Integrated Materials and Construction Practices for Concrete Pavement Manual (IMCP, 2006) defined mix design as “the process of determining required and specifiable properties of a concrete mixture”. Mix design of concrete is a means of producing the most economical and durable concrete that meet with certain properties as consistency, strength and durability by properly and systematically combining the ingredients at relative proportions (Neville, 1983).

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. According to Teychenne et al. (1997), the mix design process must take account of those factors that have a major effect on the characteristics of the concrete, but can, at least at the first stage, ignore those which only have a minor effect on the concrete. There is little point in devising a complex method of mix design which takes into account factors which are difficult to measure or which are unlikely to remain constant during the progress of the job. The basic factors in the process of mix design are expressed diagrammatically in Figure 2.4.

Performance based concrete mix-design for normal vibrated concrete was discussed by Shilstone (1999) and Shilstone S. R and Shilstone J. R (2002). The authors were discussing that existing codes and standards might be conservative in some cases because they mostly are based on strength requirements. For some cases, strength may not be the most important criteria and concrete composed due to standards and codes can be unnecessary expensive and inefficient. The main objective with the idea with performance based concrete mix-design is to facilitate the possibility to design tailor made concrete mixes regarding locally available materials, functional demands and type of application, which can increase the possibility to design safer, more reliable and cost effective concrete mixes.

Although many durability properties of concrete are important, most design procedures are based primarily on achieving a specified compressive strength at some given workability and age. It is assumed that if this is done, the other properties will also be satisfactory. Special consideration will be required where water/cement ratio has to be modified, admixtures have to be used, and compromise has to be made between the strength and workability.

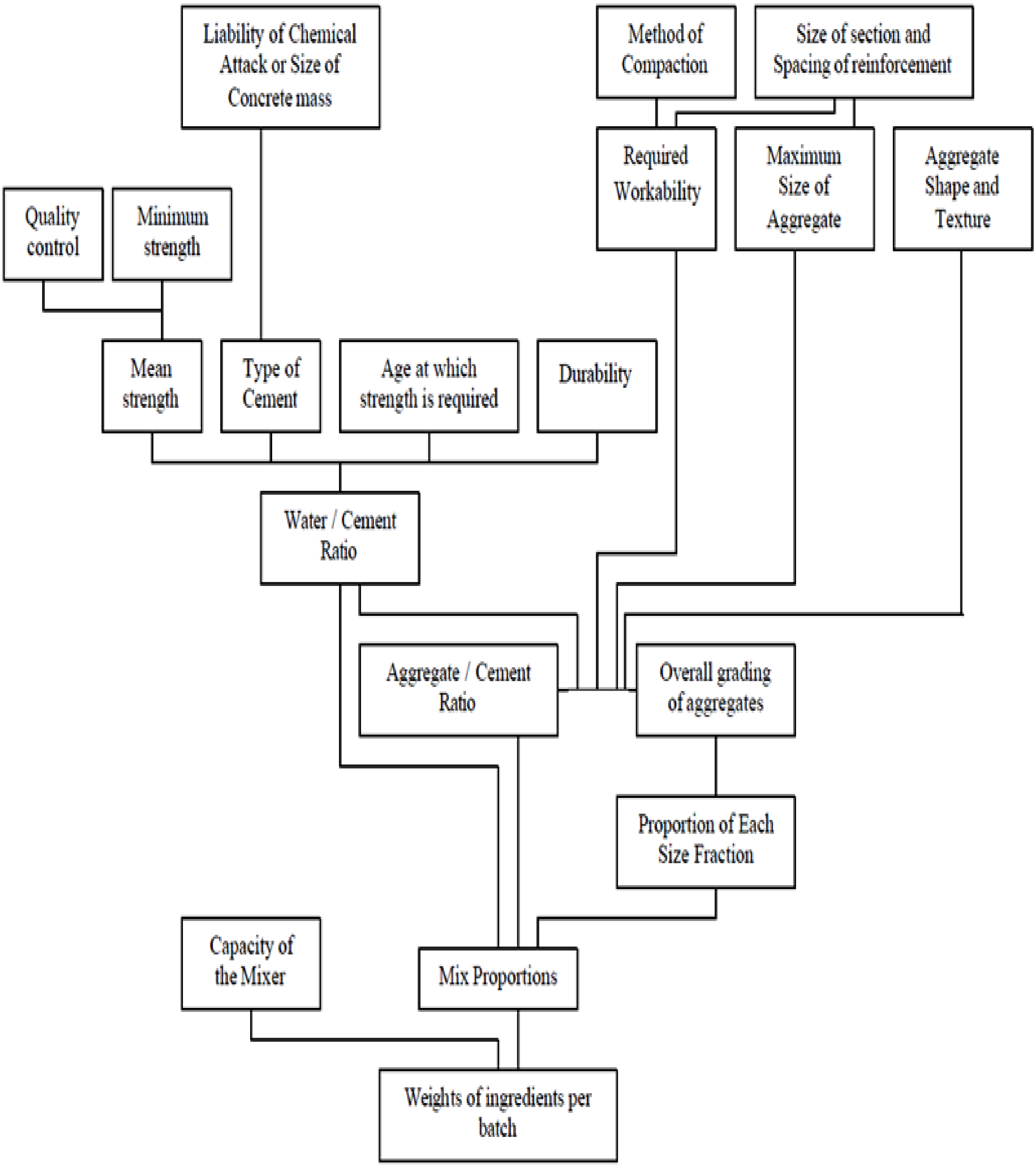


Figure 2.4 Basic Factors in the Process of Mix Design.

Source: Ezgi (2010).

Proportioning of concrete mixes or mix design determines the relative amounts of materials to be used in batches of concrete for a particular purpose. Proportioning of the ingredients is highly important because it provides the means of meeting the fundamental requirements of quality as economically as possible (Teychenne et al., 1997).

Many methods of mix-design have been developed over the years in concrete construction. However various methods used presently interpret these factors differently although the outcome may be satisfactory for all methods. Two categories of mix design method exist. One of them is the empirical method and the other is the statistical method (Simon et al., 1997). Among these methods, BS method and ACI methods are the mostly used methods. Information in the literature regarding the effect of mixture design decisions will be discussed in the following sections.

2.6.2 Empirical Method

Mathematical approaches to determine the correct proportion of component materials of a concrete mixture meeting a given set of specifications generally do not work because the materials vary widely in their characteristics. This explains why there is large number of empirical methods based on extensive test data developed from local materials (Mehta and Monteiro, 2013). Here one will rely on information from historic data or laid down rules (codes) for mixture proportioning. Historic data is just about the rule of thumb. It will only work for the materials that were used for it in the past. The method will not incorporate new material other than the ones used in the past. The use of trial mixes and repeated mixes (further trials) to enable all specified criteria to be met is inevitable. Hence, it will be difficult if not impossible to achieve an optimal mixture for desired criterion with historic information method of concrete mixture design. Empirical methods are prone to trial and error which results in material wastage whenever they are used (Ezeh and Ibearugbulem, 2009).

The use of laid down rules or codes in concrete mix design is an improvement over the historic information method. This is so because there is a stepwise (though rigorous) approach and it can accommodate a wider range of materials. Some common laid down rules in concrete mix design are Road Note No.4, ACI Standards 211.1-77 and 211.3-75 and 1975 British method (Ibearugbulem, 2006; Simon and others, 1997; Road Research, 1950; ACI Committee 211, 2002; and Teychenne and others, 1975).

2.6.2.1 BS Method of Mix Design

The BS method of mix design is based on a combination of British method (1987), Current British method (1981) and Modified Road Note No. 4 method. It was developed by Teychenne, D.C, Franklin, R.E and Erntroy, H.C of BRE, TRRL and CCA or UK. – Under DOE, BRE, TRE. The method uses SI units and is published by HMSO. The principle behind this method is that from the restricted data usually available at the mix design stage, mix proportions are derived in an attempt to produce a concrete having the required workability and strength.

Because of the variability of concrete strengths the mix must be designed to have a considerably higher mean strength than the strength specified (Metcalf, 1970). The method of specifying concrete by its minimum strength has been replaced in British Standards and Codes of Practice such as BS 5328 and BS 8110 by a ‘characteristic strength’. The difference between the specified characteristic strength and the target mean strength is called the ‘margin’. Combined aggregate grading curves are not used in this method of mix design which refers instead to the percentage of fine aggregate passing the 600 µmm test sieve. The higher the percentage passing the 600 µmm test sieve, the finer the fine aggregate. The most fundamental way to specify mix parameters is in terms of the absolute volumes of the different materials required in a concrete mix. This method of mix design results in the mix being specified in terms of the mass in kilograms of the different materials required to produce one cubic meter of finished concrete (Teychenne et al., 1997).

The design process is divided into five stages. Each of these stages deals with a particular aspect of the design and ends with an important parameter or final unit proportions.

Stage 1 deals with strength leading to the free-water/ cement ratio.

Stage 2 deals with workability leading to the free-water content.

Stage 3 combines the results of Stages 1 and 2 to give the cement content.

Stage 4 deals with the determination of the total aggregate content.

Stage 5 deals with the selection of the fine and coarse aggregate contents.

The preceding design method determines a set of mix proportions for producing a concrete that has approximately the required properties of strength and workability. The method, however, is based on simplified classifications for type and quality of the materials and it still remains to check whether or not the particular aggregates and cement selected for use in a given case will behave as anticipated. This is the object of making the trial mix, and the subsequent feedback of information from the trial mix is an essential part of the mix design process.

According to Teychenne et al. (1997), in order to avoid the possible delay caused by a need to prepare a second trial mix as a result of strength tests, it may be expedient to prepare two or more initial trial mixes with the same water content but with different water/cement ratios. Normally, when typical materials are to be used, a single trial mix would be sufficient. Adjustments to be made to the original mix proportions, if necessary, will differ according to how much the results of the trial mixes differ from the designed values which will depend partly upon how typical the materials are of their classifications. Depending on these, there are three courses of action open:

- To use the trial mix proportions in the production mixes
- To modify the trial mix proportions slightly in the production mixes
- To prepare further trial mixes incorporating major changes to the mix proportions.

2.6.2.2 ACI Method of Mix Design

This is the most common method in use and is established by ACI recommended practice 211.1. It has advantages of simplicity in that it applies equally well and with more or less identical procedure irrespective of the shape and weight of aggregate and air entrainment feature of concrete. In usual practice ACI method of mix design is actually the combination of experience and laboratory investigation aided absolute method of mix design.

This method is based on determining the coarse aggregate content based on, dry rodded coarse aggregate bulk density and fineness modulus of sand. It takes into account the actual voids in compacted coarse aggregates that are to be filled by sand and water. It is most suitable for design of plum concrete, air-entrained concrete and gives separate values of water and sand content for maximum size of aggregate up to 150mm. It also gives values for 12.5 and 25mm down coarse aggregate (Sreejith, 2009). Two procedures are followed which involve a sequence of nine steps given in chapter 3, the first six steps being common.

The following limitations have been identified with ACI method:

i. It gives coarse aggregate content for sand with FM (Fineness Modulus) range of 2.4 to 3.0. It is found that sand available in most parts of the world is extremely coarse with FM more than 3.2.

ii. The density of fresh concrete is not given as function of specific gravity of its ingredients.

iii. The values of density of fresh concrete given in this method range from 2285Kg/m3 for 10mm down aggregate to 2505Kg/m3 for 150mm down coarse aggregate. The weights calculated from the given densities often result in higher cement contents than that assumed.

iv. The ACI method does not take into account the effect of the surface texture and flakiness of aggregate an sand and water content, neither does it distinguish between crushed stone aggregates and natural aggregates.

v. It does not have specific method of combining 10mm aggregates with 20mm aggregates.

vi. The fine aggregate content cannot be adjusted for different cement contents. Hence the richer mixes and leaner mixes may have same proportion, for a given set of materials.

However, it must be remembered that this method will (like other design methods) provide only a first approximation of proportions, which must be checked by trial batches to adjust as necessary to produce the desired concrete characteristics (Mehta and Monteiro, 2013).

2.6.2.3 Road Note 4 Method

In this method, the aggregate to cement are worked out on the basis of type of aggregate, maximum size aggregate and different levels of workability. The relative proportion of aggregates is worked on basis of combined grading curves. This method facilitates use of different types of fine and coarse aggregates in the same mix. The relative proportion of these can be easily calculated from combined grading curves. The values of aggregate to cement ratio are available for angular rounded or irregular coarse aggregate (Sreejith, 2009).

This method has the following limitations:

i. Road note 4 method leads to very high cement content and thus is becoming obsolete.

ii. In many cases, use of gap graded aggregate becomes unavoidable. In many parts of the world the practice is to use 20mm coarse aggregates without 10mm aggregate. This is because of poor quality of 10mm aggregates produced from jaw crusher. Gap grading does not fit into the standard combined grading curves of this method.

iii. It is difficult to adjust the sand content to match any of the standard combined grading curves. This is because sand available in some parts of the world is graded such that it is high on coarse fraction (1.18mm and above) and low on fines (600µm and below). The combined grading curve often cuts across more than one standard curves in such cases.

iv. Different aggregate to cement ratios are given for different levels of workability ranging from low to high. But these levels of workability are not different in terms of slump, compacting factor or Vee Bee time as in cases of other methods.

v. The fine aggregate content cannot be adjusted for different cement contents. Hence the richer mixes and leaner mixes may have same sand proportion, for a given set methods.

2.6.3 Statistical Experimental Method

The method of statistical experimental design involves the use of planned approach to experimental investigation. It makes use of theory of statistics and some specified laboratory results from practical experiments to formulate a mathematical model (equation), which will later be used to predict concrete mix ratio when desired target strength is known. The long run advantage will always pay off or compensate the initial input and justifies it. Unlike the empirical method, there won't be any further trial mixes once the model has been formulated.

The limitation to this method is the model is always particularized and cannot be used as a generalized model. For example, if a model is formulated for sandstone aggregate concrete, it cannot be used to predict the concrete mixture ratio involving aggregate other than the sandstone. Some statistical experimental methods are Simplex design (Obam, 1998), Axial design, Mixture experiments involving process variables, Mixture model with inverse terms (Draper and St. John, 1997), K-model (Draper and Pukelsheim, 1999), Artificial neural network, and Regression models.

2.6.3.1 Simplex Design

Scheffe (1958) was the first person to introduce simplex lattice design. Later in 1963 he also introduced simplex centroid design. Simplex is a factor space or a polygon. The simplest factor space is a straight line comprising of two components at both ends. The points within a simplex lattice (space factor) are symmetrically arranged and equidistant from the centroid. In the design a suitable polynomial will be chosen to represent the response surface of the entire space. The number of points on the space corresponds to the number of parameters in the chosen polynomial equation. The difference between the simplex lattice and simplex centroid design is, in simplex lattice design the points considered either fall at the vertices of the factor space or at the edges, while in the centroidal design one of the points must fall at the centre of the factor space.

Scheffe's method of optimization is applicable to mixtures in which the desired response depends on the proportion of components present in the mixture, rather than on the quality of the mixture (Scheffe, 1958). The use of the optimization model eliminated the arbitrary choice of concrete design mixes and its associated disadvantages. It also yielded optimum concrete mixtures which minimizes cost and satisfies specific performance requirement (Ezeh and Ibearugbulem, 2009; Ibearugbulem, 2006; Okere et al., 2013).

Onwuka et al. (2013) developed a mathematical model to predict and optimize compressive of Sawdust ash-cement concrete using Scheffe's five component second degree simplex lattices. The model was used to optimize the compressive strength of concrete made from water, cement, sawdust ash, sand and granites. Mbadike and Osadebe (2013) used Scheffe's model in optimization of compressive strength of lateritic concrete.

In their study, Anyaogu and Ezeh (2013) developed a mathematical model for optimizing the compressive strength of fly ash blended cement concrete based on Scheffe's Simplex Polynomial theory. The five component second degree (5, 2) mathematical model compared favorably with the experimental data. These researchers all concluded after testing their predictions with the statistical Fischer test that Scheffe's simplex model was adequate for the optimization of concrete/mortar at 95% confidence level.

2.6.3.2 Axial Design

The points in simplex designs fall on the surface (vertices and edges) and only one point falls at the centre. The axial design is an improvement over the simplex design because most of the points fall inside the factor space (Ibearugbulem, 2006).

2.6.3.3 Mixture Experiments Involving Process Variables

This method is an addition to simplex design. In simplex design the response only depends on the mixture components present in the mixture. But this method in addition to the proportion of mixture considers the process variable. The process variables are factors involved in the experiment that do not form part of the mixture, however, they influence interaction of the mixture ingredients. The process here involves two small designs. One, the design is meant for mixture components and the other is for process variables.

2.6.3.4 Mixture Models with Inverse Terms

Draper and St. John (1997) introduced the method of mixture model with inverse terms to statistical experiment method. The only difference between this model and simplex design is in the addition of inverse terms. The inverse terms take care of the extreme change in response as a component move towards zero.

2.6.3.5 K - Model

Draper and Pukelsheim (1999) developed K – Model of mixture design. The model is quite different from the simplex design and all the above mentioned designs. It makes use of kronecker algebra of vectors and matrices. The use of any factor space is not involved at all.

2.6.3.6 Artificial Neural Network

Artificial Neural Network (ANN) is a network of artificial neurons, an information processing units is inspired by the way in which human brain performs a particular task or function of interest. A neural network is a computational method inspired by studies of the brain and nervous systems in biological organism. It represents highly ideological mathematical models of our present understanding of such complex system. Artificial Neural Network models have the ability to learn and generalize the problems even when input data contain error or incomplete. Developments in artificial intelligence and computer sciences enable problems in studies on earth sciences to be modeled with increasing reliability. With the modeling results, evaluation of the forms of behavior observed in the nature strengthened empirical approaches. This caused artificial intelligence applications to become more favorable (Feng et al., 1997).

ANNs are made up of artificial neural cells called neurons. Artificial neural cells are units including a set of data and processing various inputs from external sources or from other neurons. Neurons are the basic parts of the general architecture used to calculate an output. ANN models are specified by net topology, node characteristics and training or learning rules. These rules specify an initial set of weights and indicate how weights should be adopted during use of improve performance. A crude classification can be done on the grounds of type input patterns (i.e. either binary inputs or continuous valued inputs). This can further be refined on the basis of types of learning (i.e. supervised on unsupervised).

The main phase in neural network is learning. The programming for adjusting the weights is called training and the training can be done either by given weights computed from a set of training data or by automatically adjusting the weights according to some criterion. Learning performance is improved by iteratively updating the weight in the network. The learning of ANN is generally classified as:

i. Supervised Learning: ANN is trained as a set of input-output patterns. The weights are adjusted to minimize the error between network output and actual output.

ii. Unsupervised Learning: In unsupervised training the network adjusts its weights in response to input patterns without the benefit of target answers.

Reinforcement learning is a variant of supervised learning in which network attempts to learn the input-output patterns through trial and error with a view to minimize a performance index called the reinforcement signal. The targets values are not provided for learning instead of error signal of the output are given to the ANN.

Researchers have developed artificial networks for the prediction of properties concrete. Nwobi-okoye et al. (2013) carried out a research which sought to develop a computational model based on artificial neural networks for the determination of the compressive strength of concrete materials made from a prevalent coarse aggregate component from Nigeria. The work involved building a multilayer perception neural network model which was trained using experimental data obtained from compressive strength test of concrete made from granite. The result of the study has ably demonstrated a cheap, simple, very quick and accurate alternative to experimental method of concrete strength determination. This method is also found to be simpler and quicker than analytical methods based on regression analysis. Serban (2009) concluded that the compressive and flexural tensile strength values of mortars containing various amounts of class C fly ash can be predicted in quite short period of time with tiny error rates by using the multilayer feed-forward neural network models than regression techniques.

2.6.3.7 Regression Method

Regression analysis is a statistical tool for the investigation of relationships between variables. The researcher seeks to ascertain the casual effect of one variable upon another. It is widely used for prediction and forecasting, where its use has substantial overlap with the field of machine learning (Armstrong, 2012). Regression analysis is also used to understand which among the independent variables are related to the dependent variable, and to explore the forms of these relationships.

A set of parameters X1, X2, X3, …, Xn, known as predictors can be used to predict the probable value of a dependent variable, Y with a particular degree of certainty. This means that so long as the values of the predictors are know the corresponding value of the dependent variable can be predicted with some degree of certainty (Ibearugbulem, 2006). Many techniques have been developed for carrying out regression analysis over the years. Familiar methods such as linear regression and ordinary least squares regression are parametric, in that the regression function is defined in terms of a finite number of unknown parameters that are estimated from the data. Non-parametric refers to techniques that allow the regression function to lie in a specified set of functions, which may be infinite-dimensional. Ibearugbulem’s improved regression model and Osadebe’s regression model fall under the category of non-parametric regression analysis.

The performance of regression analysis methods in practice depends on the form of the data generating process, and how it relates to the regression approach being used. Since the true form of the data-generation process is generally not known, regression analysis often depends to some extent on making assumptions about this process. These assumptions are sometimes testable if a sufficient quantity of data is available. Regression models for prediction are often useful even when the assumptions are moderately violated, although they may not perform optimally. However, according to David (2005), in many applications, especially with small effects or questions of causality based on observational data, regression analysis methods can give misleading results.

Onwuka et al. (2011) formulated a mathematicalmodel based on modified regression which can predict the strength of concrete if the mix proportions are specified. Osadebe (2003) developed a generalized mathematical model for optimizing compressive strength of normal concrete as a multivariant function. Osadebe et al. (2004) used the developed model in optimization of compressive strength of sand-laterite blocks blocks. Osadebe and Ibearugbulem (2008) further modified the model to optimize the compressive strength of periwinkle shell-granite concrete. Scheffe’s model and Osadebe’s model are statistical methods of concrete mix design most frequently used in Civil Engineering and are quite suitable for concrete mix optimization, but are greatly limited in that a predetermined number of experiments must be carried out in other to formulate them and they can only be applied for mix ratios that fall within the predetermined observation points. They cannot be used to optimize an already conducted series of laboratory tests.

The search for alternative method that can overcome the limitation identified above led to the development of a new regression model for optimizing concrete mixes called – Ibearugbulem’s Regression Model (Ibearugbulem et al., 2013). In this method few points of observation is used to formulate the model. Once the model has been formulated and tested for “goodness of fit”, it can then be used to predict future values of the dependent variable.

Polynomial Response Function

Osadebe (2003) assumed that the response function, F(z) is continuous and differentiable with respect to its predictors, Z_i. By making use of Taylor’s series the response function could be as shown in Equation 2.1.

$$F(z) = \sum \frac{F^{m}(z_0)*(z_i - z_0)^{m}}{m!} \tag{2.1}$$

Where $0 \leq m \leq \infty$

Since $F^{m}(z_0)$ is the derivative of the function $F(z_0)$ to m degree, Equation 2.1 can be rewritten as in Equation 2.2.

$$F(z) = \sum \frac{d^{m}F(z_0)}{dZ_0{}^{m}} * \frac{(z_i - z_0)^{m}}{m!} \tag{2.2}$$

$0 \leq m \leq \infty$, $2 \leq m \leq \infty$

The number of terms in Equation 2.2 is dependent on the degree of polynomial, m, and the number of independent variables, i. Taking m equal to 1, Equation 2.2 can be written as in Equation 2.3.

$$F(z) = \sum \frac{d^{0}F(z_0)}{dZ_0{}^{0}} * \frac{(z_i - z_0)^{0}}{0!} + \sum \frac{dF(z_0)}{dz_0} * \frac{(z_i - z_0)}{1!} \tag{2.3}$$

$0 \leq m \leq \infty$, $2 \leq m \leq \infty$

If m is equal to 2, expanding Equation 2.3 up to the second order gives Equation 2.4.

$$F(z) = \sum \frac{d^{0}F(z_0)}{dZ_0{}^{0}} * \frac{(z_i - z_0)^{0}}{0!} + \sum \frac{dF(z_0)}{dz_0} * \frac{(z_i - z_0)}{1!} + \sum \frac{d^{2}F(z_0)}{dZ_0{}^{2}} * \frac{(z_i - z_0)^{2}}{2!} \sum \frac{d^{2}F(z_0)}{dZ_0{}^{2}} * \frac{(z_i - z_0)^{2}(z_i - z_j)}{2!} \tag{2.4}$$

$0 \leq m \leq \infty$, $2 \leq m \leq \infty$

It is assumed that the origin is z_0, which is equal to zero. Since the products and quotients of constants are themselves constants, this equation can be written simply as shown in Equation 2.5.

$$F(z) = \sum b_m \, . \, z_i{}^{m} \tag{2.5}$$

$0 \leq m \leq \infty$, $2 \leq m \leq \infty$

It can be seen from Equation 2.5 that:

For m = 0, $\quad b_m = b$ (2.6)

For m = 1, $b_m = b_i$ (2.7)

For m = 2, $b_m = b_{ii}$ (for z_i^2 term) (2.8)

$b_m = b_{ij}$ (for $z_i z_j$ term) (2.9)

For m = 3, $b_m = b_{iii}$ (for z_i^3 term) (2.10)

$b_m = b_{ijk}$ (for $z_i z_j z_k$ term) (2.11)

$b_m = b_{iij}$ (for $z_i^2 z_j$ term) (2.12)

$b_m = b_{ijj}$ (for $z_i z_j^2$ term) (2.13)

$b_m = b_{iik}$ (for $z_i^2 z_k$ term) (2.14)

$b_m = b_{ikk}$ (for $z_i z_k^2$ term) (2.15)

$b_m = b_{jjk}$ (for $z_j^2 z_k$ term) (2.16)

$b_m = b_{jkk}$ (for $z_j z_k^2$ term) (2.17)

Equation (3.10) can also be written as in Equation 2.18.

$F(z) = b_0 + \sum bm \,.\, zim$ (2.18)

$1 \leq m \leq \infty$, $2 \leq m \leq \infty$

For $i = n$, $1 \leq m \leq n$ (2.19)

The implication of Equation 2.19 is that the maximum degree of polynomial that can be used is equal to the number of independent variables, i.

Boundary Conditions

Both Scheffe, and Osadebe and Ibearugbulem restricted the summation of the independent variables to unity, as expressed in Equation 2.20.

$\sum zi = 1$ (2.20)

Scheffe also restricted the value of each arbitrary independent variable to between zero and one, as expressed in Equation 2.21.

$0 \leq m \leq 1$ (2.21)

2.6.3.8 Ibearugbulem's Regression Model

Multiplying Equation 2.20 by b_0 gives Equation 2.22.

$$b_0 = \sum b_0 z_i \tag{2.22}$$

Multiplying Equation 3.25 by z_i and rearranging gives Equation 2.23.

$$Z_i^2 = z_i - z_1 z_i - z_2 z_i - \ldots - z_i z_n \tag{2.23}$$

Multiplying Equation 3.25 by z_i^r and rearranging gives Equation 2.24.

$$Z_i^{r+1} = z_i^r - z_1 z_i^r - z_2 z_i^r - \ldots - z_i^r z_n \tag{2.24}$$

Taking the highest degree of the polynomial and substituting Equations 2.22 and 2.24 into Equation 2.18 and factorizing, making sure that every term has no independent variable of more than one degree will yield Equation 2.25, which is the new Ibearugbulem's regression model.

$$F(z) = \sum \alpha_i z_i + \sum \alpha_{ij} z_i z_j + \sum \alpha_{ijk} z_i z_j z_k + \ldots + \sum \alpha_{ijk\ldots\infty} z_i z_j z_k \ldots z_\infty \tag{2.25}$$

$1 \le i \le \infty$, $1 \le i \le j \le \infty$, $1 \le i \le j \le k \le \infty$, … , $1 \le i \le j \le k \le \ldots \le \infty$

For i = 2, Equation 2.25 can be expressed as in Equation 2.26.

$$F(z) = \alpha_1 z_1 + \alpha_2 z_2 + \alpha_{12} z_1 z_2 \tag{2.26}$$

For i = 3, Equation 2.25 can be expressed as in Equation 2.27.

$$F(z) = \alpha_1 z_1 + \alpha_2 z_2 + \alpha_3 z_3 + \alpha_{12} z_1 z_2 + \alpha_{13} z_1 z_3 + \alpha_{23} z_2 z_3 + \alpha_{123} z_1 z_2 z_3 \tag{2.27}$$

For i = 4, Equation 2.25 can be expressed as in Equation 2.28.

$$F(z) = \alpha_1 z_1 + \alpha_2 z_2 + \alpha_3 z_3 + \alpha_4 z_4 + \alpha_{12} z_1 z_2 + \alpha_{13} z_1 z_3 + \alpha_{14} z_1 z_4 + \alpha_{23} z_2 z_3 + \alpha_{24} z_2 z_4 + \alpha_{34} z_3 z_4 + \alpha_{123} z_1 z_2 z_3 + \alpha_{124} z_1 z_2 z_4 + \alpha_{134} z_1 z_3 z_4 + \alpha_{234} z_2 z_3 z_4 + \alpha_{1234} z\ z_2 z_3 z_4 \tag{2.28}$$

Pseudo and Actual Variables

The independent variables used in the regression function (Equation 2.25) are pseudo variables. They are not the actual variables. However, a relationship exists between the pseudo variables, z_i and the actual variables, s_i.

$$Z_i = s_i / S \tag{2.29}$$

$S = \sum s_i$ (2.30)

2.6.3.9 Coefficients of the Regression Function

Summing Equation 2.25 for n observation points gives Equation 2.31.

$$\sum_r F(z) = \sum_r \sum \alpha_i z_i + \sum_r \sum \alpha_{ij} z_i z_j + \dots \dots \qquad (2.31)$$

$1 \le r \le n$

Multiplying Equation 2.31 by z_w gives Equation 2.32.

$$\sum_r Z_w.F(z) = \sum_r \sum \alpha_i z_i \,.\, z_w + \sum_r \sum \alpha_{ij} z_i z_j \,.\, z_w + \dots \qquad (2.32)$$

Multiplying Equation 2.31 by $z_q z_s z_t$ …. gives Equation 2.33.

$$\sum_r Z_q z_s z_t F(z) = \sum_r \sum \alpha_i z_i \,.\, Z_q z_s z_t + \sum_r \sum \alpha_{ij} z_i z_j \,.\, Z_q z_s z_t + \dots \qquad (2.33)$$

Adding Equations 2.32 and 2.33 will give n simultaneous equations with n unknowns, represented in matrix form as shown in Equation 2.34a.

$$\begin{bmatrix} \sum_r z_1 . F(z) \\ \sum_r z_2 . F(z) \\ \sum_r z_3 . F(z) \\ \vdots \\ . \\ \sum_r z_1 z_2 z_3 \dots F(z) \end{bmatrix} = \begin{bmatrix} \sum_r \sum z_1 . z_1 & \sum_r \sum z_2 . z_1 & \sum_r \sum z_3 . z_1 & \dots & \dots \\ \sum_r \sum z_1 . z_2 & \sum_r \sum z_2 . z_2 & \sum_r \sum z_3 . z_2 & \dots & \dots \\ \sum_r \sum z_1 . z_3 & \sum_r \sum z_2 . z_3 & \sum_r \sum z_3 . z_3 & \dots & \dots \\ \vdots & \vdots & \vdots & & \\ . & . & . & & \\ \sum_r \sum z_1 . z_1 . z_2 & \dots & \sum_r \sum z_2 . z_1 . z_2 & \dots & \end{bmatrix} \begin{bmatrix} \alpha_1 \\ \alpha_2 \\ \alpha_3 \\ \vdots \\ . \\ \alpha_{123..} \end{bmatrix} \qquad (2.34a)$$

Solving the simultaneous equation expressed in Equation 2.34a gives the values of the coefficients of regression function in Equation 2.25. Equation 2.34 can be written in a short form as shown in Equation 2.34b.

$$[F(z).Z] = [CC][\alpha] \tag{2.34b}$$

Where, CC is always a symmetric matrix. For a mixture of three components, CC is a 7 x 7 matrix as shown in Equation 2.35.

$$\begin{vmatrix}
\sum\sum Z_1Z_1 & \sum\sum Z_1Z_2 & \sum\sum Z_1Z_3 & \sum\sum Z_1Z_1Z_2 & \sum\sum Z_1Z_1Z_3 & \sum\sum Z_1Z_2Z_3 & \sum\sum Z_1Z_1Z_2Z_3 \\
\sum\sum Z_1Z_2 & \sum\sum Z_2Z_2 & \sum\sum Z_2Z_3 & \sum\sum Z_1Z_2Z_2 & \sum\sum Z_1Z_2Z_3 & \sum\sum Z_2Z_2Z_3 & \sum\sum Z_1Z_2Z_2Z_3 \\
\sum\sum Z_1Z_3 & \sum\sum Z_2Z_3 & \sum\sum Z_3Z_3 & \sum\sum Z_1Z_2Z_3 & \sum\sum Z_1Z_3Z_3 & \sum\sum Z_2Z_3Z_3 & \sum\sum Z_1Z_2Z_3Z_3 \\
\sum\sum Z_1Z_1Z_2 & \sum\sum Z_1Z_2Z_2 & \sum\sum Z_1Z_2Z_3 & \sum\sum Z_1Z_1Z_2Z_2 & \sum\sum Z_1Z_1Z_2Z_3 & \sum\sum Z_1Z_2Z_2Z_3 & \sum\sum Z_1Z_1Z_2Z_2Z_3 \\
\sum\sum Z_1Z_1Z_3 & \sum\sum Z_1Z_2Z_3 & \sum\sum Z_1Z_3Z_3 & \sum\sum Z_1Z_1Z_2Z_3 & \sum\sum Z_1Z_1Z_3Z_3 & \sum\sum Z_1Z_2Z_3Z_3 & \sum\sum Z_1Z_1Z_2Z_3Z_3 \\
\sum\sum Z_1Z_2Z_3 & \sum\sum Z_2Z_2Z_3 & \sum\sum Z_2Z_3Z_3 & \sum\sum Z_1Z_2Z_2Z_3 & \sum\sum Z_1Z_2Z_3Z_3 & \sum\sum Z_2Z_2Z_3Z_3 & \sum\sum Z_1Z_2Z_2Z_3Z_3 \\
\sum\sum Z_1Z_1Z_2Z_3 & \sum\sum Z_1Z_2Z_2Z_3 & \sum\sum Z_1Z_2Z_3Z_3 & \sum\sum Z_1Z_1Z_2Z_2Z_3 & \sum\sum Z_1Z_1Z_2Z_3Z_3 & \sum\sum Z_1Z_2Z_2Z_3Z_3 & \sum\sum Z_1Z_1Z_2Z_2Z_3Z_3
\end{vmatrix} \tag{2.35}$$

CHAPTER THREE

MATERIALS AND METHODS

3.1 Materials

Four primary constituents (materials) of concrete were used in this research work. The materials are cement, water, sand, and coarse aggregate (sandstone).

3.1.1 Cement

Ibeto brand of Ordinary Portland Cement, produced in Nigeria with properties in conformity with the requirements of BS 12 (1978) was used in preparation of concrete cube specimen. The cement grade was 32.5, corresponding to 32.5 MPa 28th day strength.

3.1.2 Water

The water used for this work was taken from bore-hole tap from Federal Polytechnic Nekede, Imo State. It was clean and safe for drinking in accordance with the requirements of ASTM C 1602 (2004).

3.1.3 Sand

These are aggregates passing sieve No. 4 (4.75mm) and are predominantly retained on sieve No. 200 (75µm). The sand used in this work was obtained from Otamiri River in Nekede, Owerri. It was obtained wet and then dried before use. The sand was tested and prepared in accordance with standards specified by BS 882, 1973 and ASTM 136, 2004). It was also free from deleterious matters. The sand was found to be normal weight natural aggregate with bulk density of 1745.78kg/m^3 (compacted) and 1610.35kg/m^3 (non-compacted), void ratio of 0.385 and specific gravity of 2.618. It fell in range of 212µmm – 2.36mm and zone 2 of the grading curve. The grain size distribution of the sand is shown in Figure 3.1.

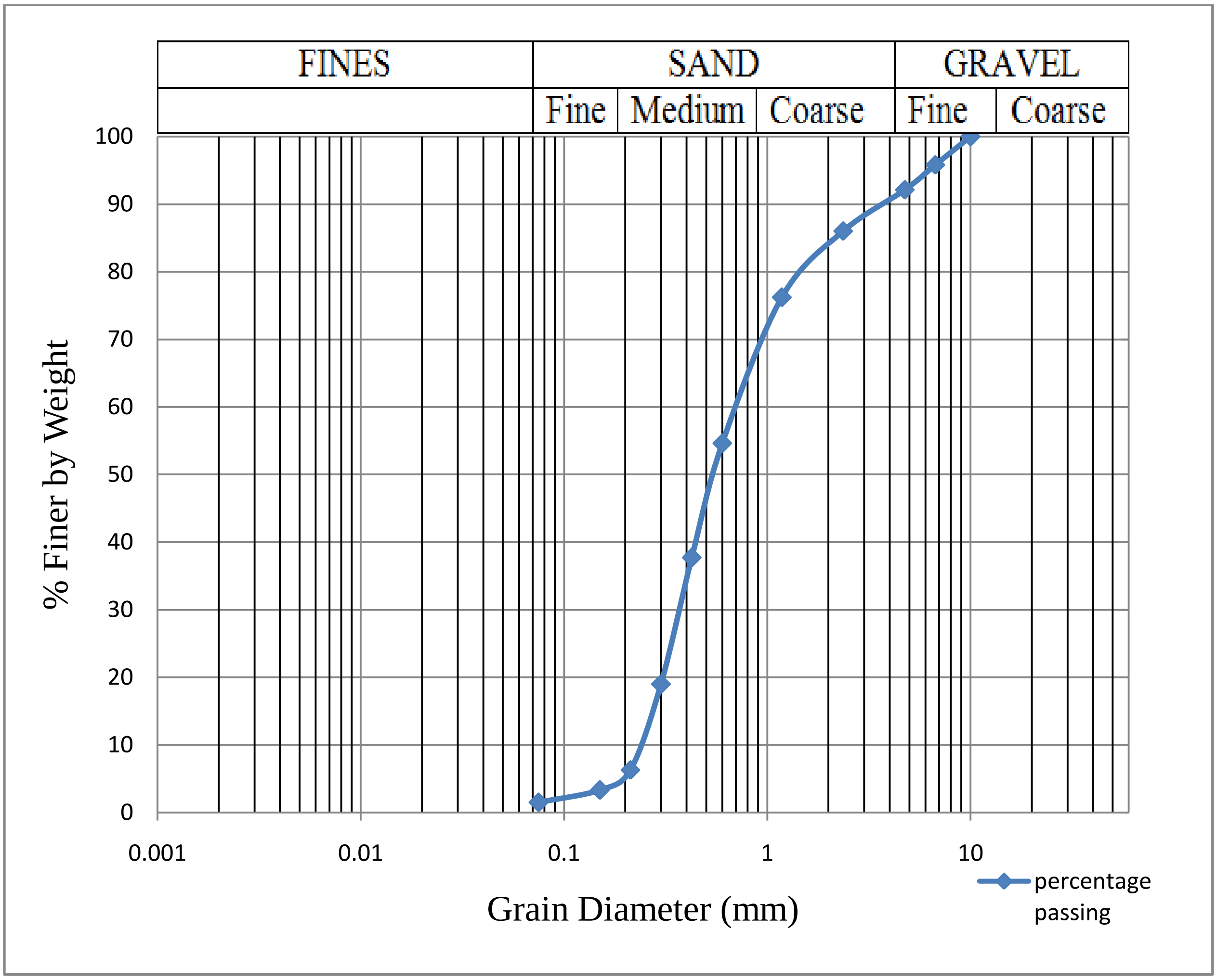


Figure 3.1 Grain Size Distribution of Sand

3.1.4 Coarse Aggregate

Sandstone was used for this research work. It was obtained from Ikperejere, a village in Ihitte Uboma Local Government Area Council of Imo State, Nigeria. The sandstone was obtained wet, then dried, and care was taken to remove deleterious matters in order not to compromise the integrity of the concrete produced. The sandstone was tested and prepared in accordance with standards specified by BS 882, 1973 and ASTM 136, 2004). The sandstone was normal weight natural coarse aggregate with bulk density of 1736.39kg/m^3 (compacted) and 1438.72kg/m^3 (non-compacted), void ratio of 0.418, water absorption capacity of 2.06% and specific gravity of 2.467. It has impact value of 15.31%, abrasion value of 17.92% and attrition value of 11.46% making it suitable for all forms of concreting operations. It fell in range of 19mm – 25mm and zone 3 of the grading curve. The grain size distribution of sandstone is shown in Figure 3.2.

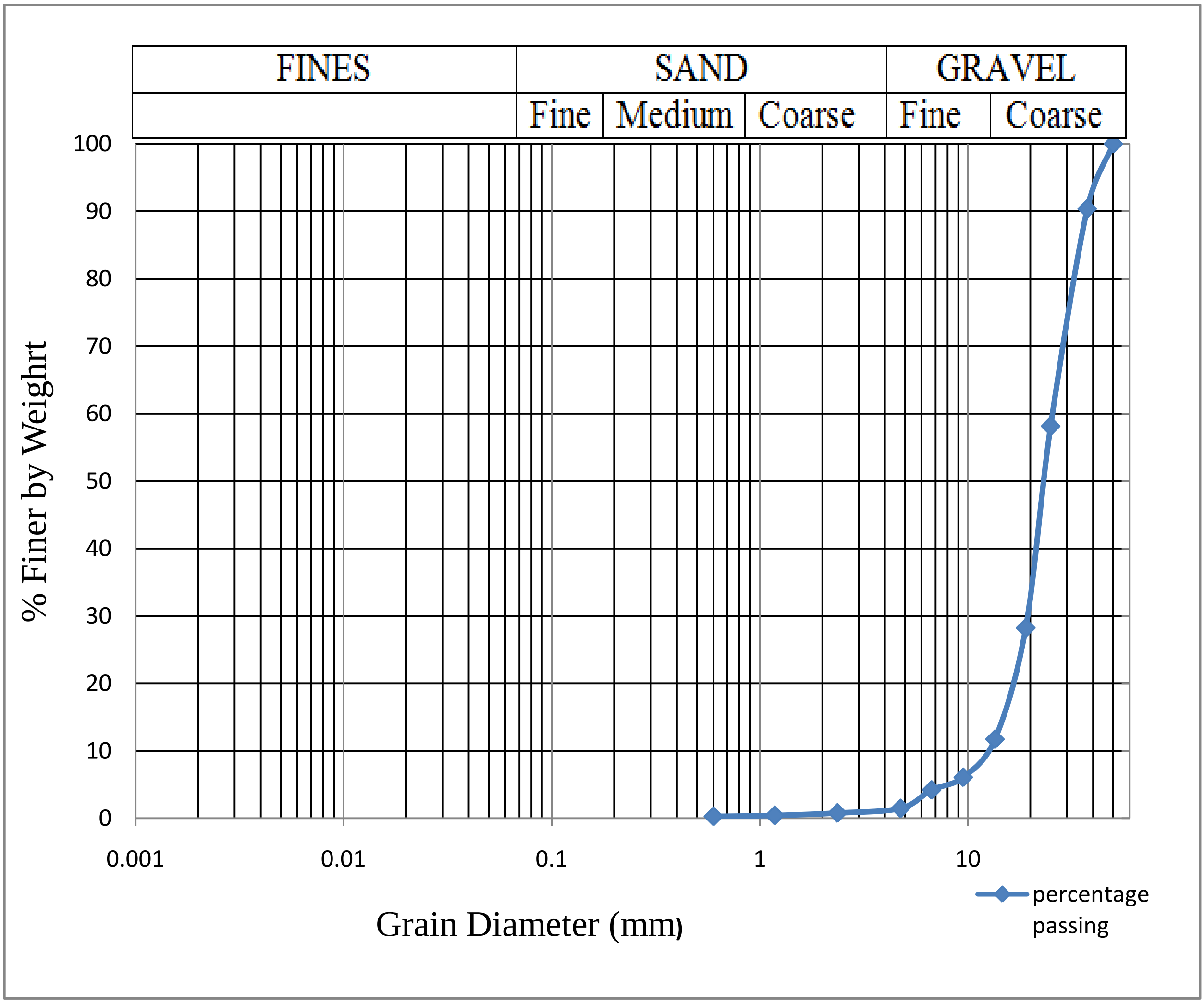


Figure 3.2 Grain Size Distribution of Sandstone

3.2 Method of Experiment

The test methods used to evaluate the properties of concrete and its constituent materials conformed to the relevant British Standard test procedures (BS 1881, 1986 and BS 812, 1990) for strength and mechanical test on aggregates respectively.

3.2.1 Characterization Tests of Aggregates

The tests on physical properties of the materials used for this work were carried out at the Concrete Laboratory of Civil Engineering Department, Federal Polytechnic, Nekede. Specific gravity, bulk density, water absorption, void ratio and sieve analysis of sand and sandstone were tested. Other tests include; impact value, abrasion and attrition values. The methods were in accordance with relevant standards which include; ASTM C29, 2004; ASTM C125, 2003; ASTM C33, 2004).

3.2.2 Concrete Test Procedure

The procedures employed are presented herein in accordance with relevant standards.

3.2.2.1 Concrete Mix Design

The BS method of design of normal concrete mixes developed by Teychenne et al. (1997) was employed. It is restricted to designing concrete mixes to meet workability, compressive strength and durability requirements using Portland cements complying to BS 12 and natural aggregates complying with BS 882. The following requirements are specified and thus entered under the relevant item on the mix design form, as shown in Table 3.1.

i. Characteristic compressive strength, 20 N/mm^2 at 28 days, with a 10% defective rate (k = 1.28) - Item 1.1
ii. Portland cement class 32.5 - Item 1.5
iii. Slump required, 10–30 mm - Item 2.1
iv. Maximum aggregate size, 40 mm - Item 2.2
v. Maximum free-water/cement ratio, 0.6 - Item 1.8
vi. Minimum cement content, 290 kg/m^3 - Item 3.3

The figures and tables referenced in the calculation sheet can be found in Appendix C.

Table 3.1 Mix Design Calculation Sheet

Stage	Item	Reference or calculation	Values
1	1.1 Characteristic strength	Specified	…………20………..… N/mm^2 at …………28 days
			Proportion defective ……………………………10 %
	1.2 Standard deviation	Fig 3	…………..…… N/mm^2 or no data ……4…… N/mm^2
	1.3 Margin	C1	(k = 1.28) 1.28 × 4 = 5 N/mm^2
	1.4 Target mean strength	C2	…20… + …5… = …25… N/mm^2
	1.5 Cement strength class	Specified	32.5
	1.6 Aggregate type: coarse		Crushed
	Aggregate type: fine		Uncrushed
	1.7 Free-water/cement ratio		…………0.45....………
			Use 0.55
	1.8 Maximum free-water/ cement ratio	Specified	…………0.6...…………
2	2.1 Slump	Specified	Slump ……………………….. 10 – 30 mm
	2.2 Maximum aggregate size	Specified	………………………………..…….……….. 40mm
	2.3 Free-water content	Table 3	………………………………….……… 140 kg/m^3
3	3.1 Cement content	C3	……..175…… ÷ …0.55… = …………. 318 kg/m^3
	3.2 Maximum cement content	Specified	..…..…. kg/m^3
	3.3 Minimum cement content	Specified	………290 kg/m^3

Use 3.1 if ≤ 3.2

Use 3.3 if > 3.1 …… 318 kg/m^3

3.4 Modified free-water/cement ratio ……………………………………………………

---

4 4.1 Relative density of aggregate (SSD) ……2.6……known/~~assumed~~

4.2 Concrete density Fig 5 …… 2400 kg/m^3

4.3 Total aggregate content C4 2400 – 318 – 175 = 1907 kg/m^3

5 5.1 Grading of fine aggregate Percentage passing 600 μm sieve ……..…………………..… 54.6 %

5.2 Proportion of fine aggregate Fig 6 ………………………………30 to 40, say……33.4 %

5.3 Fine aggregate content } C5 { 1907 × 0.334 = 636 kg/m^3

5.4 Coarse aggregate content } C5 { 1907 – 636 = 1271 kg/m^3

Quantities	**Water (kg)**	**Cement (kg)**	**Fine aggregate (kg)**	**Coarse aggregate (kg) 10 mm 20 mm 40 mm**
Per m^3 (to nearest 5 kg)	175	318	636	1271
Per trial mix of 0.05 m^3	8.75	15.9	31.8	63.6

From the mix design proportions:

Water-cement Ratio = 175/318 = 0.55

Cement Ratio = 318/318 = 1

Fine Aggregate Ratio = 636/318 = 2

Coarse Aggregate Ratio = 1271/318 = 4

Therefore, mix ratio = 0.55:1:2:4

3.2.2.2 Batching of Materials

Batching in concrete is the proportioning of individual quantities of materials to be used in concrete production. By varying the proportions of the mix components, 44 mix ratios were developed from the 0.55:1:2:4 mix ratio derived from the concrete mixture design (Table 3.1). By varying the proportions of the key factors affecting compressive strength of concrete, namely, water/cement ratio (0.45, 0.5, 0.55, 0.6) and fine aggregate/coarse aggregate ratio(1.5:4.5, 1.75:4.25, 2:4, 2.25:3.75, 2.5:3.5, 2.75:3.25, 3:3, 3.25:2.75, 3.5:2.5, 3.75:2.25, 4:2) 4 w/c ratios and 11 solid mix ratios were obtained. Proportion of cement was kept constant and the 4 water-cement ratios were applied on the 11 developed solid mix ratios to get a total of 44 concrete mix ratios as contained in Table 3.2. Batching of different constituent materials was by mass using a weighing balance of 50kg capacity and the developed mix ratios given in Table 3.2. The procedure adopted to arrive at the different masses of the constituent materials given in Table 3.3 is outlined below:

Volume of prototype concrete cube = 150x150x150 = 0.003375m^3

Adding 10% for waste;

Volume of prototype cube = 0.0037125m^3,

For 3 samples, volume = 0.0111375m^3

Unit weight of concrete = 24KN/m^3

Density of concrete = 2400/9.8 = 2448.98Kg/m^3

Mass of 3 sample cubes = 2448.98 x 0.011137

= 27.275515Kg

Total mass for a single mix = 27.28Kg

Using an approximate mass of 29kg and the developed mix ratios given in Table 3.2, the masses for three samples of prototype concrete cubes were obtained for different mix ratios as given in Table 3.3. The constituent materials were batched using a 50Kg weighing balance as can be seen in Plate B.2 to B.5 in appendix B.

Table 3.2 Mix Design Ratios for Concrete Cubes

S/N	Water	Cement	Sand	Sandstone
1	0.45	1	1.5	4.5
2	0.45	1	1.75	4.25
3	0.45	1	2	4
4	0.45	1	2.25	3.75
5	0.45	1	2.5	3.5
6	0.45	1	2.75	3.25
7	0.45	1	3	3
8	0.45	1	3.25	2.75
9	0.45	1	3.5	2.5
10	0.45	1	3.75	2.25
11	0.45	1	4	2
12	0.5	1	1.5	4.5
13	0.5	1	1.75	4.25
14	0.5	1	2	4
15	0.5	1	2.25	3.75
16	0.5	1	2.5	3.5
17	0.5	1	2.75	3.25
18	0.5	1	3	3
19	0.5	1	3.25	2.75
20	0.5	1	3.5	2.5
21	0.5	1	3.75	2.25
22	0.5	1	4	2
23	0.55	1	1.5	4.5
24	0.55	1	1.75	4.25
25	0.55	1	2	4
26	0.55	1	2.25	3.75
27	0.55	1	2.5	3.5
28	0.55	1	2.75	3.25
29	0.55	1	3	3
30	0.55	1	3.25	2.75
31	0.55	1	3.5	2.5
32	0.55	1	3.75	2.25
33	0.55	1	4	2
34	0.6	1	1.5	4.5
35	0.6	1	1.75	4.25
36	0.6	1	2	4
37	0.6	1	2.25	3.75
38	0.6	1	2.5	3.5
39	0.6	1	2.75	3.25
40	0.6	1	3	3
41	0.6	1	3.25	2.75
42	0.6	1	3.5	2.5
43	0.6	1	3.75	2.25
44	0.6	1	4	2

Table 3.3 Mix Design Quantities for Concrete Cubes

S/N	Water (Kg)	Cement (Kg)	Sand (Kg)	Sandstone (Kg)
1	1.75	3.89	5.84	17.52
2	1.75	3.89	6.81	16.54
3	1.75	3.89	7.79	15.57
4	1.75	3.89	8.76	14.60
5	1.75	3.89	9.73	13.63
6	1.75	3.89	10.71	12.65
7	1.75	3.89	11.68	11.68
8	1.75	3.89	12.65	10.71
9	1.75	3.89	13.63	9.73
10	1.75	3.89	14.60	8.76
11	1.75	3.89	15.57	7.79
12	1.95	3.89	5.84	17.52
13	1.95	3.89	6.81	16.54
14	1.95	3.89	7.79	15.57
15	1.95	3.89	8.76	14.60
16	1.95	3.89	9.73	13.63
17	1.95	3.89	10.71	12.65
18	1.95	3.89	11.68	11.68
19	1.95	3.89	12.65	10.71
20	1.95	3.89	13.63	9.73
21	1.95	3.89	14.60	8.76
22	1.95	3.89	15.57	7.79
23	2.14	3.89	5.84	17.52
24	2.14	3.89	6.81	16.54
25	2.14	3.89	7.79	15.57
26	2.14	3.89	8.76	14.60
27	2.14	3.89	9.73	13.63
28	2.14	3.89	10.71	12.65
29	2.14	3.89	11.68	11.68
30	2.14	3.89	12.65	10.71
31	2.14	3.89	13.63	9.73
32	2.14	3.89	14.60	8.76
33	2.14	3.89	15.57	7.79
34	2.34	3.89	5.84	17.52
35	2.34	3.89	6.81	16.54
36	2.34	3.89	7.79	15.57
37	2.34	3.89	8.76	14.60
38	2.34	3.89	9.73	13.63
39	2.34	3.89	10.71	12.65
40	2.34	3.89	11.68	11.68
41	2.34	3.89	12.65	10.71
42	2.34	3.89	13.63	9.73
43	2.34	3.89	14.60	8.76
44	2.34	3.89	15.57	7.79

3.2.2.3 Mixing of Materials

Manual mixing was adopted with the help of spade. Water was first sprinkled on the banker (mixing surface) 2 hours before the actual mixing so as to saturate the surface to avoid absorbing the mixing water. The fine aggregate was first measured, followed by coarse aggregate. The aggregates were mixed dry before adding cement and then mixed. This was followed by the addition of water (clean) and the constituent materials were mixed thoroughly until homogeneity was attained.

3.2.2.4 Placement of Concrete

After mixing, the concrete was cast into steel moulds measuring 150mm x 150mm x 150mm for cubes. The cube moulds were first cleaned with a metallic brush and then wiped clean with a clean dry rag to rid it of all dirt and debris. With the use of a brush, a thin layer of engine oil was applied in the inner surface of the moulds to allow for easy removal of the cast concrete cubes. The freshly mixed concrete was immediately transferred to the moulds. It was poured in the cube mould in three layers and compacted by ramming the concrete 25 times at each layer. The pouring and compaction of concrete were carried out continuously till the mould was filled to the brim. The top of the cast was then leveled with a trowel and leveler.

The cast cubes were allowed to set in the mould for 24 hours, after which they were de-moulded and set for curing (see Plates B.7 and B.8 in appendix B).

3.2.2.5 Workability Test

Workability is the ease with which concrete can be worked. In this experimental research work, slump test was performed on the different mixes of the sandstone concrete. The slump cone was filled in three layers using the scoop, with each layer being about one-third of the volume of the cone. For the first layer of concrete that was poured, 16mm rod was used to tamp it 35 times for proper compaction. This was repeated for the second and third layers. The cone was then overfilled and the tamping rod was used to level it. The cone was removed immediately after the leveling and the slump (reduction in height) was measured using a 30cm meter rule as can be seen in

Plates B.10 and B.11 in appendix B. The test was conducted in accordance with the specification of ASTM C143 (2004).

3.2.2.6 Curing of Concrete

Curing was done in accordance with ASTM C192 (2004). The concrete cubes were removed from the moulds after 24 hours and completely immersed in an open water-filled curing tank. Curing was done at normal room temperature. The cubes were removed on the 28th day and dried at room temperature for 1 hour before testing. Plate B.9 in appendix B shows the concrete cubes being transferred to the open curing tank for the remainder of the curing period.

3.2.2.7 Crushing of Concrete Cubes

Crushing was carried out in accordance with BS 1881-115, 1986. In the compressive strength test, the measured concrete cubes were removed from the curing tank after 28 days and allowed to dry for one hour (Plate 3.12). It was weighed and concentrically placed in contact with the plate of the Okhard Digital Display Universal Testing Machine. The constant rate of loading was on the concrete cubes until fracture (failure) occurred. This can be seen in Plates B.13 and B.14 in appendix B. The load causing failure was recorded and Equation 3.1 gives expression of the compressive strength of cube.

$$\text{Compressive strength} = \frac{Load\ at\ failure\ (N)}{Cross{-}sectional\ area\ of\ cube\ (mm2)} \qquad (3.1)$$

Cross-sectional area of cube = 150 x 150mm

FC = Failure load (KN) / Nominal cross section area (m^2)

Where FC is the compressive strength

The saturated-surface-dry density of the concrete cubes is obtained from the equation:

SSDD = Mass (kg) / Volume of cube (m^3)

Where SSDD means saturated-surface-dry density

The results of compressive strength test of the concrete cubes are shown in Table 4.2 in chapter four.

3.3 Method of Analysis

The proposed approach to optimizing and predicting the compressive strength of sandstone concrete mixtures is based on the planned experimental works and statistical analysis of the data generated, which would reduce the number of trial batches needed.

3.3.1 Statistical Modeling

For the purpose of this research work, the method used in light of the methods reviewed in chapter two is the Ibearugbulem's regression method. The simplicity of the model formulation and fewer numbers of experimental mixes required informed the decision to adopt this method. Also the method is selected because of the ease in testing adequacy of model. The statistical model derived utilizing the experimental data can be used to obtain the optimal mixture proportions satisfying the specified compressive strength.

Table 3.4 was developed from Table 3.1 with an additional column for mix notation. Mixes with numbers labeled M1, M2, M3, etc. are the model mix ratios used to formulate the model while those with numbers labeled V1, V2, V3, etc. are the validation mix ratios used to validate the model. Table 3.4 shows the model mix ratios separated from the validation mix ratios.

Table 3.4 Model and Validation Mix Ratios

Mixture Proportion (Kg)					
Model Mix Ratios					
S/N	Mix Notation	Water	Cement	Sand	Sandstone
1	M1	0.45	1	1.5	4.5
2	M2	0.45	1	2	4
3	M3	0.45	1	2.5	3.5
4	M3	0.45	1	3	3
5	M5	0.45	1	3.5	2.5
6	M6	0.45	1	4	2

7	M7	0.5	1	1.75	4.25
8	M8	0.5	1	2.25	3.75
9	M9	0.5	1	2.75	3.25
10	M10	0.5	1	3.25	2.75
11	M11	0.5	1	3.75	2.25
12	M12	0.55	1	1.5	4.5
13	M13	0.55	1	2	4
14	M14	0.55	1	2.5	3.5
15	M15	0.55	1	3	3
16	M16	0.55	1	3.5	2.5
17	M17	0.55	1	4	2
18	M18	0.6	1	1.75	4.25
19	M19	0.6	1	2.25	3.75
20	M20	0.6	1	2.75	3.25
21	M21	0.6	1	3.25	2.75
22	M22	0.6	1	3.75	2.25
Validation Mix Ratios					
23	V1	0.45	1	1.75	4.25
24	V2	0.45	1	2.25	3.75
25	V3	0.45	1	2.75	3.25
26	V4	0.45	1	3.25	2.75
27	V5	0.45	1	3.75	2.25
28	V6	0.5	1	1.5	4.5
29	V7	0.5	1	2	4
30	V8	0.5	1	2.5	3.5
31	V9	0.5	1	3	3
32	V10	0.5	1	3.5	2.5
33	V11	0.5	1	4	2
34	V12	0.55	1	1.75	4.25
35	V13	0.55	1	2.25	3.75
36	V14	0.55	1	2.75	3.25
37	V15	0.55	1	3.25	2.75
38	V16	0.55	1	3.75	2.25
39	V17	0.6	1	1.5	4.5
40	V18	0.6	1	2	4
41	V19	0.6	1	2.5	3.5
42	V20	0.6	1	3	3
43	V21	0.6	1	3.5	2.5
44	V22	0.6	1	4	2

3.3.1.1 Mixture Components and Pseudo Variables

Because the proportion of cement was constant for all the mix ratios, three variable mixture components was used in the modeling namely; water/cement ratio (S_1), sand/cement ratio (S_2), and sandstone/cement ratio (S_3). From Equation 2.29 and 2.30:

$$Z_i = S_i/S \qquad (3.2)$$

Where, i = 1, 2, 3

$$S = S_1 + S_2 + S_3 \tag{3.3}$$

Using Equations 3.2 and 3.3, the values of the mixture components (S) the corresponding pseudo variables (Z) were obtained as shown in Table 3.5. The Z-matrix for model mixes was derived from the 22 mixes used for the model formulation as shown in Table 3.6.

Table 3.5 Values of Mix Components (S) and Pseudo Variables (Z)

Mix	S_1	S_2	S_3	S	Z_1	Z_2	Z_3	Z_1Z_2	Z_1Z_3	Z_2Z_3	$Z_1Z_2Z_3$
M1	0.45	1.5	4.5	6.45	0.06976	0.23255	0.69767	0.01622	0.04867	0.16224	0.01131
M2	0.45	2	4	6.45	0.06976	0.31007	0.62015	0.02163	0.04326	0.19229	0.01341
M3	0.45	2.5	3.5	6.45	0.06976	0.38759	0.54263	0.02704	0.03785	0.21032	0.01467
M3	0.45	3	3	6.45	0.06976	0.46511	0.46511	0.03244	0.03245	0.21633	0.01509
M5	0.45	3.5	2.5	6.45	0.06976	0.54263	0.38759	0.03785	0.02704	0.21032	0.01467
M6	0.45	4	2	6.45	0.06976	0.62015	0.31007	0.04326	0.02163	0.19229	0.01341
M7	0.5	1.75	4.25	6.5	0.07692	0.26923	0.65384	0.02071	0.05029	0.17603	0.01354
M8	0.5	2.25	3.75	6.5	0.07692	0.34615	0.57692	0.02662	0.04437	0.19970	0.01536
M9	0.5	2.75	3.25	6.5	0.07692	0.42307	0.5	0.03254	0.03846	0.21153	0.01627
M10	0.5	3.25	2.75	6.5	0.07692	0.5	0.42307	0.03846	0.03254	0.21153	0.01627
M11	0.5	3.75	2.25	6.5	0.07692	0.57692	0.34615	0.04437	0.02662	0.19970	0.01536
M12	0.55	1.5	4.5	6.55	0.08396	0.22900	0.68702	0.01922	0.05768	0.15733	0.01321
M13	0.55	2	4	6.55	0.08396	0.30534	0.61068	0.02563	0.05127	0.18646	0.01565
M14	0.55	2.5	3.5	6.55	0.08396	0.38167	0.53435	0.03204	0.04486	0.20395	0.01712
M15	0.55	3	3	6.55	0.08396	0.45801	0.45801	0.03845	0.03845	0.20977	0.01761
M16	0.55	3.5	2.5	6.55	0.08396	0.53435	0.38167	0.04486	0.03204	0.20395	0.01712
M17	0.55	4	2	6.55	0.08396	0.61068	0.30534	0.05127	0.02563	0.18646	0.01565
M18	0.6	1.75	4.25	6.6	0.09090	0.26515	0.64393	0.02410	0.05853	0.17074	0.01552
M19	0.6	2.25	3.75	6.6	0.09090	0.34090	0.56818	0.03099	0.05165	0.19369	0.01760
M20	0.6	2.75	3.25	6.6	0.09090	0.41666	0.49242	0.03787	0.04476	0.20517	0.01865
M21	0.6	3.25	2.75	6.6	0.09090	0.49242	0.41666	0.04476	0.03787	0.20517	0.01865
M22	0.6	3.75	2.25	6.6	0.09090	0.56818	0.34090	0.05165	0.03099	0.19369	0.01760
V1	0.45	1.75	4.25	6.45	0.06976	0.27131	0.65891	0.01892	0.04597	0.17877	0.01247
V2	0.45	2.25	3.75	6.45	0.06976	0.34883	0.58139	0.02433	0.04056	0.20281	0.01414
V3	0.45	2.75	3.25	6.45	0.06976	0.42635	0.50387	0.02974	0.03515	0.21483	0.01498
V4	0.45	3.25	2.75	6.45	0.06976	0.50387	0.42635	0.03515	0.02974	0.21483	0.01498
V5	0.45	3.75	2.25	6.45	0.06976	0.58139	0.34883	0.04056	0.02433	0.20281	0.01414
V6	0.5	1.5	4.5	6.5	0.07692	0.23076	0.69230	0.01775	0.05325	0.15976	0.01228
V7	0.5	2	4	6.5	0.07692	0.30769	0.61538	0.02366	0.04733	0.18934	0.01456
V8	0.5	2.5	3.5	6.5	0.07692	0.38461	0.53846	0.02958	0.04142	0.20710	0.01593
V9	0.5	3	3	6.5	0.07692	0.46153	0.46153	0.03550	0.03550	0.21301	0.01638
V10	0.5	3.5	2.5	6.5	0.07692	0.53846	0.38461	0.04142	0.02958	0.20710	0.01593
V11	0.5	4	2	6.5	0.07692	0.61538	0.30769	0.04733	0.02366	0.18934	0.01456
V12	0.55	1.75	4.25	6.55	0.08396	0.26717	0.64885	0.02243	0.05448	0.17335	0.01455
V13	0.55	2.25	3.75	6.55	0.08396	0.34351	0.57251	0.02884	0.04807	0.19666	0.01651
V14	0.55	2.75	3.25	6.55	0.08396	0.41984	0.49618	0.03525	0.04166	0.20832	0.01749
V15	0.55	3.25	2.75	6.55	0.08396	0.49618	0.41984	0.04166	0.03525	0.20832	0.01749
V16	0.55	3.75	2.25	6.55	0.08396	0.57251	0.34351	0.04807	0.02884	0.19666	0.01651
V17	0.6	1.5	4.5	6.6	0.09090	0.22727	0.68181	0.02066	0.06198	0.15495	0.01408
V18	0.6	2	4	6.6	0.09090	0.30303	0.60606	0.02754	0.05509	0.18365	0.01669
V19	0.6	2.5	3.5	6.6	0.09090	0.37878	0.53030	0.03443	0.04820	0.20087	0.01826
V20	0.6	3	3	6.6	0.09090	0.45454	0.45454	0.04132	0.04132	0.20661	0.01878
V21	0.6	3.5	2.5	6.6	0.09090	0.53030	0.37878	0.04820	0.03443	0.20087	0.01826
V22	0.6	4	2	6.6	0.09090	0.60606	0.30303	0.05509	0.02754	0.18365	0.01669

Table 3.6 Z-Matrix for Model Mixes

Mix	Z_1	Z_2	Z_3	Z_1Z_2	Z_1Z_3	Z_2Z_3	$Z_1Z_2Z_3$
M1	0.06976744	0.23255814	0.69767442	0.01622499	0.048675	0.162249865	0.01131976
M2	0.06976744	0.31007752	0.62015504	0.02163332	0.0432666	0.192296136	0.01341601
M3	0.06976744	0.3875969	0.54263566	0.02704164	0.0378583	0.210323899	0.01467376
M3	0.06976744	0.46511628	0.46511628	0.03244997	0.03245	0.216333153	0.01509301
M5	0.06976744	0.54263566	0.3875969	0.0378583	0.0270416	0.210323899	0.01467376
M6	0.06976744	0.62015504	0.31007752	0.04326663	0.0216333	0.192296136	0.01341601
M7	0.07692308	0.26923077	0.65384615	0.02071006	0.0502959	0.176035503	0.01354119
M8	0.07692308	0.34615385	0.57692308	0.02662722	0.0443787	0.199704142	0.01536186
M9	0.07692308	0.42307692	0.5	0.03254438	0.0384615	0.211538462	0.01627219
M10	0.07692308	0.5	0.42307692	0.03846154	0.0325444	0.211538462	0.01627219
M11	0.07692308	0.57692308	0.34615385	0.0443787	0.0266272	0.199704142	0.01536186
M12	0.08396947	0.22900763	0.6870229	0.01922965	0.0576889	0.157333489	0.01321121
M13	0.08396947	0.30534351	0.61068702	0.02563953	0.0512791	0.18646932	0.01565773
M14	0.08396947	0.38167939	0.53435115	0.03204941	0.0448692	0.203950819	0.01712564
M15	0.08396947	0.45801527	0.45801527	0.0384593	0.0384593	0.209777985	0.01761495
M16	0.08396947	0.53435115	0.38167939	0.04486918	0.0320494	0.203950819	0.01712564
M17	0.08396947	0.61068702	0.30534351	0.05127906	0.0256395	0.18646932	0.01565773
M18	0.09090909	0.26515152	0.64393939	0.02410468	0.0585399	0.170741506	0.01552196
M19	0.09090909	0.34090909	0.56818182	0.03099174	0.0516529	0.193698347	0.01760894
M20	0.09090909	0.41666667	0.49242424	0.03787879	0.0447658	0.205176768	0.01865243
M21	0.09090909	0.49242424	0.41666667	0.04476584	0.0378788	0.205176768	0.01865243
M22	0.09090909	0.56818182	0.34090909	0.05165289	0.0309917	0.193698347	0.01760894

Substituting the values from Table 3.6 (Z-matrix) into Equation 2.35 containing elements of CC matrix gives the CC matrix shown in Table 3.7 with its inverse in Table 3.8.

Table 3.7 CC Matrix

0.14241832	0.7421168	0.8770471	0.0599482	0.0708478	0.3438392	0.0277688
0.74211682	4.2350373	4.2987873	0.3383295	0.3438392	1.8375206	0.1468334
0.87704715	4.2987873	5.7866418	0.3438392	0.4623601	2.1174275	0.169237
0.05994817	0.3383295	0.3438392	0.0272911	0.0277688	0.1468334	0.0118472
0.07084784	0.3438392	0.4623601	0.0277688	0.037302	0.169237	0.0136578
0.34383919	1.8375206	2.1174275	0.1468334	0.169237	0.8456819	0.0675624
0.02776878	0.1468334	0.169237	0.0118472	0.0136578	0.0675624	0.0054501

Table 3.8 Inverse of CC Matrix

13410360.6	87086.414	91155.834	-15674746	-15726697	53078.7	-678170.8
87086.414	2536.7335	1824.2821	-126608.7	-117661.4	-7023.758	88438.31
91155.8345	1824.2821	1603.8774	-122080.5	-119294.8	-4781.1102	60191.611
-15674746	-126608.7	-122080.48	18636884	18579826	30799.873	-388137.5
-15726697	-117661.4	-119294.83	18579826	18600669	2554.2483	-29048.68
53078.6996	-7023.758	-4781.1102	30799.873	2554.2483	29217.474	-368296.4
-678170.84	88438.31	60191.611	-388137.5	-29048.68	-368296.37	4685923.8

3.3.1.2 RZ Vector for Compressive Strength Model

Using the values of Z_i from the Z-matrix of Table 3.6 and experimental compressive strength given in Table 4.2, the RZ vector was obtained for the compressive strength model as:

$\sum(F(z).Z_1) = 30.34747398$ (3.4)

$\sum(F(z).Z_2) = 151.8465549$ (3.5)

$\sum(F(z).Z_3) = 193.0759711$ (3.6)

$\sum(F(z).Z_1Z_2) = 12.29426383$ (3.7)

$\sum(F(z).Z_1 Z_3) = 15.57597287$ (3.8)

$\sum(F(z).Z_2 Z_3) = 73.06663959$ (3.9)

$\sum(F(z).Z_1 Z_2 Z_3) = 5.906785404$ (3.10)

3.3.1.3 Coefficients for the Compressive Strength Model

Using Equation 2.34b and by making α (the coefficient) the subject of the equation, coefficients of the regression model were obtained. That is, multiplying the CC inverse matrix give in Table 3.8 and the RZ vectors given in Equations 3.4 – 3.10, the coefficients for the model compressive strength of sandstone concrete were obtained as:

$\alpha1 = -1248.33976$ (3.11)

$\alpha2 = 204.1475408$ (3.12)

$\alpha3 = 207.1288857$ (3.13)

$\alpha12 = -1409.00007$ (3.14)

$\alpha13 = -1115.89088$ (3.15)

$\alpha23 = -1024.82821$ (3.16)

$\alpha123 = 14075.96126$ (3.17)

Substituting the values of the derived coefficients for the model compressive strength in Equations 3.11 - 17 into Equation 2.28, the model regression equation was obtained as:

$F(z) = -1248.33976Z_1 + 204.1475408Z_2 + 207.1288857Z_3 - 1409.000071Z_1Z_2 - 1115.89088Z_1Z_3 - 1024.82821Z_2Z_3 + 14075.96126Z_1Z_2Z_3$ (3.18)

3.3.1.4 RZ Vector for Density Model

Using the values of Z_i from the Z matrix of Table 3.6 and experimental SSD density given in Table 4.3, the RZ vector was obtained for the cube density model as:

$\sum(F(z).Z_1) = 4354.835118$ (3.19)

$\sum(F(z).Z_2) = 22749.2322$ (3.20)

$\sum(F(z).Z_3) = 27172.3527$ (3.21)

$\sum(F(z).Z_1Z_2) = 1823.88228$ (3.22)

$\sum(F(z).Z_1 Z_3) = 2178.20931$ (3.23)

$\sum(F(z).Z_2 Z_3) = 10592.4366$ (3.24)

$\sum(F(z).Z_1 Z_2 Z_3) = 848.971932$ (3.25)

3.3.1.5 Coefficients for the Cube Density Model

Using Equation 2.34b and by making α (the coefficient) the subject of the equation, coefficients of the regression model were obtained. That is, multiplying the CC inverse matrix give in Table 3.8 and the RZ vectors given in Equations 3.19 – 3.25, the coefficients for the cube density model of sandstone concrete were obtained as:

$\alpha1 = -466974.46$ (3.26)

$\alpha2 = -1837.6871$ (3.27)

$\alpha3 = -1373.4003$ (3.28)

$\alpha12 = 562322.02$ (3.29)

$\alpha13 = 561428.90$ (3.30)

$\alpha23 = -141.30139$ (3.31)

$\alpha123 = -315.03831$ (3.32)

Substituting the values of the derived coefficients for the model cube density in Equations 3.26 - 32 into Equation 2.28, the model regression equation was obtained:

$$F(z) = - 466974.46Z_1 - 1837.6871Z_2 - 1373.40037Z_3 + 562322.02Z_1Z_2 + 561428.90Z_1Z_3 - 141.30139Z_2Z_3 - 315.03831Z_1Z_2Z_3 \quad (3.33)$$

3.3.2 Test for Goodness of Fit of the Models

It is expected that the results of the models will be accepted with about 95% risk of being correct or 5% risk of being incorrect. The results being correct mean that there is no significant difference between model results and experimental test results. At this point it will be wise to state the statistical hypothesis for accepting or otherwise the adequacy of the models' results as follows:

i. Null Hypothesis (H_0): There is no significant difference between the Models' results and experimental test results.

ii. Alternative Hypothesis (H_1): There is a significant difference between the Models' results and the experimental test results

iii. The risk involved is that 5% or below of the Models' results will be incorrect.

For the purpose of this work statistical Fisher test was employed.

3.3.2.1 Error of the Replicates

During the experimental test several errors were introduced. Some occurred due to human inconsistency, variation in test tools and equipment. Others occurred due to conditional variation like humidity, temperature, pressure and the like. At any arbitrary point of observation, replicate values are obtained and used to get the mean value of response at that point. The mean value is used because the replicate values vary from each other due to the inconsistencies mentioned above. The variance of the replicates at the arbitrary point of observation is designated as S_i^2. Cramer (1946) gave the equation of variance as:

$$Sy^2 = 1/ (n-1) * [\Sigma (yI – ý)^2] \tag{3.34}$$

1 £ i £ n

Where n is the number of observation, y_i is the value at any arbitrary point of observation and ý is the mean value of the y_i values.

$$\text{That is } ý = \Sigma y_i / n \tag{3.35}$$

1 £ i £ n

When equation 3.11 is expanded it becomes approximately equal to:

$$S_y^2 = 1/(n-1) * [\Sigma y_i^2 – 1/n * (\Sigma y_i)^2] \tag{3.36}$$

1 £ i £ n

Thus, the equation of variance of the replicates at any arbitrary point of observation will be given as:

$$S_i^2 = 1/(n_i-1) * [\Sigma y_i^2 – 1/n_i * (\Sigma y_i)^2] \tag{3.37}$$

1 £ i £ n_i

Hence, the variance of the replicates at all the points of observation will be the summation of the individual variances divided by number of points of observation. That is to say:

$$S_y^2 = 1/V * \Sigma S_i^2 \tag{3.38}$$

1 £ i £ N

Where V is the degree of freedom, which is equal to N – 1. Therefore, the random error or standard deviation becomes:

$$S_y = Ö (S_y^2) \tag{3.39}$$

3.3.2.2 Fisher Test

Formulation of a model for predicting mix ratios where desired response is known and vice versa involves generation of a mathematical equation with unknown coefficients and then carrying out experimental test to generate the unknown coefficients. This means that response can be got by the experimental test that uses replicates and as well from the model when the coefficients are known. It is very vital that the variance S_T^2 from the model should be compared with the variance S_E^2 from the experimental test to see if they are similar or different (Gelman and Stern, 2006).

A means of establishing the fact to prove whether or not S_E^2 and ST^2 are similar through the use Fisher test was devised. S_1^2 and S_2^2 were used in this test. S_1^2 is the greater of S_E^2 and S_T^2. The equation for the Fisher-test is given as:

$$F = S_1^2 / S_2^2 \qquad (3.15)$$

The range of S_1^2 / S_2^2 to accept the fact that S_E^2 and S_T^2 are similar is shown as:

$$1 / f\alpha\ (v1,v2) < S_1^2 / S_2^2 < f\alpha\ (v1,v2) \qquad (3.16)$$

α is significant level, v is the degree of freedom = N – 1.

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 Results

The results of the experiments and the formulated models are presented in section 4.1.1 and 4.1.2.

4.1.1 Experimental Results

The slump values for the 44 mix ratios are shown in Table 4.1. Average 28-day compressive strength test results for all the 44 concrete mixtures are presented in Table 4.2. These values were obtained from the failure loads of the different sample cubes tested. The values for three replicates were obtained and the average compressive strength calculated for all the mixes. Average density of concrete cubes for all the 44 concrete mixtures (three replicates each) are presented in Table 4.3.

The data given in Table 4.2 were utilized for statistical analysis to examine the significance of the mixture factors and subsequently to obtain a regression model for compressive strength and saturated surface dry density in terms of the factors considered.

Table 4.1 Slump Values of Sandstone Concrete

S/N	Mix Notation	Mix Ratio	Slump Value (mm)
1	M1	0.45:1:1.5:4.5	0
2	M2	0.45:1:2:4	1
3	M3	0.45:1:2.5:3.5	2
4	M3	0.45:1:3:3	0
5	M5	0.45:1:3.5:2.5	0
6	M6	0.45:1:4:2	0
7	M7	0.5:1:1.75:4.25	12
8	M8	0.5:1:2.25:3.75	14
9	M9	0.5:1:2.75:3.25	4
10	M10	0.5:1:3.25:2.75	3
11	M11	0.5:1:3.75:2.25	2
12	M12	0.55:1:1.5:4.5	15
13	M13	0.55:1:2:4	12
14	M14	0.55:1:2.5:3.5	13
15	M15	0.55:1:3:3	5
16	M16	0.55:1:3.5:2.5	4
17	M17	0.55:1:4:2	2
18	M18	0.6:1:1.75:4.25	135
19	M19	0.6:1:2.25:3.75	100
20	M20	0.6:1:2.75:3.25	65
21	M21	0.6:1:3.25:2.75	17
22	M22	0.6:1:3.75:2.25	11
23	V1	0.45:1:1.75:4.25	1
24	V2	0.45:1:2.25:3.75	3
25	V3	0.45:1:2.75:3.25	0
26	V4	0.45:1:3.25:2.75	0
27	V5	0.45:1:3.75:2.25	0
28	V6	0.5:1:1.5:4.5	10
29	V7	0.5:1:2:4	6
30	V8	0.5:1:2.5:3.5	5
31	V9	0.5:1:3:3	1
32	V10	0.5:1:3.5:2.5	2
33	V11	0.5:1:4:2	1
34	V12	0.55:1:1.75:4.25	22
35	V13	0.55:1:2.25:3.75	22
36	V14	0.55:1:2.75:3.25	17
37	V15	0.55:1:3.25:2.75	14
38	V16	0.55:1:3.75:2.25	3
39	V17	0.6:1:1.5:4.5	108
40	V18	0.6:1:2:4	122
41	V19	0.6:1:2.5:3.5	51
42	V20	0.6:1:3:3	45
43	V21	0.6:1:3.5:2.5	9
44	V22	0.6:1:4:2	8

Table 4.2 Compressive Strength Test Results of Sandstone Concrete Cubes

S/N	MIX RATIO	SAMPLE	FAILURE LOAD ON CUBES (KN)	COMPRESSIVE STRENGTH (N/mm^2)	AV. COMPRESSIVE STRENGTH (N/mm^2)
1	0.45:1:1.5:4.5	A	452.92	20.13	21.42
		B	508.27	22.59	
		C	484.43	21.53	
2	0.45:1:2:4	A	373.95	16.62	17.61
		B	381.60	16.96	
		C	433.12	19.25	
3	0.45:1:2.5:3.5	A	317.91	14.13	14.49
		B	300.38	13.35	
		C	359.77	15.99	
4	0.45:1:3:3	A	291.6	12.96	13.78
		B	297.92	13.24	
		C	340.65	15.14	
5	0.45:1:3.5:2.5	A	251.15	11.16	11.94
		B	299.40	13.31	
		C	255.25	11.34	
6	0.45:1:4:2	A	223.08	9.47	9.77
		B	243.45	10.82	
		C	202.95	9.02	
7	0.5:1:1.75:4.25	A	423.00	18.80	19.03
		B	435.15	19.34	
		C	426.14	18.94	
8	0.5:1:2.25:3.75	A	372.36	16.55	17.62
		B	404.33	17.97	
		C	412.65	18.34	
9	0.5:1:2.75:3.25	A	370.80	16.48	17.35
		B	435.16	19.34	
		C	365.20	16.23	
10	0.5:1:3.25:2.75	A	378.90	16.84	15.10
		B	368.11	16.36	
		C	272.03	12.09	
11	0.5:1:3.75:2.25	A	286.22	12.72	12.54
		B	277.88	12.35	
		C	282.61	12.56	

Table 4.2 Compressive Strength Test Results of Sandstone Concrete Cubes continued.

S/N	MIX RATIO	SAMPLE	FAILURE LOAD ON CUBES (KN)	COMPRESSIVE STRENGTH (N/mm^2)	AVERAGE COMPRESSIVE STRENGTH (N/mm^2)
12	0.55:1:1.5:4.5	A	409.52	18.20	17.52
		B	377.10	16.76	
		C	395.76	17.59	
13	0.55:1:2:4	A	405.67	18.03	18.58
		B	402.32	17.88	
		C	446.18	19.83	
14	0.55:1:2.5:3.5	A	527.62	23,45	22.27
		B	471.15	20.91	
		C	504.45	22.42	
15	0.55:1:3:3	A	478.33	21.26	19.79
		B	419.40	18.64	
		C	438.01	19.47	
16	0.55:1:3.5:2.5	A	365.70	16.25	16.86
		B	382.46	17.00	
		C	390.03	17.33	
17	0.55:1:4:2	A	233.50	10.38	12.24
		B	312.32	13.88	
		C	280.04	12.45	
18	0.6:1:1.75:4.25	A	391.24	17.39	18.04
		B	370.62	16.47	
		C	455.55	20.25	
19	0.6:1:2.25:3.75	A	521.33	23.17	23.23
		B	499.95	22.22	
		C	546.75	24.30	
20	0.6:1:2.75:3.25	A	504.90	22.44	22.67
		B	472.96	21.02	
		C	552.36	24.55	
21	0.6:1:3.25:2.75	A	385.65	17.14	18.18
		B	382.95	17.02	
		C	458.55	20.38	
22	0.6:1:3.75:2.25	A	331.16	14.71	15.24
		B	387.05	17.20	
		C	310.60	13.80	

Table 4.2 Compressive Strength Test Results of Sandstone Concrete Cubes continued.

S/N	MIX RATIO	SAMPLE	FAILURE LOAD ON CUBES (KN)	COMPRESSIVE STRENGTH (N/mm^2)	AV. COMPRESSIVE STRENGTH (N/mm^2)
23	0.45:1:1.75:4.25	A	409.73	18.21	19.44
		B	457.42	20.33	
		C	445.05	19.78	
24	0.45:1:2.25:3.75	A	345.35	15.35	15.48
		B	363.60	16.16	
		C	335.71	14.92	
25	0.45:1:2.75:3.25	A	285.30	12.68	14.81
		B	377.33	16.77	
		C	337.37	14.99	
26	0.45:1:3.25:2.75	A	293.53	13.05	12.60
		B	272.60	12.12	
		C	284.19	12.63	
27	0.45:1:3.75:2.25	A	216.68	9.63	10.28
		B	262.82	11.68	
		C	214.65	9.54	
28	0.5:1:1.5:4.5	A	469.81	20.88	19.44
		B	430.66	19.14	
		C	411.75	18.30	
29	0.5:1:2:4	A	391.26	17.39	18.29
		B	415.12	18.45	
		C	428.4	19.04	
30	0.5:1:2.5:3.5	A	413.11	18.36	17.68
		B	360.45	16.02	
		C	420.07	18.67	
31	0.5:1:3:3	A	355.28	15.79	16.61
		B	406.13	18.05	
		C	360.01	16.00	
32	0.5:1:3.5:2.5	A	354.37	15.75	14.46
		B	321.29	14.28	
		C	300.63	13.36	
33	0.5:1:4:2	A	255.61	11.36	10.75
		B	204.75	8910	
		C	265.73	11.81	

Table 4.2 Compressive Strength Test Results of Sandstone Concrete Cubes continued.

S/N	MIX RATIO	SAMPLE	FAILURE LOAD ON CUBES (KN)	COMPRESSIVE STRENGTH (N/mm^2)	AV. COMPRESSIVE STRENGTH (N/mm^2)
34	0.55:1:1.75:4.25	A	362.70	16.12	18.25
		B	461.46	20.51	
		C	407.51	18.11	
35	0.55:1:2.25:3.75	A	437.16	19.94	20.32
		B	477.00	21.20	
		C	457.42	20.33	
36	0.55:1:2.75:3.25	A	528.75	23.50	20.97
		B	423.68	18.83	
		C	462.83	20.57	
37	0.55:1:3.25:2.75	A	397.51	17.67	18.30
		B	415.62	18.47	
		C	422.13	18.76	
38	0.55:1:3.75:2.25	A	390.21	17.34	14.57
		B	282.09	12.58	
		C	310.18	13.79	
39	0.6:1:1.5:4.5	A	354.15	15.74	16.51
		B	389.25	17.30	
		C	371.23	16.50	
40	0.6:1:2:4	A	451.79	20.08	21.04
		B	480.61	23.36	
		C	488.03	21.69	
41	0.6:1:2.5:3.5	A	570.25	25.34	24.18
		B	522.04	23.20	
		C	539.68	24.00	
42	0.6:1:3:3	A	495.57	22.02	20.04
		B	452.93	20.13	
		C	404.01	17.96	
43	0.6:1:3.5:2.5	A	364.71	16.21	16.70
		B	368.11	16.36	
		C	394.65	17.54	
44	0.6:1:4:2	A	310.27	13.79	13.31
		B	290.48	12.91	
		C	297.67	13.23	

Table 4.3 SSD Density of Sandstone Concrete Cubes

S/N	MIX RATIO	SAMPLE	WEIGHT OF CUBES (Kg)	DENSITY (Kg/m^3)	AVERAGE DENSITY (Kg/m^3)
1	0.45:1:1.5:4.5	A	7.80	2311.11	2459.26
		B	8.50	2518.52	
		C	8.60	2548.15	
2	0.45:1:2:4	A	8.20	2447.26	2427.14
		B	8.34	2473.26	
		C	8.24	2444.89	
3	0.45:1:2.5:3.5	A	7.55	2296.29	2380.25
		B	8.15	2414.81	
		C	8.20	2429.63	
4	0.45:1:3:3	A	8.00	2370.37	2380.25
		B	7.70	2281.48	
		C	8.00	2370.37	
5	0.45:1:3.5:2.5	A	7.65	2266.67	2311.11
		B	8.05	2385.19	
		C	7.70	2281.48	
6	0.45:1:4:2	A	7.90	2340.74	2311.11
		B	7.80	2311.11	
		C	7.70	2281.48	
7	0.5:1:1.75:4.25	A	8.55	2533.33	2562.96
		B	8.75	2592.59	
		C	8.65	2562.96	
8	0.5:1:2.25:3.75	A	8.30	2459.26	2479.01
		B	8.40	2488.89	
		C	8.40	2488.89	
9	0.5:1:2.75:3.25	A	8.20	2429.63	2454.32
		B	8.30	2459.26	
		C	8.35	2474.07	
10	0.5:1:3.25:2.75	A	8.50	2518.52	2439.51
		B	7.85	2325.93	
		C	8.35	2474.07	
11	0.5:1:3.75:2.25	A	8.00	2370.37	2400.00
		B	8.30	2459.26	
		C	8.00	2370.37	

Table 4.3 SSD Density of Sandstone Concrete Cubes continued.

S/N	MIX RATIO	SAMPLE	WEIGHT OF CUBES (Kg)	DENSITY (Kg/m^3)	AVERAGE DENSITY (Kg/m^3)
12	0.55:1:1.5:4.5	A	8.80	2607.41	2570.25
		B	8.50	2518.52	
		C	8.55	2533.33	
13	0.55:1:2:4	A	8.64	2560.00	2576.79
		B	8.60	2548.15	
		C	8.85	2622.22	
14	0.55:1:2.5:3.5	A	8.70	2577.78	2561.98
		B	8.64	2560.00	
		C	8.60	2548.15	
15	0.55:1:3:3	A	8.50	2518.52	2526.42
		B	8.45	2503.70	
		C	8.63	2557.04	
16	0.55:1:3.5:2.5	A	8.30	2459.26	2483.97
		B	8.40	2488.89	
		C	8.45	2503.7	
17	0.55:1:4:2	A	8.40	2488.89	2437.53
		B	8.43	2497.78	
		C	7.85	2325.92	
18	0.6:1:1.75:4.25	A	8.60	2548.15	2592.59
		B	8.85	2622.22	
		C	8.80	2607.41	
19	0.6:1:2.25:3.75	A	8.45	2503.70	2513.58
		B	8.50	2518.52	
		C	8.50	2518.52	
20	0.6:1:2.75:3.25	A	8.45	2503.70	2481.97
		B	8.38	2482.96	
		C	8.3	2459.26	
21	0.6:1:3.25:2.75	A	8.30	2459.26	2469.14
		B	8.20	2429.63	
		C	8.50	2518.52	
22	0.6:1:3.75:2.25	A	8.30	2459.26	2457.28
		B	8.58	2482.96	
		C	8.30	2429.63	

Table 4.3 SSD Density of Sandstone Concrete Cubes continued.

S/N	MIX RATIO	SAMPLE	WEIGHT OF CUBES (Kg)	DENSITY (Kg/m^3)	AVERAGE DENSITY (Kg/m^3)
23	0.45:1:1.75:4.25	A	7.80	2311.11	2429.63
		B	8.50	2518.52	
		C	8.30	2459.26	
24	0.45:1:2.25:3.75	A	7.65	2266.67	2395.06
		B	8.00	2370.37	
		C	8.60	2548.15	
25	0.45:1:2.75:3.25	A	8.00	2370.37	2380.25
		B	8.10	2400.00	
		C	8.00	2370.37	
26	0.45:1:3.25:2.75	A	8.00	2370.37	2340.74
		B	7.70	2281.48	
		C	8.00	2370.37	
27	0.45:1:3.75:2.25	A	7.80	2311.11	2306.17
		B	7.85	2325.93	
		C	7.70	2281.48	
28	0.5:1:1.5:4.5	A	8.85	2622.22	2572.84
		B	8.70	2577.78	
		C	8.50	2518.52	
29	0.5:1:2:4	A	8.15	2414.82	2526.42
		B	8.70	2577.78	
		C	8.73	2586.67	
30	0.5:1:2.5:3.5	A	8.25	2444.44	2469.13
		B	8.50	2518.52	
		C	8.25	2444.44	
31	0.5:1:3:3	A	8.30	2459.26	2451.36
		B	8.32	2465.19	
		C	8.20	2429.63	
32	0.5:1:3.5:2.5	A	8.30	2459.26	2437.53
		B	8.20	2429.63	
		C	8.18	2423.70	
33	0.5:1:4:2	A	7.80	2311.11	2321.00
		B	7.70	2281.48	
		C	8.00	2370.37	

Table 4.3 SSD Density of Sandstone Concrete Cubes continued.

S/N	MIX RATIO	SAMPLE	WEIGHT OF CUBES (Kg)	DENSITY (Kg/m^3)	AVERAGE DENSITY (Kg/m^3)
34	0.55:1:1.75:4.25	A	8.65	2562.96	2583.70
		B	8.76	2595.56	
		C	8.75	2592.59	
35	0.55:1:2.25:3.75	A	8.57	2539.26	2564.94
		B	8.70	2577.78	
		C	8.70	2577.78	
36	0.55:1:2.75:3.25	A	8.57	2539.26	2533.33
		B	8.58	2542.22	
		C	8.50	2518.52	
37	0.55:1:3.25:2.75	A	8.37	2480.00	2505.68
		B	8.40	2488.89	
		C	8.60	2548.15	
38	0.55:1:3.75:2.25	A	8.50	2518.52	2475.06
		B	8.20	2429.63	
		C	8.36	2477.04	
39	0.6:1:1.5:4.5	A	8.40	2488.89	2550.12
		B	8.80	2607.41	
		C	8.62	2554.07	
40	0.6:1:2:4	A	8.40	2488.89	2543.21
		B	8.75	2592.59	
		C	8.60	2548.15	
41	0.6:1:2.5:3.5	A	8.60	2548.15	2508.64
		B	8.50	2518.52	
		C	8.30	2459.26	
42	0.6:1:3:3	A	8.20	2429.63	2479.01
		B	8.42	2494.81	
		C	8.48	2512.59	
43	0.6:1:3.5:2.5	A	8.38	2423.70	2467.16
		B	8.40	2488.89	
		C	8.40	2488.89	
44	0.6:1:4:2	A	8.50	2488.89	2446.42
		B	8.10	2400.00	
		C	8.27	2450.37	

4.1.2 Statistical Model Application Results

The first 22 mixes (model mixes) of the sandstone concrete shown in Table 4.2 were used to formulate regression models and the last 22 (validation mixes) were used to validate the models. The compressive strength and density of the concrete cubes predicted by the models are presented in section 4.1.2.1 and 4.1.2.3.

4.1.2.1 Model Compressive Strength Data

The regression analysis equation (Equation 3.4), derived in chapter three was used to predict the model compressive cube strength as shown in Table 4.4

Table 4.4 Compressive Strength Results from Experiment and Model

Mix Notation	Mix Ratio	Compressive Strength (N/mm^2)	
		Experimental	Model
M1	0.45:1:1.5:4.5	21.42	20.77
M2	0.45:1:2:4	17.61	17.67
M3	0.45:1:2.5:3.5	14.49	15.08
M4	0.45:1:3:3	13.78	13.01
M5	0.45:1:3.5:2.5	11.94	11.45
M6	0.45:1:4:2	09.77	10.41
M7	0.5:1:1.75:4.25	19.03	19.26
M8	0.5:1:2.25:3.75	17.62	18.67
M9	0.5:1:2.75:3.25	17.35	17.39
M10	0.5:1:3.25:2.75	15.10	15.43
M11	0.5:1:3.75:2.25	12.54	12.78
M12	0.55:1:1.5:4.5	17.52	17.48
M13	0.55:1:2:4	18.58	19.95
M14	0.55:1:2.5:3.5	22.27	20.60
M15	0.55:1:3:3	19.79	19.40
M16	0.55:1:3.5:2.5	16.86	16.38
M17	0.55:1:4:2	12.24	11.53
M18	0.6:1:1.75:4.25	18.04	18.24
M19	0.6:1:2.25:3.75	23.23	21.85
M20	0.6:1:2.75:3.25	22.67	22.53
M21	0.6:1:3.25:2.75	18.18	20.28
M22	0.6:1:3.75:2.25	15.24	15.11
V1	0.45:1:1.75:4.25	19.44	19.16
V2	0.45:1:2.25:3.75	15.48	16.31
V3	0.45:1:2.75:3.25	14.81	13.98
V4	0.45:1:3.25:2.75	12.60	12.17
V5	0.45:1:3.75:2.25	10.28	10.86
V6	0.5:1:1.5:4.5	19.44	19.30
V7	0.5:1:2:4	18.29	19.05
V8	0.5:1:2.5:3.5	17.68	18.12
V9	0.5:1:3:3	16.61	16.49
V10	0.5:1:3.5:2.5	14.46	14.19
V11	0.5:1:4:2	10.75	11.20
V12	0.55:1:1.75:4.25	18.25	18.95
V13	0.55:1:2.25:3.75	20.32	20.50
V14	0.55:1:2.75:3.25	20.97	20.23
V15	0.55:1:3.25:2.75	18.30	18.12
V16	0.55:1:3.75:2.25	14.57	14.18
V17	0.6:1:1.5:4.5	16.51	15.34
V18	0.6:1:2:4	21.04	20.41
V19	0.6:1:2.5:3.5	24.18	22.55
V20	0.6:1:3:3	20.04	21.77
V21	0.6:1:3.5:2.5	16.70	18.06
V22	0.6:1:4:2	13.31	11.43

4.1.2.2 Fisher Test for Adequacy of Compressive Strength Model

Fisher F-test was carried out to determine whether there is significant difference between values of compressive strengths from laboratory and those from the model.

Legend:

Y_E = response from experiment

Y_M = response from model

$\ddot{Y}_E$ = mean response from experiment = $\frac{\sum Y_E}{N}$

$\ddot{Y}_M$ = mean response from model = $\frac{\sum Y_M}{N}$

N = total points of observation (for controls) = 22

S_E^2 = variance from the experiment = $\frac{\sum(Y_E - \ddot{Y}_E)2}{(N-1)}$

S_M^2 = variance from the model = $\frac{\sum(Y_M - \ddot{Y}_M)2}{(N-1)}$

(N-1) = degree of freedom = 22 – 1 = 21

$$\ddot{Y}_E = \frac{\sum Y_E}{N} = 17.001$$

$$\ddot{Y}_M = \frac{\sum Y_M}{N} = 16.9262$$

$$S_E^2 = \frac{\sum(Y_E - \ddot{Y}_E)2}{(N-1)} = 12.3013$$

$$S_M^2 = \frac{\sum(Y_M - \ddot{Y}_M)2}{(N-1)} = 12.5473$$

The full result is shown in Table 4.5

Table 4.5 Fisher F-Test on the Compressive Strength from the Model

Response Symbol	$\mathbf{Y_E}$	$\mathbf{Y_M}$	$\mathbf{Y_E - \ddot{Y}_E}$	$\mathbf{Y_M - \ddot{Y}_M}$	$\mathbf{(Y_E - \ddot{Y}_E)^2}$	$\mathbf{(Y_M - \ddot{Y}_M)^2}$
V1	19.44	19.16	2.439	2.231	5.94695	4.97785
V2	15.48	16.31	-1.521	-0.614	2.31455	0.37656
V3	14.81	13.98	-2.191	-2.944	4.80207	8.66826
V4	12.60	12.17	-4.401	-4.761	19.3720	22.6627
V5	10.28	10.86	-6.721	-6.063	45.1767	36.7560
V6	19.44	19.30	2.439	2.374	5.94695	5.63565
V7	18.29	19.05	1.289	2.124	1.66058	4.51303
V8	17.68	18.12	0.679	1.189	0.46055	1.41412
V9	16.61	16.49	-0.391	-0.432	0.15317	0.18638
V10	14.46	14.19	-2.541	-2.738	6.45853	7.49801
V11	10.75	11.20	-6.251	-5.730	39.0795	32.8380
V12	18.25	18.95	1.249	2.021	1.55909	4.08521
V13	20.32	20.50	3.319	3.577	11.0133	12.7959
V14	20.97	20.23	3.969	3.302	15.7501	10.9026
V15	18.30	18.12	1.299	1.196	1.68646	1.42930
V16	14.57	14.18	-2.431	-2.742	5.91153	7.51859
V17	16.51	15.34	-0.491	-1.584	0.24144	2.51031
V18	21.04	20.41	4.039	3.483	16.3106	12.1311
V19	24.18	22.55	7.179	5.626	51.5328	31.6472
V20	20.04	21.77	3.039	4.843	9.23331	23.4590
V21	16.70	18.06	-0.301	1.137	0.09082	1.29175
V22	13.31	11.43	-3.691	-5.495	13.6262	30.1961
Total	374.030	372.376			258.327	263.494
Mean	17.001	16.926				

Therefore, S_1^2 = 12.5473 and S_2^2 = 12.30313

$F_{calculted} = S_1^2/S_2^2 = 1.019846$

$F_{table} = F_{0.05}(21,21)$

From appendix A, $F_{0.05}(21,21)$ = 2.9 (by interpolation)

$\frac{1}{F_{table}} = 0.344828$

Therefore, 0.344828 < 1.019846 < 2.9

Thus, the condition $1/F < S_1^2/S_2^2 < F$ has been satisfied.

4.1.2.3 Model Density Data

Using Equation 3.5 derived in chapter three for the cube density model, the predicted values were obtained as given in Table 4.6, together with their corresponding experimental values.

Table 4.6 SSD Density Results from Experiment and Model

Mix Notation	Mix Ratio	Cube Density (Kg/m^3)	
		Experimental	Model
M1	0.45:1:1.5:4.5	2459.26	2459.54
M2	0.45:1:2:4	2427.14	2423.47
M3	0.45:1:2.5:3.5	2380.25	2389.36
M4	0.45:1:3:3	2380.25	2357.22
M5	0.45:1:3.5:2.5	2311.11	2327.04
M6	0.45:1:4:2	2311.11	2298.82
M7	0.5:1:1.75:4.25	2562.96	2540.26
M8	0.5:1:2.25:3.75	2479.01	2505.92
M9	0.5:1:2.75:3.25	2454.32	2473.53
M10	0.5:1:3.25:2.75	2439.51	2443.1
M11	0.5:1:3.75:2.25	2400	2414.63
M12	0.55:1:1.5:4.5	2570.25	2599.11
M13	0.55:1:2:4	2576.79	2564.5
M14	0.55:1:2.5:3.5	2561.98	2531.85
M15	0.55:1:3:3	2526.42	2501.16
M16	0.55:1:3.5:2.5	2483.97	2472.42
M17	0.55:1:4:2	2437.53	2445.63
M18	0.6:1:1.75:4.25	2592.59	2567.72
M19	0.6:1:2.25:3.75	2513.58	2534.8
M20	0.6:1:2.75:3.25	2481.97	2503.82
M21	0.6:1:3.25:2.75	2469.14	2474.8
M22	0.6:1:3.75:2.25	2457.28	2447.73
V1	0.45:1:1.75:4.25	2429.63	2441.26
V2	0.45:1:2.25:3.75	2395.06	2406.17
V3	0.45:1:2.75:3.25	2380.25	2373.05
V4	0.45:1:3.25:2.75	2340.74	2341.89
V5	0.45:1:3.75:2.25	2306.17	2312.69
V6	0.5:1:1.5:4.5	2572.84	2558.17
V7	0.5:1:2:4	2526.42	2522.85
V8	0.5:1:2.5:3.5	2469.14	2489.48
V9	0.5:1:3:3	2451.36	2458.07
V10	0.5:1:3.5:2.5	2437.53	2428.62
V11	0.5:1:4:2	2321	2401.13
V12	0.55:1:1.75:4.25	2583.7	2581.56
V13	0.55:1:2.25:3.75	2564.94	2547.93
V14	0.55:1:2.75:3.25	2533.33	2516.26
V15	0.55:1:3.25:2.75	2505.68	2486.54
V16	0.55:1:3.75:2.25	2475.06	2458.78
V17	0.6:1:1.5:4.5	2550.12	2584.91
V18	0.6:1:2:4	2543.21	2551.01
V19	0.6:1:2.5:3.5	2508.64	2519.07
V20	0.6:1:3:3	2479.01	2489.07
V21	0.6:1:3.5:2.5	2467.16	2461.02
V22	0.6:1:4:2	2446.42	2434.92

4.1.2.4 Fisher Test for Adequacy of Density Model

Fisher F-test was carried out to determine whether there is significant difference between values of cube density from laboratory and those from the model.

Legend:

D_E = response from experiment

D_M = response from model

$Đ_E$ = mean response from experiment = $\frac{\sum D_E}{N}$

$Đ_M$ = mean response from model = $\frac{\sum D_M}{N}$

N = total points of observation (for controls) = 22

S_E^2 = variance from the experiment = $\frac{\sum(D_E - Đ_E)2}{(N-1)}$

S_M^2 = variance from the model = $\frac{\sum(D_M - Đ_M)2}{(N-1)}$

(N-1) = degree of freedom = 22 – 1 = 21

$Đ_E = \frac{\sum D_E}{N} = 2467.61$

$Đ_M = \frac{\sum D_M}{N} = 2471.11$

$S_E^2 = \frac{\sum(D_E - Đ_E)2}{(N-1)} = 6535.91$

$S_M^2 = \frac{\sum(D_M - Đ_M)2}{(N-1)} = 5613.64$

The full result is shown in Table 4.7

Table 4.7 Fisher F-Test on the SSD Density from the Model

Response Symbol	D_E	D_M	$D_E - Đ_E$	$D_M - Đ_M$	$(D_E - Đ_E)^2$	$(D_M - Đ_M)^2$
V1	2429.63	2441.256	-37.9795	-29.8542	1442.446	891.2736
V2	2395.06	2406.171	-72.5495	-64.94	5263.437	4217.199
V3	2380.25	2373.047	-87.3595	-98.0633	7631.69	9616.417
V4	2340.74	2341.886	-126.87	-129.224	16095.88	16698.92
V5	2306.17	2312.688	-161.44	-158.423	26062.73	25097.81
V6	2572.84	2558.172	105.2305	87.0618	11073.45	7579.757
V7	2526.42	2522.845	58.81045	51.73464	3458.67	2676.473
V8	2469.14	2489.477	1.530455	18.36647	2.342291	337.3272
V9	2451.36	2458.068	-16.2495	-13.0427	264.0477	170.1121
V10	2437.53	2428.618	-30.0795	-42.4929	904.7791	1805.645
V11	2321	2401.127	-146.61	-69.9841	21494.36	4897.769
V12	2583.7	2581.559	116.0905	110.448	13476.99	12198.76
V13	2564.94	2547.932	97.33045	76.82087	9473.217	5901.446
V14	2533.33	2516.259	65.72045	45.14881	4319.178	2038.415
V15	2505.68	2486.542	38.07045	15.43182	1449.36	238.1411
V16	2475.06	2458.781	7.450455	-12.3301	55.50927	152.0313
V17	2550.12	2584.913	82.51045	113.8022	6807.975	12950.95
V18	2543.21	2551.014	75.60045	79.90326	5715.429	6384.532
V19	2508.64	2519.066	41.03045	47.95495	1683.498	2299.678
V20	2479.01	2489.068	11.40045	17.9573	129.9704	322.4647
V21	2467.16	2461.021	-0.44955	-10.0897	0.202091	101.8019
V22	2446.42	2434.925	-21.1895	-36.186	448.9968	1309.429
Total	54287.4	54364.4			137254	117886
Mean	2467.61	2471.11				

Therefore, $S_1^2 = 6535.91$ and $S_2^2 = 5613.64$

$F_{calculted} = S_1^2/S_2^2 = 1.16429$

$F_{table} = F_{0.05}(21,21)$

From appendix A, $F_{0.05}(21,21) = 2.9$ (by interpolation)

$\frac{1}{F_{table}} = 0.344828$

Therefore, $0.344828 < 1.16429 < 2.9$

Thus, the condition $1/F < S_1^2/S_2^2 < F$ has been satisfied.

4.2 Discussions

The results are discussed herein based on the data obtained from experiment and the developed models.

4.2.1 Experimental Results

Results obtained for slump, compressive strength and saturated surface dry density of sandstone concrete are discussed in section 4.2.1.1 to 4.2.1.3.

4.2.1.1 Slump Values

From Table 4.12 showing the slump values of different sets of mix ratios, the maximum slump value of 135mm was obtained for sandstone concrete at a mix ratio of 0.6:1:1.75:4.25 (M18). The set of mix ratios with w/c of 0.45 had 0.0mm slump values except mix ratios 0.45:1:2:4 (M2), 0.45:1:2.25:3.75 (V2) and 0.45:1:2.5:3.5 (M3) which had 1mm, 3mm and 2mm respectively. It can be deduced from the slump values that sandstone concrete requires a bit more water to be workable due to its water absorption capacity. Mixes with the least water-cement ratio were less workable compared to those with the more water-cement ratio were more workable. Also, mixes with more fine/coarse aggregate ratio had less slump that those with less fine/coarse aggregate ratio.

4.2.1.2 Compressive Strength from Experimental

The results presented in Table 4.2 indicate that the maximum compressive strength obtained from experiment is 24.18N/mm^2 and corresponds to 0.6:1:2.5:3.5 mix ratio with cube density of 2508.64Kg/m^3 while the least compressive strength achieved is 9.77N/mm^3 for 0.5:1:4:2 with cube density of 2321.00Kg/m^3. It can be observed that the highest water-cement ratio produced the highest strength while the lowest produced the least strength. Also, the least water-cement ratio together with lower fine/coarse aggregates ratio produced higher strength. This means that mixes with less water-cement ratio had poor workability and less water for hydration leading to low strength while mixes with higher water-cement ratio were more workable and had more water for hydration leading to higher strength.

Generally, mixes with the highest fine/coarse aggregate ratio had low compressive strength. The compressive strength of the sandstone concrete mixes generally increased as the fine/coarse aggregate ratio reduces.

4.2.1.3 Saturated Surface Dry Density from Experiment

The results presented in Tables 4.3 indicate that the maximum cube density of sandstone concrete cube is 2592.59Kg/m^3 for mix ratio 0.6:1:1.75:4.25 with compressive strength of 18.04N/mm^2 while the minimum cube density obtained is 2306.17Kg/m^3 for mix ratio 0.45:1:3.75:2.25 with compressive strength of 10.28N/mm^2. It can be observed that the cube density increases as the water-cement ratio increases. However, at the same water-cement ratio, mixes with lower fine/coarse aggregates ratio gave higher cube density values.

The data obtained from the experiment as presented in Tables 4.2 and 4.3, were plotted to depict the variations of compressive strength with water/cement ratio and fine/coarse aggregate ratio at different cement contents (see Figures E.1 – E.25 in appendix E). It can be seen that the compressive strength increases with the increase in the water/cement ratio and the decrease in the fine/coarse aggregate ratio. Therefore, the graphs can be utilized to select an adequate value of the water/cement ratio and aggregate content for a given value of the target compressive strength satisfying the workability and durability requirements. For example, in the case of a normal exposure, a higher value of the water/cement ratio and a lower value of fine/coarse aggregate content can be selected which would give more workability at a lower cost, whereas, for harsh exposure conditions, a lower value of the water/cement ratio and a moderate value of fine/coarse aggregate content can be selected, which would provide better durability.

4.2.2 Model Application

The results show that the statistical regression models were successfully used to predict the unknown strength parameters. Thus, the application of a computational model in sandstone concrete was successfully shown.

4.2.2.1 Compressive Strength from Model

The results of regression model for compressive strength are presented in Table 4.4. The maximum compressive strength predicted by the model is 22.55N/mm^2 and corresponds to 0.6:1:2.5:3.5 mix ratio while the least compressive strength predicted is 10.41N/mm^3 for 0.5:1:4:2. A factor was considered to have significant effect on the compressive strength if F value was found to be less than 0.05 (95% confidence level). The F value was obtained from Fisher's distribution table which depends on error degree of freedom (DF) and the mean squares (MS). Table 4.5 shows that the null hypothesis that “there is no significant difference between the experimental and the model expected result” is accepted.

The Fisher F-test revealed that the compressive cube strength predicted by the mathematical model is very close to those obtained from laboratory experiment. The calculated F (F(calculated) = 1.019846) is less than the allowable F or F from statistical table (F(table) = 2.9) at 95% confidence level. Thus, within 95% confidence level, one can predict the compressive cube strength of concrete made with water, cement (OPC), river sand and sandstone using this model.

4.2.2.2 Saturated Surface Dry Density from Model

Table 4.6 presents results of the regression model for cube density. The maximum density of sandstone concrete cube predicted by the model is 2567.7Kg/m^3 and corresponds to mix ratio 0.6:1:1.75:4.25 while the minimum density predicted is 2312.69Kg/m^3 for mix ratio 0.45:1:3.75:2.25. From Table 4.7, we can deduce that the null hypothesis that “there is no significant difference between the experimental and the model expected result” is accepted.

The Fisher F-test revealed that the cube density predicted by the statistical model is very close to those obtained from laboratory experiment. The calculated F (F(calculated) = 1.16429) is less than the allowable F or F from statistical table (F(table) = 2.9) at 95% confidence level. Thus, within 95% confidence level, one can predict the density of concrete made with water, cement (OPC), river sand and sandstone using this model.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

Based on the results obtained from this research work, the following conclusions were drawn:

i. With bulk density (non-compacted) of 1438.72Kg/m3, water absorption value of 2.06%, void ratio of 0.418, specific gravity of 2.467, impact, abrasion and attrition values of 15.31%, 17.92% and 11.46% respectively, the sandstone is a normal weight aggregate and adequate as coarse aggregate for concrete production.

ii. The optimum compressive strength from experiment is 24.18N/mm^2 while that predicted by the model is 22.55 N/mm^2 and both correspond to mix ratio 0.6:1:2.5:3.5 (water, cement, sand and sandstone).

iii. The compressive strength increases as the water/cement ratio increases and fine/coarse aggregate ratio decreases. However, the strength obtained is still less than the target strength of 25 N/mm^2. Also, high water/cement ratio and low fine/coarse aggregate ratio resulted in high densities.

iv. Statistical models for three component (third degree) sandstone concrete were developed using Ibearugbulem's regression method. These statistical regression models were successfully used to predict the compressive strength and density of sandstone concrete when the mix ratios are known.

v. The predictions from the statistical models were tested at 95% accuracy level using statistical fisher test and found to be adequate.

5.2 Recommendations

This research work developed statistical models for predicting compressive strength and density of sandstone concrete. Based on the experimental analysis and conclusions, the following recommendations are given:

i. The use of sandstone in concrete production is encouraged. Future research work should optimize the use of sandstone in combination with another coarse aggregate for concrete, that is to say sandstone-granite or sandstone-gravel concrete.

ii. Similar researches are recommended for concrete beams and slab section to ascertain the flexural behavior of sandstone concrete.

iii. Durability studies of concrete cubes made with sandstone should be carried out.

iv. The models developed in this work can be used with confidence to predict compressive strengths as well as densities of sandstone concrete at 28 days when the mixture proportions are known.

5.3 Contributions to Knowledge

i. This research work developed statistical models for the predicting the compressive strength and density of concrete made from water, OPC, river sand and sandstone.

ii. The models make prescription of compressive strength and density easy when a desired mix ratio is known and so, will help save enormous time and effort invested in carrying out trial mixes.

iii. The work will be a resource material for students of civil engineering who major in structural engineering in both universities and polytechnics.

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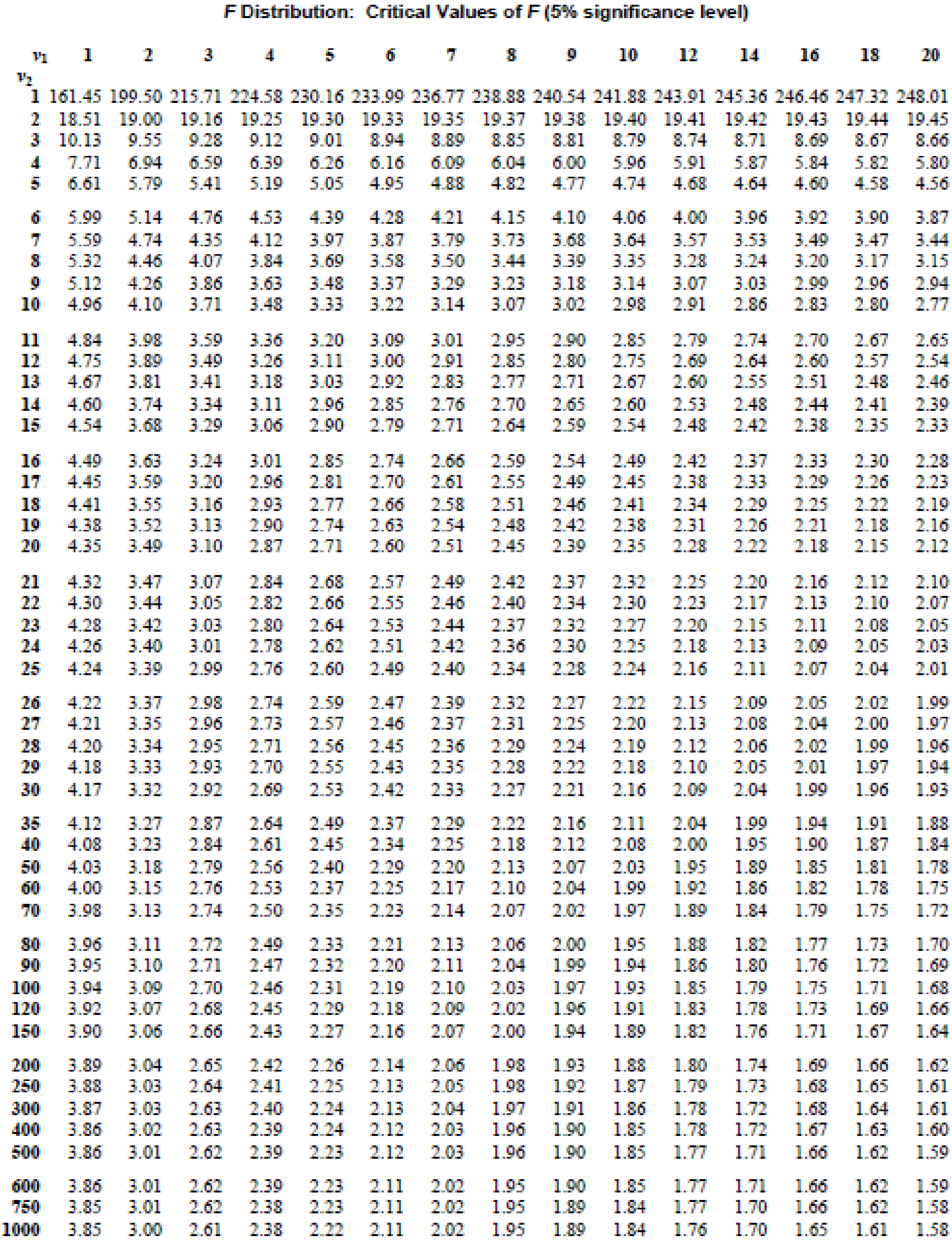
APPENDIX A

Table A.1 Critical values of the F distribution



F Distribution: Critical Values of F (5% significance level)

v_1 v_2	1	2	3	4	5	6	7	8	9	10	12	14	16	18	20
1	161.45	199.50	215.71	224.58	230.16	233.99	236.77	238.88	240.54	241.88	243.91	245.36	246.46	247.32	248.01
2	18.51	19.00	19.16	19.25	19.30	19.33	19.35	19.37	19.38	19.40	19.41	19.42	19.43	19.44	19.45
3	10.13	9.55	9.28	9.12	9.01	8.94	8.89	8.85	8.81	8.79	8.74	8.71	8.69	8.67	8.66
4	7.71	6.94	6.59	6.39	6.26	6.16	6.09	6.04	6.00	5.96	5.91	5.87	5.84	5.82	5.80
5	6.61	5.79	5.41	5.19	5.05	4.95	4.88	4.82	4.77	4.74	4.68	4.64	4.60	4.58	4.56
6	5.99	5.14	4.76	4.53	4.39	4.28	4.21	4.15	4.10	4.06	4.00	3.96	3.92	3.90	3.87
7	5.59	4.74	4.35	4.12	3.97	3.87	3.79	3.73	3.68	3.64	3.57	3.53	3.49	3.47	3.44
8	5.32	4.46	4.07	3.84	3.69	3.58	3.50	3.44	3.39	3.35	3.28	3.24	3.20	3.17	3.15
9	5.12	4.26	3.86	3.63	3.48	3.37	3.29	3.23	3.18	3.14	3.07	3.03	2.99	2.96	2.94
10	4.96	4.10	3.71	3.48	3.33	3.22	3.14	3.07	3.02	2.98	2.91	2.86	2.83	2.80	2.77
11	4.84	3.98	3.59	3.36	3.20	3.09	3.01	2.95	2.90	2.85	2.79	2.74	2.70	2.67	2.65
12	4.75	3.89	3.49	3.26	3.11	3.00	2.91	2.85	2.80	2.75	2.69	2.64	2.60	2.57	2.54
13	4.67	3.81	3.41	3.18	3.03	2.92	2.83	2.77	2.71	2.67	2.60	2.55	2.51	2.48	2.46
14	4.60	3.74	3.34	3.11	2.96	2.85	2.76	2.70	2.65	2.60	2.53	2.48	2.44	2.41	2.39
15	4.54	3.68	3.29	3.06	2.90	2.79	2.71	2.64	2.59	2.54	2.48	2.42	2.38	2.35	2.33
16	4.49	3.63	3.24	3.01	2.85	2.74	2.66	2.59	2.54	2.49	2.42	2.37	2.33	2.30	2.28
17	4.45	3.59	3.20	2.96	2.81	2.70	2.61	2.55	2.49	2.45	2.38	2.33	2.29	2.26	2.23
18	4.41	3.55	3.16	2.93	2.77	2.66	2.58	2.51	2.46	2.41	2.34	2.29	2.25	2.22	2.19
19	4.38	3.52	3.13	2.90	2.74	2.63	2.54	2.48	2.42	2.38	2.31	2.26	2.21	2.18	2.16
20	4.35	3.49	3.10	2.87	2.71	2.60	2.51	2.45	2.39	2.35	2.28	2.22	2.18	2.15	2.12
21	4.32	3.47	3.07	2.84	2.68	2.57	2.49	2.42	2.37	2.32	2.25	2.20	2.16	2.12	2.10
22	4.30	3.44	3.05	2.82	2.66	2.55	2.46	2.40	2.34	2.30	2.23	2.17	2.13	2.10	2.07
23	4.28	3.42	3.03	2.80	2.64	2.53	2.44	2.37	2.32	2.27	2.20	2.15	2.11	2.08	2.05
24	4.26	3.40	3.01	2.78	2.62	2.51	2.42	2.36	2.30	2.25	2.18	2.13	2.09	2.05	2.03
25	4.24	3.39	2.99	2.76	2.60	2.49	2.40	2.34	2.28	2.24	2.16	2.11	2.07	2.04	2.01
26	4.22	3.37	2.98	2.74	2.59	2.47	2.39	2.32	2.27	2.22	2.15	2.09	2.05	2.02	1.99
27	4.21	3.35	2.96	2.73	2.57	2.46	2.37	2.31	2.25	2.20	2.13	2.08	2.04	2.00	1.97
28	4.20	3.34	2.95	2.71	2.56	2.45	2.36	2.29	2.24	2.19	2.12	2.06	2.02	1.99	1.96
29	4.18	3.33	2.93	2.70	2.55	2.43	2.35	2.28	2.22	2.18	2.10	2.05	2.01	1.97	1.94
30	4.17	3.32	2.92	2.69	2.53	2.42	2.33	2.27	2.21	2.16	2.09	2.04	1.99	1.96	1.93
35	4.12	3.27	2.87	2.64	2.49	2.37	2.29	2.22	2.16	2.11	2.04	1.99	1.94	1.91	1.88
40	4.08	3.23	2.84	2.61	2.45	2.34	2.25	2.18	2.12	2.08	2.00	1.95	1.90	1.87	1.84
50	4.03	3.18	2.79	2.56	2.40	2.29	2.20	2.13	2.07	2.03	1.95	1.89	1.85	1.81	1.78
60	4.00	3.15	2.76	2.53	2.37	2.25	2.17	2.10	2.04	1.99	1.92	1.86	1.82	1.78	1.75
70	3.98	3.13	2.74	2.50	2.35	2.23	2.14	2.07	2.02	1.97	1.89	1.84	1.79	1.75	1.72
80	3.96	3.11	2.72	2.49	2.33	2.21	2.13	2.06	2.00	1.95	1.88	1.82	1.77	1.73	1.70
90	3.95	3.10	2.71	2.47	2.32	2.20	2.11	2.04	1.99	1.94	1.86	1.80	1.76	1.72	1.69
100	3.94	3.09	2.70	2.46	2.31	2.19	2.10	2.03	1.97	1.93	1.85	1.79	1.75	1.71	1.68
120	3.92	3.07	2.68	2.45	2.29	2.18	2.09	2.02	1.96	1.91	1.83	1.78	1.73	1.69	1.66
150	3.90	3.06	2.66	2.43	2.27	2.16	2.07	2.00	1.94	1.89	1.82	1.76	1.71	1.67	1.64
200	3.89	3.04	2.65	2.42	2.26	2.14	2.06	1.98	1.93	1.88	1.80	1.74	1.69	1.66	1.62
250	3.88	3.03	2.64	2.41	2.25	2.13	2.05	1.98	1.92	1.87	1.79	1.73	1.68	1.65	1.61
300	3.87	3.03	2.63	2.40	2.24	2.13	2.04	1.97	1.91	1.86	1.78	1.72	1.68	1.64	1.61
400	3.86	3.02	2.63	2.39	2.24	2.12	2.03	1.96	1.90	1.85	1.78	1.72	1.67	1.63	1.60
500	3.86	3.01	2.62	2.39	2.23	2.12	2.03	1.96	1.90	1.85	1.77	1.71	1.66	1.62	1.59
600	3.86	3.01	2.62	2.39	2.23	2.11	2.02	1.95	1.90	1.85	1.77	1.71	1.66	1.62	1.59
750	3.85	3.01	2.62	2.38	2.23	2.11	2.02	1.95	1.89	1.84	1.77	1.70	1.66	1.62	1.58
1000	3.85	3.00	2.61	2.38	2.22	2.11	2.02	1.95	1.89	1.84	1.76	1.70	1.65	1.61	1.58

(Source: Dougherty, 2002)

APPENDIX B

AGGREGATES CHARACTERIZATION TESTS PROCEDURES

B1 Physical Characterization Tests of Aggregates

The tests of physical properties of the materials used for this work were carried out at the Concrete Laboratory of Civil Engineering Department, Federal Polytechnic, Nekede. Specific gravity, bulk density, water absorption, void ratio and sieve analysis of sand and sandstone were tested. The apparatus used include a 50kg weighing machine, four plastic baths, a flat table, a calibrated cylindrical glass jar, hydrometer and scoop. Others were 16mm steel rod, a 150mm x 150mm steel cube container, three plastic buckets, three plastic baskets and four standard sieves (19.5mm, 14mm, 10mm and 5.6mm). Relevant standards include; ASTM C29, 2004; ASTM C125, 2003; ASTM C33, 2004)

B1.1 Bulk Density

The objective of this test was to determine the compacted and un-compacted bulk density of river sand and sandstone used for this research work.

Apparatus:

i. Mould of known volume
ii. Weighing machine
iii. 16mm steel rod
iv. Plate
v. Scoop.

Procedure:

i. For non-compacted samples, a cylindrical mould is weighed empty with the thin plate and designated, W_{tools}. The aggregates (river sand and sandstone in turns) were loosely poured into the mould container till it becomes over filled. The plate is used to level the surface of the container. The weight of the container, the aggregate sample, plus the plate was found as W_1. This is repeated two more times and masses recorded.

ii. For the compacted samples, the container is filled in three layers using the scoop. Each layer is about one - third of the volume of the container, when the first layer is poured in, the 16mm rod is used to tamp on it 35 times in accordance with ASTM C29, 20004. This is done for the second and third layers and then leveled and covered with the plate. The mass of the container, the sample, plus the plate is weighed and recorded as W_2. This is repeated two more times and masses recorded.

iii. The container was emptied again and this time was filled with water till the rim of it and the plate was placed on it, no water bubble was present on the surface. The volume of the container was recorded as V. Equation B.1 was used to obtain the average bulk densities and the results are presented on Tables C.1 and C.2.

$$\text{Bulk density} = \frac{W1 - Wtools}{V} \qquad \text{(B.1)}$$

B1.2 Water Absorption

This test was carried out on coarse aggregate – sandstone in accordance with ASTM C127, 2004).

Apparatus:

i. Plastic buckets
ii. Weighing machine
iii. Plastic baskets
iv. Scoop.

Procedure:

Three dry samples of sandstone were collected using three plastic buckets. The weighing machine with its tray is set at zero point. After setting the weighing machine, the masses of the three samples were weighed and recorded. The weighed samples were put back into the three plastic buckets and topped with distilled water of known specific gravity. The buckets were allowed to stay for 24hours, after which they were emptied into three different plastic baskets, which were allowed to stand for 1hour inside the laboratory. At the end of the 1hour the baskets were respectively emptied into the tray of the weighing machine and the respective masses weighed and recorded.

The expression given in Equation B.2 was used to obtain the percentage water absorption of the sample. The result obtained is shown in Table C.3.

$$\% \text{ water absorption} = \frac{Difference\ in\ weight}{Initial\ weight} \times 100 \qquad \text{(B.2)}$$

That is,

$$\% \text{ water absorption} = \frac{W2-W1}{W1} \times 100$$

Where:

W_1 = Initial weight of sample (g)

W_2 = Weight after 24 hours (g)

B1.3 Void Ratio

The procedure for void ratio is covered by the procedures of specific gravity and bulk density. Since the bulk density and the voids between aggregate mixes are inversely proportional, our aim would be to use an aggregate of very high bulk density because the higher the bulk density, the minimal the voids. Equation B.4 was used to obtain the ratio of the coarse aggregate used in this experiment. The result is given in Table C.4.

Void ratio = 1 - (Bulk density / Specific gravity)

$$\text{Voids } \alpha \frac{1}{R.D} \qquad \text{or} \qquad \text{Percentage void} = \frac{S.G-R.D}{S.G} \times 100\% \qquad \text{(B.3)}$$

B1.4 Specific Gravity

The objective was to determine the specific gravities (relative density) of river sand and sandstone used for this research, and then relate the result obtained to the relevant standards (BS 812, 1990; ASTM C127, 2004). The specific gravity was deduced from Equation B.4 by taking the average of three tests.

$$S.G = \frac{M2-M1}{(M4-M1)-(M3-M2)} \qquad \text{(B.4)}$$

Apparatus:

i. Container
ii. Glass Plate
iii. Shallow Trays
iv. Dry soft absorbent clothes
v. Fine and coarse aggregate samples
vi. Weighing machine
vii. Sieves No. 9(mm), No. 4(mm) and No. 200(75µ)
viii. Stove.

Procedure:

i. The density bottle with the stopper was dried and weighed to the nearest 0.001g as W_1.
ii. The sample, which had been dried, was transferred to the density bottle from the desiccator in which it was cooled. The bottles and contents together with the stopper were weighed to the nearest 0.001g as W_2.
iii. The sample was covered with air-free distilled water. Wash bottle used to expel entrapped air.
iv. The sample in the density bottle was gently stirred with a clean glass rod, and care was taken to wash off the adhering particles from the rod with some drops of distilled water so that no sample particles were lost.
v. After the stopper was inserted in the density bottle, it was cleaned and weighed as W_3.
vi. The bottle was emptied, cleaned thoroughly and filled with water. The stopper was inserted in the bottle, wiped dry from the outside and weighed as W_4.
vii. Three observations were made for each specimen.

The results of specific gravities of the aggregates worked on are presented in Table C.5.

B1.5 Sieve Analysis

This test was done to determine the particle size distribution of the coarse and fine aggregates in general accordance with BS 812 (1985). A set of BS Sieve sizes – 37mm, 25mm, 20mm, 16mm, 12.5mm, 10mm, 6.3mm, 4.75mm, 3.35mm, 2.36mm, 1.18mm, 600µm, 300µm and 75µm was used as shown in Plate B.1. The materials tested were river sand and sandstone.

Apparatus:

i) Sieves

ii) Weighing machine iii) Scoop

Plate B.1 Sieve Analysis

Procedure:

i. The first thing done was to set the weighing machine with its tray to zero point. Then dry test sample was collected and poured into the tray of the weighing machine and the mass is weighed and recorded.

ii. At this point the weighed sample is poured into the 37mm sieve and by manual operation using hand the sample is sieved. The portion that passed through the sieve was collected in the 25mm sieve and manually sieved using hand. This process was continued with sieves 20mm, 16mm, and 12.5mm in that order. The portion that passed through 75µm sieve was collected using the receiver.

iii. Masses of the portions retained on 37mm, 25mm, 20mm, 16mm, 12.5mm, 10mm, 6.3mm, 4.75mm, 3.35mm, 2.36mm, 1.18mm, 600µm, 300µm and 75µm and the receiver were respectively weighed using the weighing machine. Their various masses were recorded.

iv. Cumulative weight passing through each sieve was calculated as a percentage of the total sample weight.

v. Fineness modulus was obtained by adding cumulative percentage of aggregates retained on each sieve and dividing the sum by 100.

The sieve analysis results of the aggregates tested are presented in Tables C.6 and B.7 and their grain size distribution are given in Figures 3.1 and 3.2 where graph of cumulative percentage passing is plotted against sieve sizes. The coefficient of curvature (Cc) and the coefficient of uniformity (Cu) are two important parameters used to examine the gradation of the aggregates used in this experimental work. Coefficient of uniformity measures of the uniformity of sizes of particles that made up the sample while coefficient of curvature measures the symmetry and the shape of the gradation curve.

The parameters are deduced from the following relationships:

$$C_u = \frac{D_{60}}{D_{10}} \qquad \text{(B.5)}$$

$$C_c = \frac{{D_{30}}^2}{D_{60} * D_{10}} \qquad \text{(B.6)}$$

Where:

C_u = Coefficient of uniformity.

C_c = Coefficient of curvature.

D_{60} = Particle size that is coarser than 60% (by weight) of the sample.

D_{30} = Particle size that is coarser than 30% (by weight) of the sample.

D_{10} = Effective size or the particle size that is coarser than 10% (by weight) of the sample.

B2 Mechanical Characterization Tests of Sandstone

Impact value, Abrasion value and Attrition value of sandstone were tested to determine how various mechanical exposure of the sandstone affected the overall strength of the concrete. Steel cylindrical container (148mm diameter and 297mm height), a standard hammer (BS 812: part 3: 1985), and 20kg weighing machine and scoop are some of the apparatus used. Others are 14mm, 13.2mm, 11.2mm, 10mm, 2.36mm and 2.00mm standard sieves, 12 steel charges of 92.5g each and Los Angeles Abrasion test machine.

B2.1 Impact Value

This test was carried out in accordance with BS 812 (1975).

Apparatus:

i. Impact testing machine
ii. BS Sieves of sizes – 12.5mm, 10mm and 2.36mm
iii. A steel cylindrical container
iv. A tamping rod

Procedure:

i. The first thing done was to arrange the 14mm, 10mm sieves and the receiver in that order. The dry sample of the sandstone was collected using the scoop and placed into the 14mm sieve.
ii. The actual sieving operation was done manually by hand. After handling the 14mm sieve, 10mm sieve is handled. The portion that passes 14mm sieve but did

not pass through 10mm sieve was collected, and it was poured into the tray of the weighing machine, which has been set to the zero point initially. Mass of the collected sample was weighed and recorded.

iii. Then the weighed sandstones were poured into the steel cylinder where the impact was applied. The impact was applied using the standard hammer that fall 15 times under its own weight upon the sandstone in the container (cylinder).

iv. After the impact the content of the cylinder was emptied into the 2.36mm sieve where it was sieved by manual operation using hand. Masses of portions that passes the 2.36mm sieve and the one that was retained by it were weighed using the weighing machine and thus, recorded. This process was repeated two more times.

The results obtained are presented in Table C.8 in chapter four of this work.

3.2.2.2 Abrasion and Attrition Values

This test was conducted in the Structures laboratory of Federal Polytechnics Nekede in accordance with ASTM C131.

Apparatus:

i. Los Angeles Abrasion testing machine

ii. Sieves of 13.2mm, 9.6mm, 2.36mm and 2.00mm

iii. Weighing machine

iii. Abrasive charge – 12 steel spheres approximately 48nn diameter and each weighing between 390 and 445g ensuring that the total weight of charge was 5000 + 25g.

Procedure:

i. Sieves of 13.2mm and 11.2mm and the receiver were arranged in that order. Dry sample of sandstone was placed in the 13.2mm sieve. Sieving was done manually using hand. The portion that passes through 13.2mm sieve and retains on the 9.6mm sieve was taken. Mass of the collected portion was weighed using the weighing machine and recorded. The weighed sample was poured into the Los Angeles abrasion test machine.

ii. For abrasion test, the 12 steel charges were poured into the machine and the machine closed. For attrition test, the steel charges were not added. At this point the machine is rotated at a speed of about 32 revolutions per minute for 500 revolutions. At the end of the 500 revolutions the sample was removed from the machine and preliminarily separated using the 2.36mm sieves.

iii. Portion of the sample that passes through the 2.36mm sieve was sieved manually also using hand with the 2.00mm sieve. All the portions that did not pass through the 2.00mm sieve were poured into the weighing machine and the mass weighed and recorded.

The results obtained from abrasion and attrition tests are as shown in Tables C.9 and C.10.

Plate B.2 Batching of Sand

Plate B.3 Batching of Sandstone

Plate B.4 Batching of Cement

Plate B.5 Batched Sand and Sandstone

Plate B.6 Mixing of Concrete Materials

Plate B.7 Casting of Cubes after Mixing

Plate B.8 Already Cast Cubes in Replicates

Plate B.9 Cubes Being Immersed in Open Curing Tank

Plate B.10 Slump Test (a)

Plate B.11 Slump Test (b)

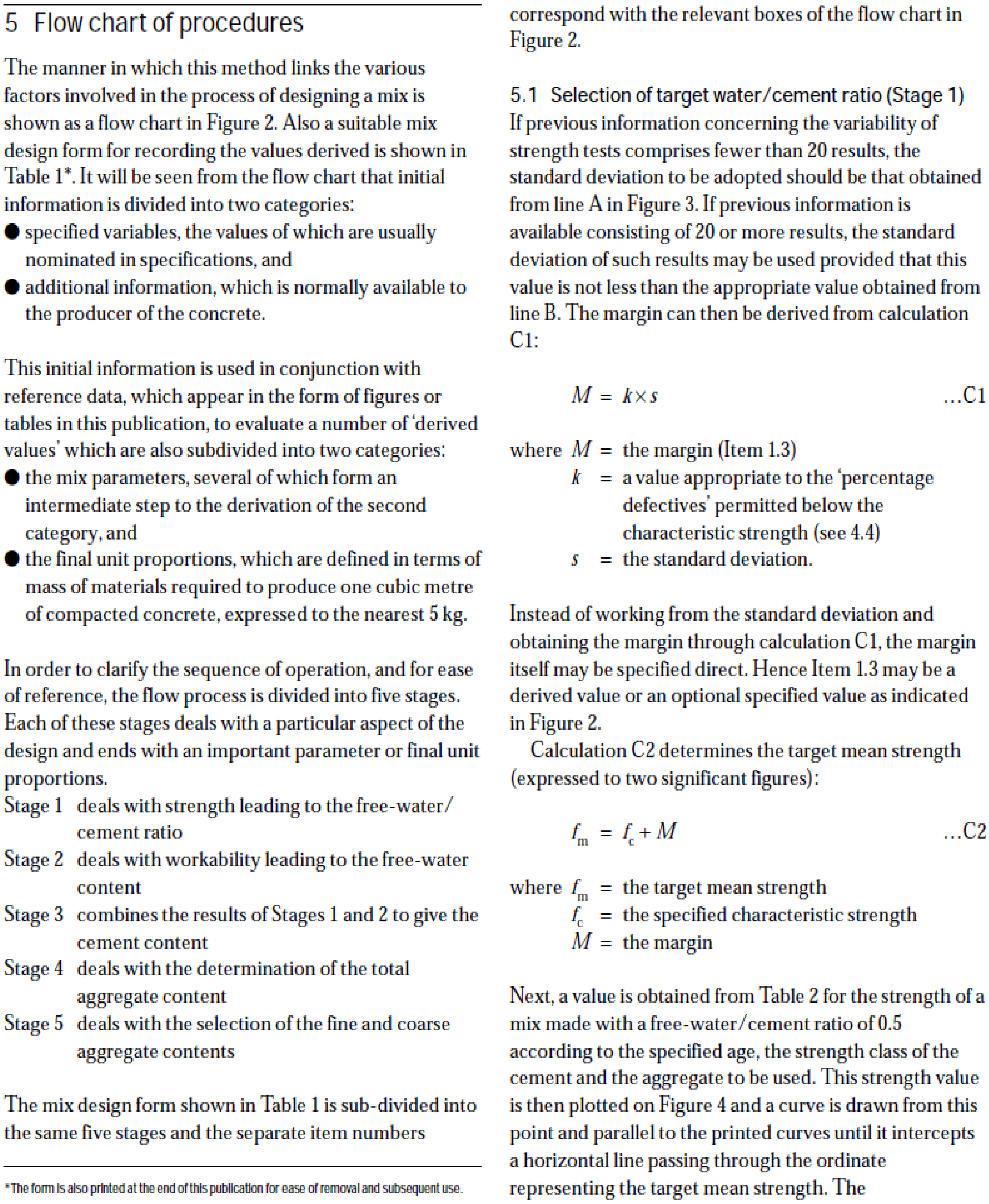
Plate B.12 Cubes Ready to be crushed

Plate B.13 Concrete Cube Placed on Crushing Machine

Plate B.14 Crushing and Recording of Results

APPENDIX C

Mix Design Tables, Charts and Figures



5 Flow chart of procedures

The manner in which this method links the various factors involved in the process of designing a mix is shown as a flow chart in Figure 2. Also a suitable mix design form for recording the values derived is shown in Table 1*. It will be seen from the flow chart that initial information is divided into two categories:

- specified variables, the values of which are usually nominated in specifications, and
- additional information, which is normally available to the producer of the concrete.

This initial information is used in conjunction with reference data, which appear in the form of figures or tables in this publication, to evaluate a number of 'derived values' which are also subdivided into two categories:

- the mix parameters, several of which form an intermediate step to the derivation of the second category, and
- the final unit proportions, which are defined in terms of mass of materials required to produce one cubic metre of compacted concrete, expressed to the nearest 5 kg.

In order to clarify the sequence of operation, and for ease of reference, the flow process is divided into five stages. Each of these stages deals with a particular aspect of the design and ends with an important parameter or final unit proportions.

Stage 1	deals with strength leading to the free-water/cement ratio
Stage 2	deals with workability leading to the free-water content
Stage 3	combines the results of Stages 1 and 2 to give the cement content
Stage 4	deals with the determination of the total aggregate content
Stage 5	deals with the selection of the fine and coarse aggregate contents

The mix design form shown in Table 1 is sub-divided into the same five stages and the separate item numbers correspond with the relevant boxes of the flow chart in Figure 2.

5.1 Selection of target water/cement ratio (Stage 1)

If previous information concerning the variability of strength tests comprises fewer than 20 results, the standard deviation to be adopted should be that obtained from line A in Figure 3. If previous information is available consisting of 20 or more results, the standard deviation of such results may be used provided that this value is not less than the appropriate value obtained from line B. The margin can then be derived from calculation C1:

$$M = k \times s \qquad \text{...C1}$$

where M = the margin (Item 1.3)
k = a value appropriate to the 'percentage defectives' permitted below the characteristic strength (see 4.4)
s = the standard deviation.

Instead of working from the standard deviation and obtaining the margin through calculation C1, the margin itself may be specified direct. Hence Item 1.3 may be a derived value or an optional specified value as indicated in Figure 2.

Calculation C2 determines the target mean strength (expressed to two significant figures):

$$f_m = f_c + M \qquad \text{...C2}$$

where f_m = the target mean strength
f_c = the specified characteristic strength
M = the margin

Next, a value is obtained from Table 2 for the strength of a mix made with a free-water/cement ratio of 0.5 according to the specified age, the strength class of the cement and the aggregate to be used. This strength value is then plotted on Figure 4 and a curve is drawn from this point and parallel to the printed curves until it intercepts a horizontal line passing through the ordinate representing the target mean strength. The

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*The form is also printed at the end of this publication for ease of removal and subsequent use.

(Source: Teychenné et al., 1997)

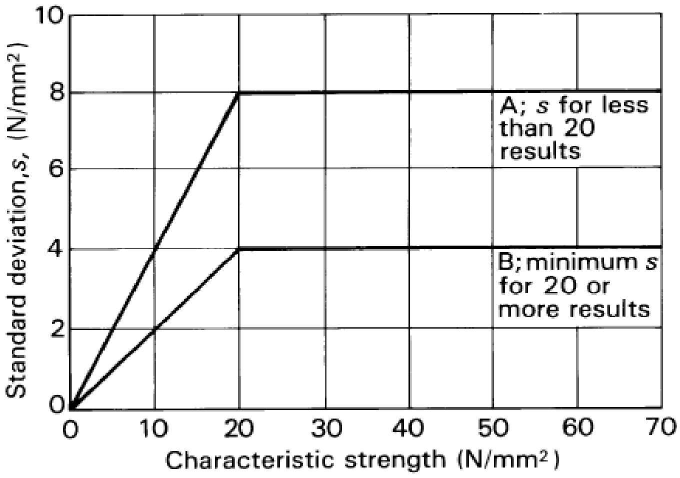

Figure 3
Relationship between standard deviation and characteristic strength

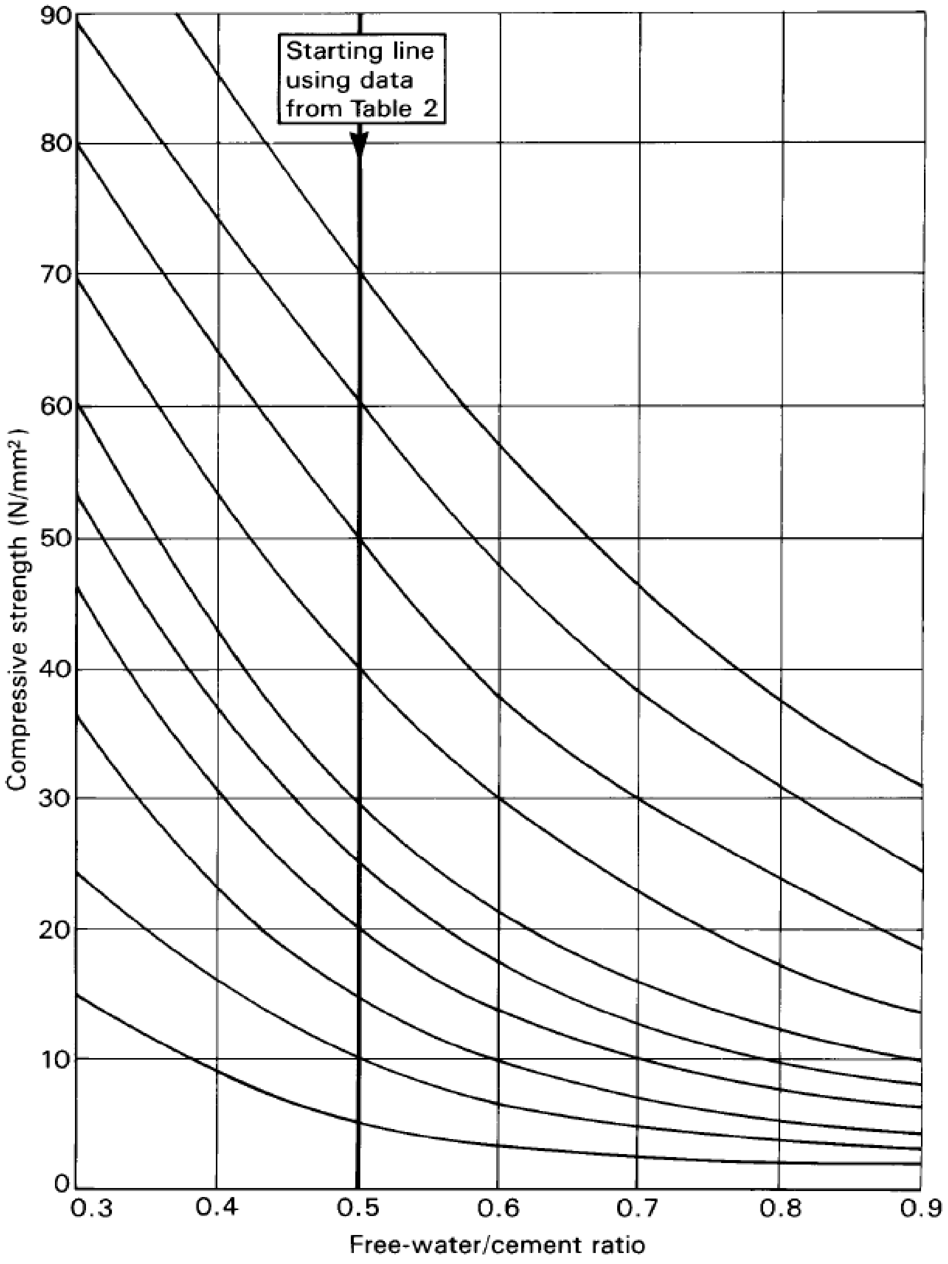

Figure 4
Relationship between compressive strength and free-water/cement ratio

(Source: Teychenné et al., 1997)

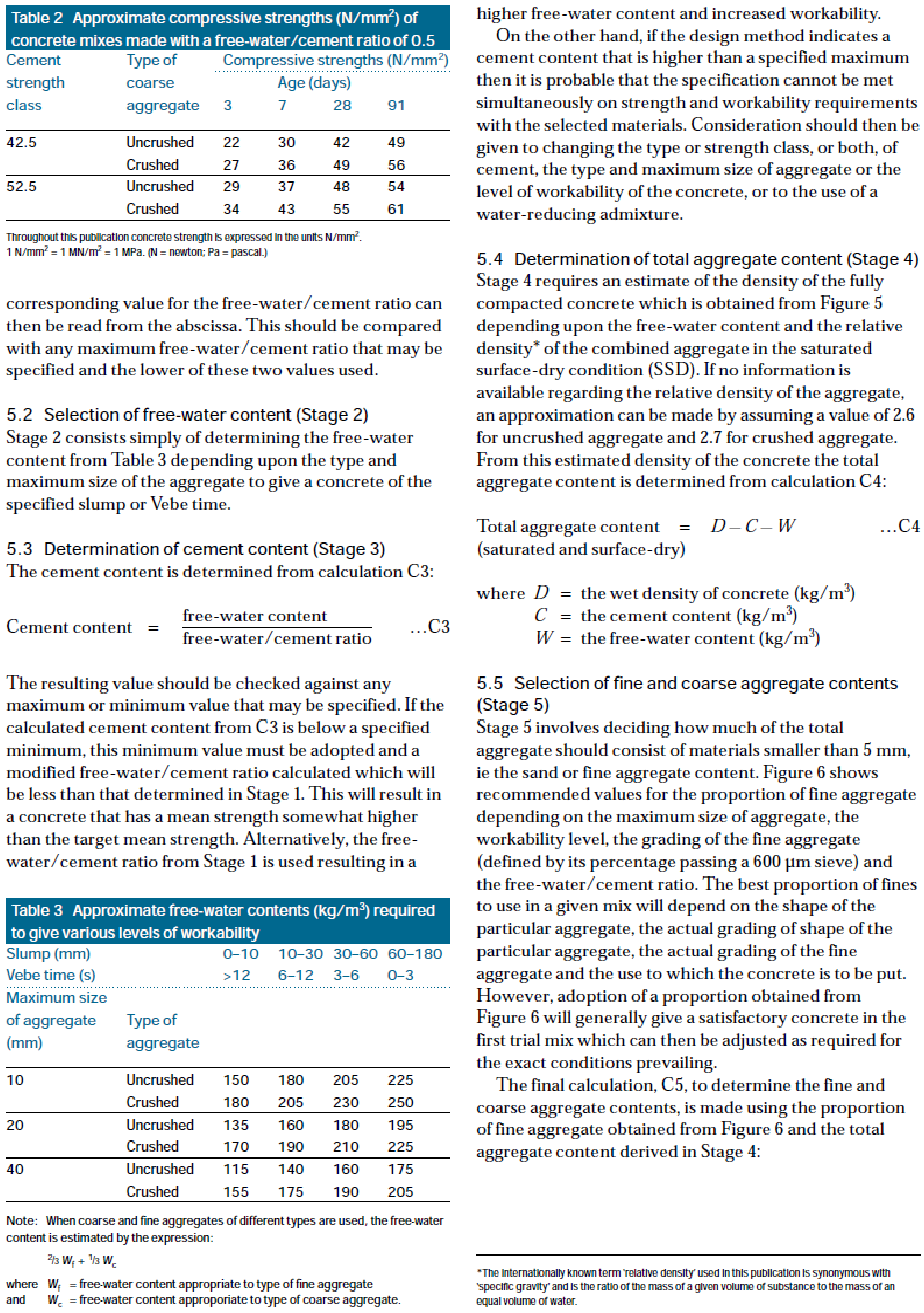

Table 2 Approximate compressive strengths (N/mm^2) of concrete mixes made with a free-water/cement ratio of 0.5

Cement strength class	Type of coarse aggregate	Compressive strengths (N/mm^2) Age (days) 3	7	28	91
42.5	Uncrushed	22	30	42	49
	Crushed	27	36	49	56
52.5	Uncrushed	29	37	48	54
	Crushed	34	43	55	61

Throughout this publication concrete strength is expressed in the units N/mm^2.
1 N/mm^2 = 1 MN/m^2 = 1 MPa. (N = newton; Pa = pascal.)

corresponding value for the free-water/cement ratio can then be read from the abscissa. This should be compared with any maximum free-water/cement ratio that may be specified and the lower of these two values used.

5.2 Selection of free-water content (Stage 2)

Stage 2 consists simply of determining the free-water content from Table 3 depending upon the type and maximum size of the aggregate to give a concrete of the specified slump or Vebe time.

5.3 Determination of cement content (Stage 3)

The cement content is determined from calculation C3:

$$\text{Cement content} = \frac{\text{free-water content}}{\text{free-water/cement ratio}} \qquad \ldots\text{C3}$$

The resulting value should be checked against any maximum or minimum value that may be specified. If the calculated cement content from C3 is below a specified minimum, this minimum value must be adopted and a modified free-water/cement ratio calculated which will be less than that determined in Stage 1. This will result in a concrete that has a mean strength somewhat higher than the target mean strength. Alternatively, the free-water/cement ratio from Stage 1 is used resulting in a higher free-water content and increased workability.

Table 3 Approximate free-water contents (kg/m^3) required to give various levels of workability

Slump (mm)		0–10	10–30	30–60	60–180
Vebe time (s)		>12	6–12	3–6	0–3
Maximum size of aggregate (mm)	Type of aggregate				
10	Uncrushed	150	180	205	225
	Crushed	180	205	230	250
20	Uncrushed	135	160	180	195
	Crushed	170	190	210	225
40	Uncrushed	115	140	160	175
	Crushed	155	175	190	205

Note: When coarse and fine aggregates of different types are used, the free-water content is estimated by the expression:

$$\tfrac{2}{3} W_f + \tfrac{1}{3} W_c$$

where W_f = free-water content appropriate to type of fine aggregate
and W_c = free-water content approporiate to type of coarse aggregate.

On the other hand, if the design method indicates a cement content that is higher than a specified maximum then it is probable that the specification cannot be met simultaneously on strength and workability requirements with the selected materials. Consideration should then be given to changing the type or strength class, or both, of cement, the type and maximum size of aggregate or the level of workability of the concrete, or to the use of a water-reducing admixture.

5.4 Determination of total aggregate content (Stage 4)

Stage 4 requires an estimate of the density of the fully compacted concrete which is obtained from Figure 5 depending upon the free-water content and the relative density* of the combined aggregate in the saturated surface-dry condition (SSD). If no information is available regarding the relative density of the aggregate, an approximation can be made by assuming a value of 2.6 for uncrushed aggregate and 2.7 for crushed aggregate. From this estimated density of the concrete the total aggregate content is determined from calculation C4:

$$\text{Total aggregate content (saturated and surface-dry)} = D - C - W \qquad \ldots\text{C4}$$

where D = the wet density of concrete (kg/m^3)
C = the cement content (kg/m^3)
W = the free-water content (kg/m^3)

5.5 Selection of fine and coarse aggregate contents (Stage 5)

Stage 5 involves deciding how much of the total aggregate should consist of materials smaller than 5 mm, ie the sand or fine aggregate content. Figure 6 shows recommended values for the proportion of fine aggregate depending on the maximum size of aggregate, the workability level, the grading of the fine aggregate (defined by its percentage passing a 600 µm sieve) and the free-water/cement ratio. The best proportion of fines to use in a given mix will depend on the shape of the particular aggregate, the actual grading of shape of the particular aggregate, the actual grading of the fine aggregate and the use to which the concrete is to be put. However, adoption of a proportion obtained from Figure 6 will generally give a satisfactory concrete in the first trial mix which can then be adjusted as required for the exact conditions prevailing.

The final calculation, C5, to determine the fine and coarse aggregate contents, is made using the proportion of fine aggregate obtained from Figure 6 and the total aggregate content derived in Stage 4:

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*The internationally known term 'relative density' used in this publication is synonymous with 'specific gravity' and is the ratio of the mass of a given volume of substance to the mass of an equal volume of water.

(Source: Teychenné et al., 1997)

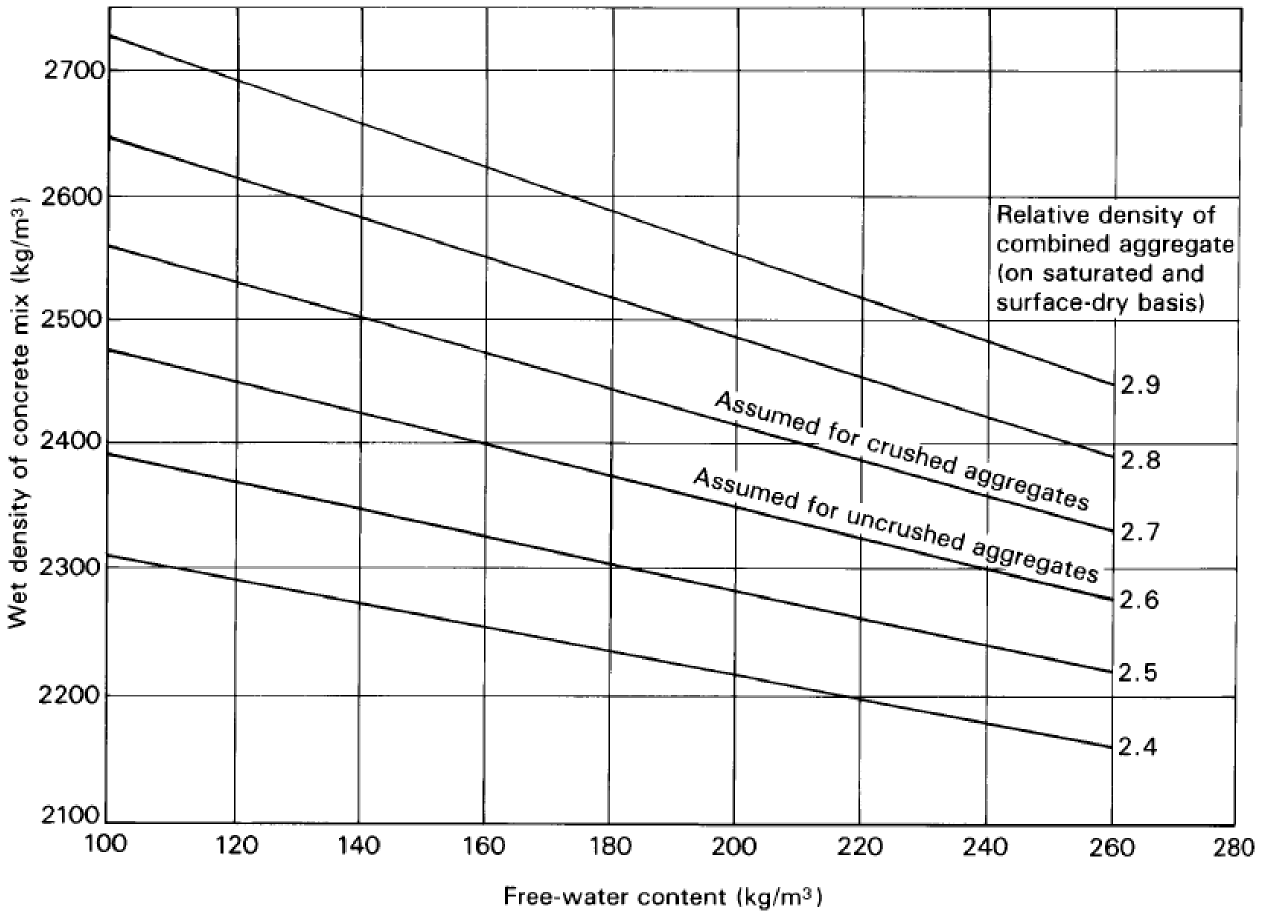


Figure 5 Estimated wet density of fully compacted concrete

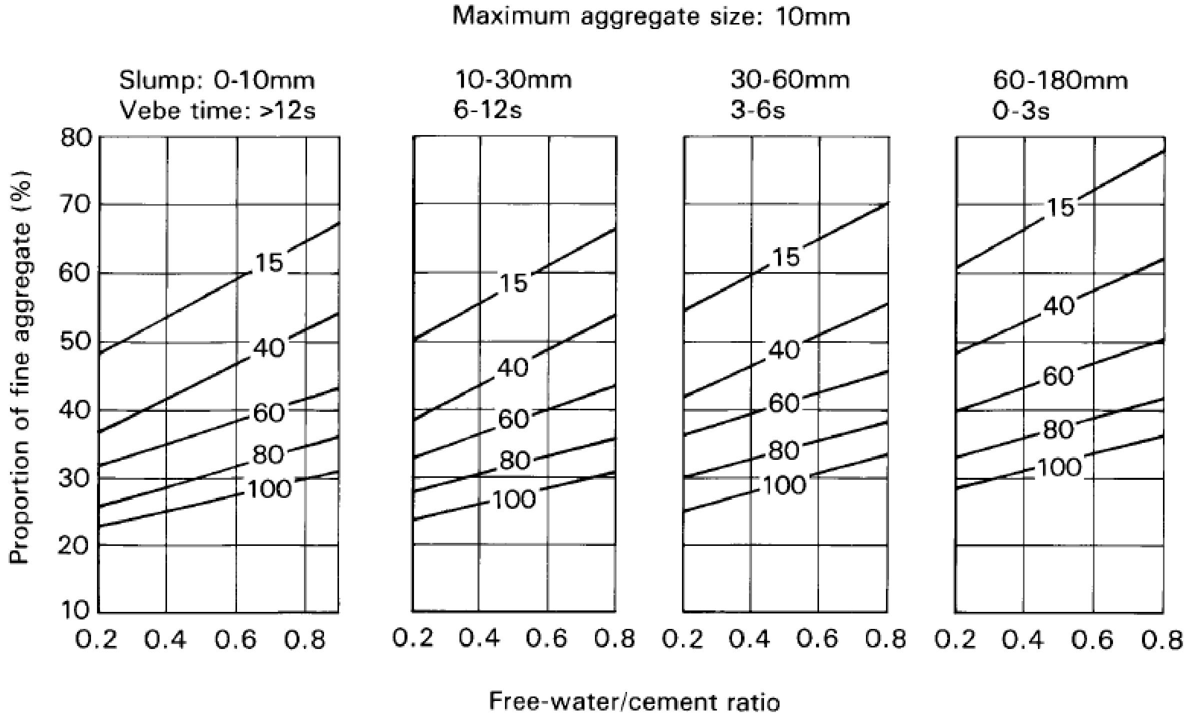


Figure 6 Recommended proportions of fine aggregate according to percentage passing a 600 μm sieve

(Source: Teychenné et al., 1997)

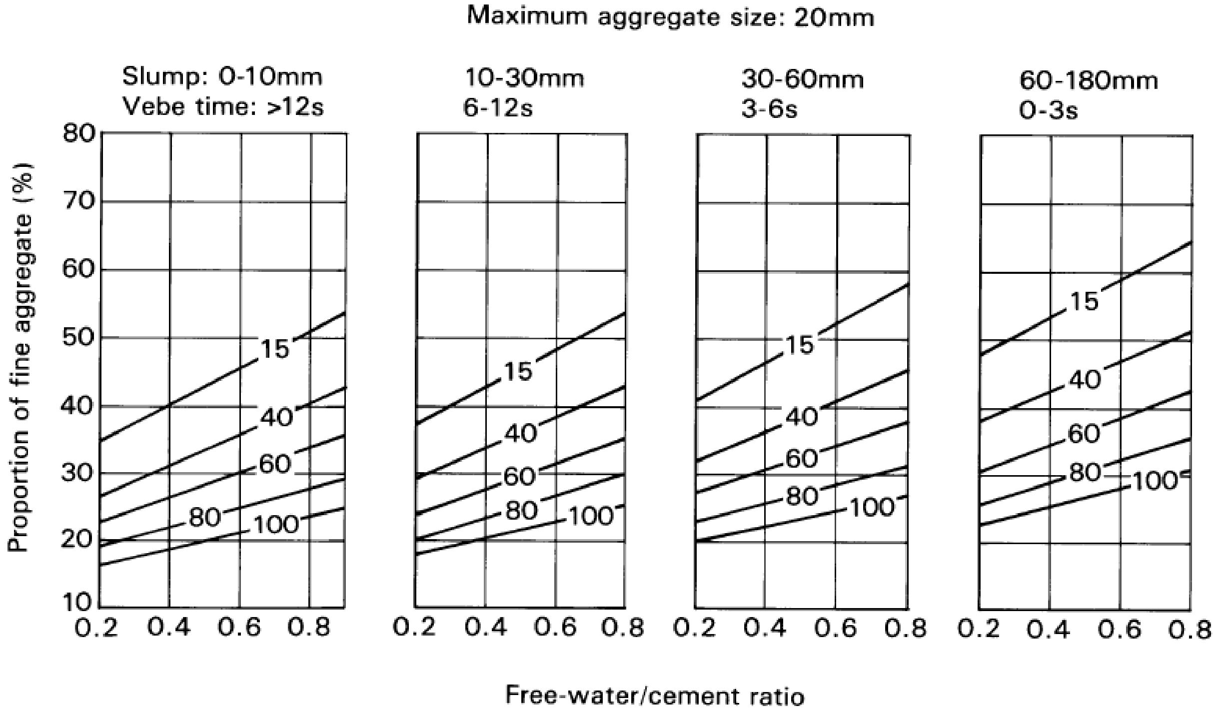


Figure 6 (continued)

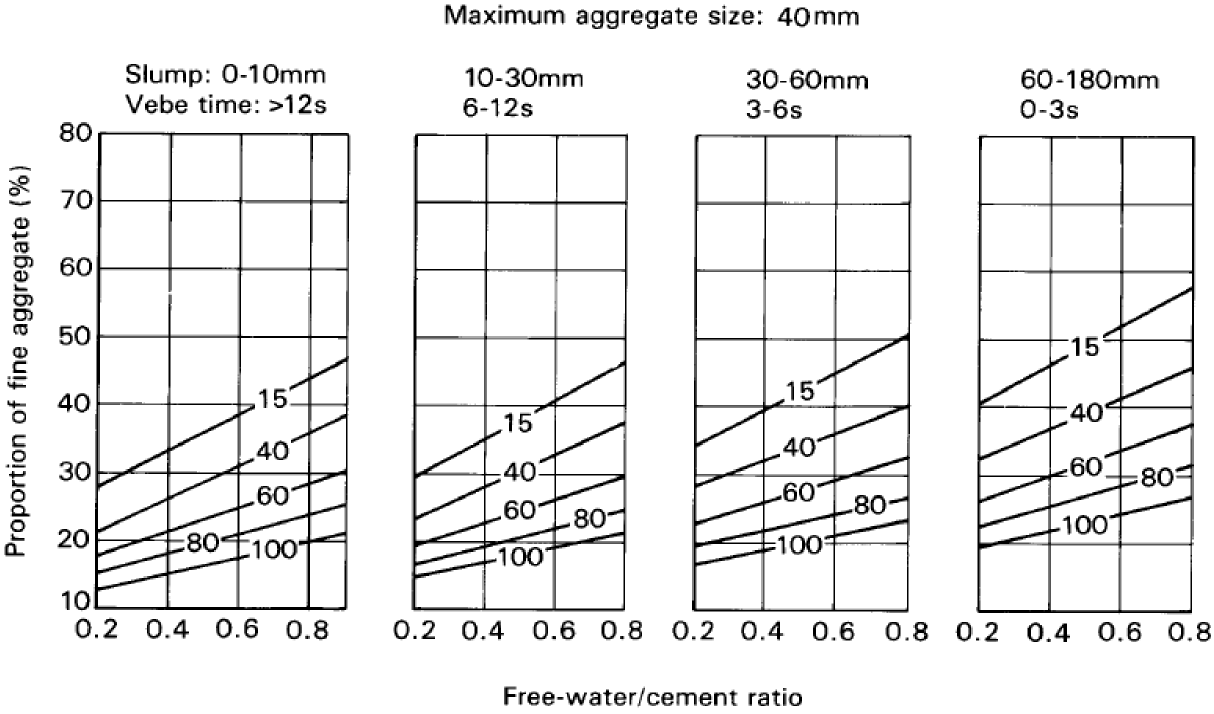
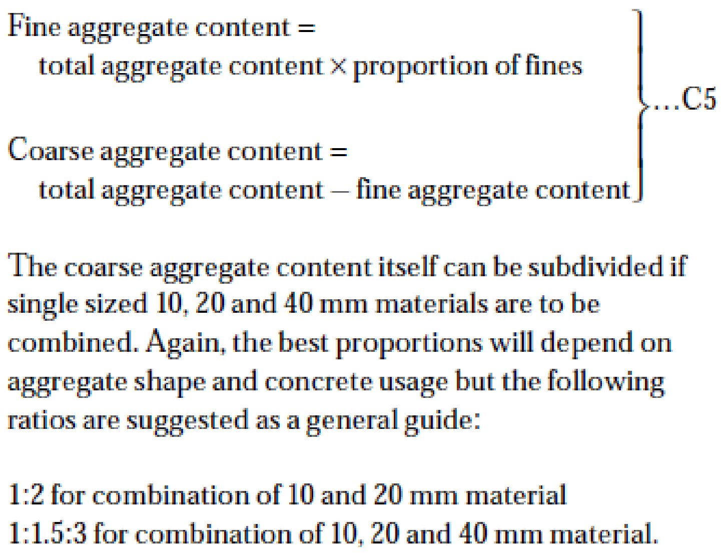


Figure 6 (continued)

(Source: Teychenné et al., 1997)



$$\left.\begin{array}{l}\text{Fine aggregate content} = \\ \quad \text{total aggregate content} \times \text{proportion of fines} \\ \\ \text{Coarse aggregate content} = \\ \quad \text{total aggregate content} - \text{fine aggregate content}\end{array}\right\}\ldots\text{C5}$$

The coarse aggregate content itself can be subdivided if single sized 10, 20 and 40 mm materials are to be combined. Again, the best proportions will depend on aggregate shape and concrete usage but the following ratios are suggested as a general guide:

1:2 for combination of 10 and 20 mm material
1:1.5:3 for combination of 10, 20 and 40 mm material.

(Source: Teychenné et al., 1997)

APPENDIX D

CHARACTERIZATION TESTS RESULTS

D1 Results of Physical Characterization Tests on Aggregates

Table D.1 Non-compacted Bulk Density of River Sand and Sandstone

Material	River Sand			Sandstone		
	A	B	C	A	B	C
Volume of Mould (cm^3)	4972.00	4972.00	4972.00	4972.00	4972.00	4972.00
Mass Of Empty Mould, W_{tool} (g)	6640.00	6640.00	6640.00	6640.00	6640.00	6640.00
Mass of Mould + Sample, W_1 (g)	14600.0	14690.0	14650.0	13700.0	13850.0	13830.0
Average Mass of Sample, W_1-W_{tool} (g)	8006.67			7153.33		
Bulk Density (Kg/m^3)	1610.35			1438.72		

Table D.2 Compacted Bulk Densities of River Sand and Sandstone

Material	River Sand			Sandstone		
	A	B	C	A	B	C
Volume of Mould (cm^3)	4972.00	4972.00	4972.00	4972.00	4972.00	4972.00
Mass of Empty Mould, W_{tool} (g)	6640.00	6640.00	6640.00	6640.00	6640.00	6640.00
Mass Of Mould + Sample, W_1 (g)	15350.0	15290.0	15320.0	15280.0	15275.0	15265.0
Average Mass of Sample (g)	8680.00			8633.00		
Bulk Density (Kg/m^3)	1745.78			1736.39		

Table D.3 Water Absorption of Sandstone

Sandstone	Sample		
	A	B	C
Mass Before Test (g)	5000.00	5000.00	5000.00
Mass After Test (g)	5095.00	510.00	1050.00
Mass of Absorbed Water (g)	95.00	110.00	105.00
Water Absorption (%)	1.90	2.20	2.10
Mean Water Absorption = 2.06%			

Table D.4 Void Ratio of River Sand and Sandstone

Material	River Sand	Sandstone
Specific Gravity of Sample	2.618	2.467
Relative Density of Sample	1.610	1.439
Void Ratio	0.385	0.418

Table D.5 Specific Gravity of River Sand and Sandstone

Material	River Sand			Sandstone		
Bottle Number	A	B	C	A	B	C
Mass of bottle + sample + water, W_3 (g)	1580	1579	1579.5	1575	1573	1573
Mass of bottle + sample, W_2 (g)	770	770	770	770	770	770
Mass of bottle full of water only, W_4 (g)	1425	1425	1425	1425	1425	1425
Mass of bottle, W_1 (g)	520.0	520.0	520.0	520.0	520.0	520.0
Mass of water used = W_3-W_2 (g)	810.0	809.0	809.5	805.0	803.0	803.0
Mass of sample used = W_2-W_1 (g)	250	250	250	250	250	250
Volume of sample = (W_4-W_1) - (W_3-W_2) ml	95.0	96.0	95.5	100	102	102
Specific Gravity of Sample $= \frac{(W2 - W1)}{(W4 - W1) - (W3 - W2)}$	2.632	2.604	2.618	2.500	2.451	2.451
Average Specific Gravity	2.618			2.467		

Table D.6 Grain Size Distribution Analysis of River Sand

BS Sieve Size	Mass Retained (g)	Cumulative Mass Retained (g)	Cumulative % Mass Retained	Cumulative % Passing
10mm	0	0	0	100
6.7mm	42	42	4.2	95.8
4.75mm	37	79	7.9	92.1
2.36mm	61	140	14.0	86.0
1.18mm	98	238	23.8	76.2
600µ	216	454	45.4	54.6
425µ	169	623	62.3	37.7
300µ	187	810	81.0	19
212µ	127	937	93.7	6.3
150µ	30	967	96.7	3.3
75µ	18	985	98.5	1.5
Pan	15	-	-	-
Total	1000		527.5	

Table D.7 Grain Size Distribution Analysis of Sandstone

BS Sieve Size	Mass Retained (g)	Cumulative Mass Retained (g)	Cumulative % Mass Retained	Cumulative % Passing
50mm	0	0	0	100
37.5mm	481	481	9.62	90.38
25mm	1613	2094	41.88	58.12
19mm	1495	3589	71.78	28.22
13.5mm	824	4413	88.26	11.74
9.5mm	283	4696	93.92	6.08
6.7mm	94	4790	95.80	4.2
4.75mm	136	4926	98.52	1.48
2.36mm	36	4962	99.24	0.76
1.18mm	18	4980	99.60	0.4
600µ	7	4987	99.74	0.26
Pan	13	-	-	-
TOTAL	5000		798.36	

Table D.8 Coefficient of Uniformity and Fineness Modulus of Aggregates

Tested Samples	D10	D30	D60	Cu = $\frac{D_{60}}{D_{10}}$	Cc = $\frac{{D_{30}}^2}{D_{60} * D_{10}}$	Fineness Modulus
River Sand	0.236	0.370	0.750	3.18	0.77	5.28
Sandstone	12.3	20.2	26.5	2.15	1.25	7.98

D2 Results of Mechanical Characterization Tests on Sandstone

Table C.9 Impact Value Test Result of Sandstone (Size 19mm – 25mm)

Sandstone	Sample		
	A	B	C
Initial mass of sample, W_1 (g)	350.0	350.0	350.0
Mass of sample after test, passing 2.36mm sieve, W_2 (g)	53.8	47.5	59.5
Mass retained (g)	296.2	302.5	290.5
Percentage mass retained (%)	84.63	86.42	83.00
Impact Value $= \frac{W2}{W1} * 100$	15.37	13.57	17.00
Average Impact Value (%)	15.31		

Table D.10 Abrasion Value Test Results of SSD[1] Sandstone

Sieve Size (mm)	Mass Passing (g)	Mass Retained (G)	Mass Bigger Than 2.00mm (g)
13.2	500	0	-
9.50	247	253	253
2.00	76	171	424
RECEIVER	0	76	-
LA Abrasion Number = $\frac{(mass\ smaller\ than\ 2.00)}{(mass\ bigger\ than\ 2.00)}$ x 100 = $\frac{76}{424}$ x 100 = 17.92%			

Table D.11 Attrition Value Test Results of SSD[1] Sandstone

Sieve Size (mm)	Mass Passing (g)	Mass Retained (g)	Mass Bigger Than 2.00mm (g)
13.20	350	0	-
9.50	75	275	275
2.00	36	39	314
Receiver	0	36	-
LA Attrition Number = $\frac{(mass\ smaller\ than\ 2.00)}{(mass\ bigger\ than\ 2.00)}$ x 100 = $\frac{36}{314}$ x 100 = 11.46%			

1. SSD – Saturated Surface Dry

APPENDIX E

EMPIRICAL MODELS

E1. Model Formulation

The method employed involved working out aggregate-cement, water-cement content, and the relative proportion of aggregates. The saturated surface dry density (SSD) and compressive strength from experiment are combined graphically to get the trending lines and equations of trending lines which gives the empirical equations. The equations were used to get model results for compressive strength and SSD densities for the mix ratios. The results of this method will are shown in chapter four.

A total number of 25 Empirical models were developed from the relationship between mix ratio (water-cement), SSD density and compressive strength plotted in form of graphs. The equations of the trending lines were used to derive the model compressive strength and SSD density values as show in Figures E.1 to E.25.

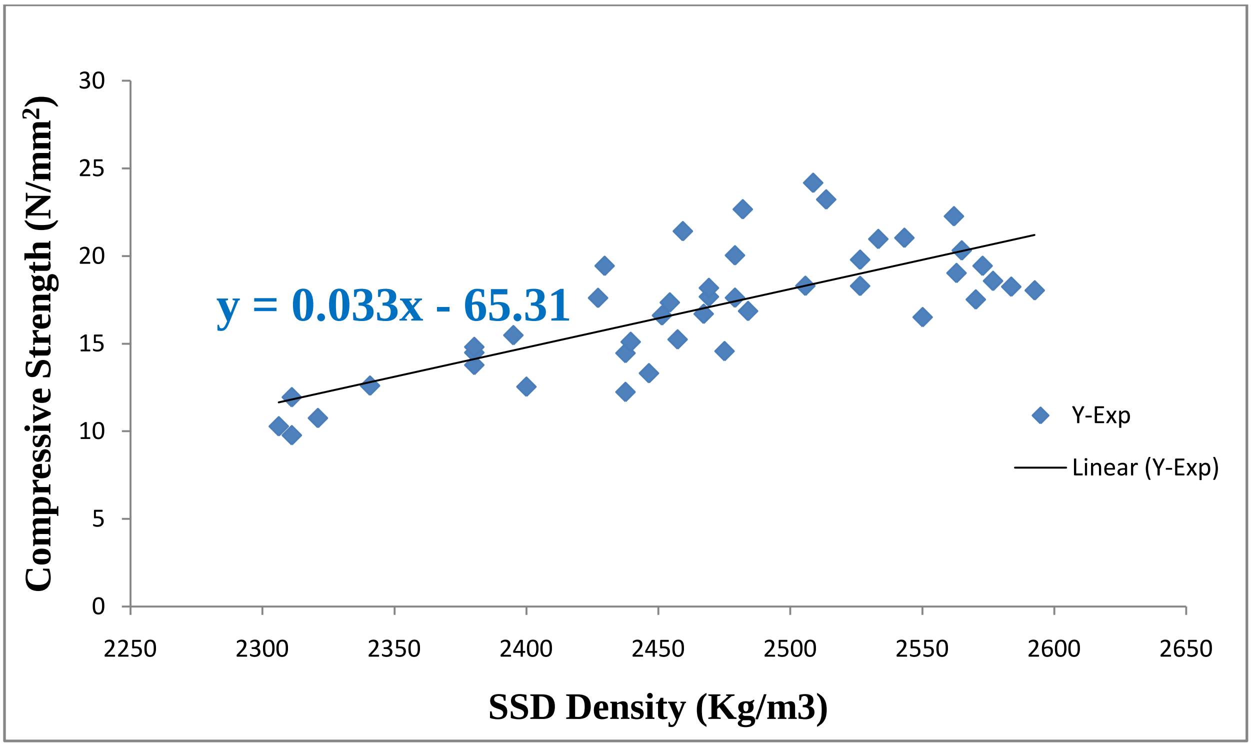


Figure E.1 SSD* Density verses Compressive Strength

* Saturated Surface Dry Density

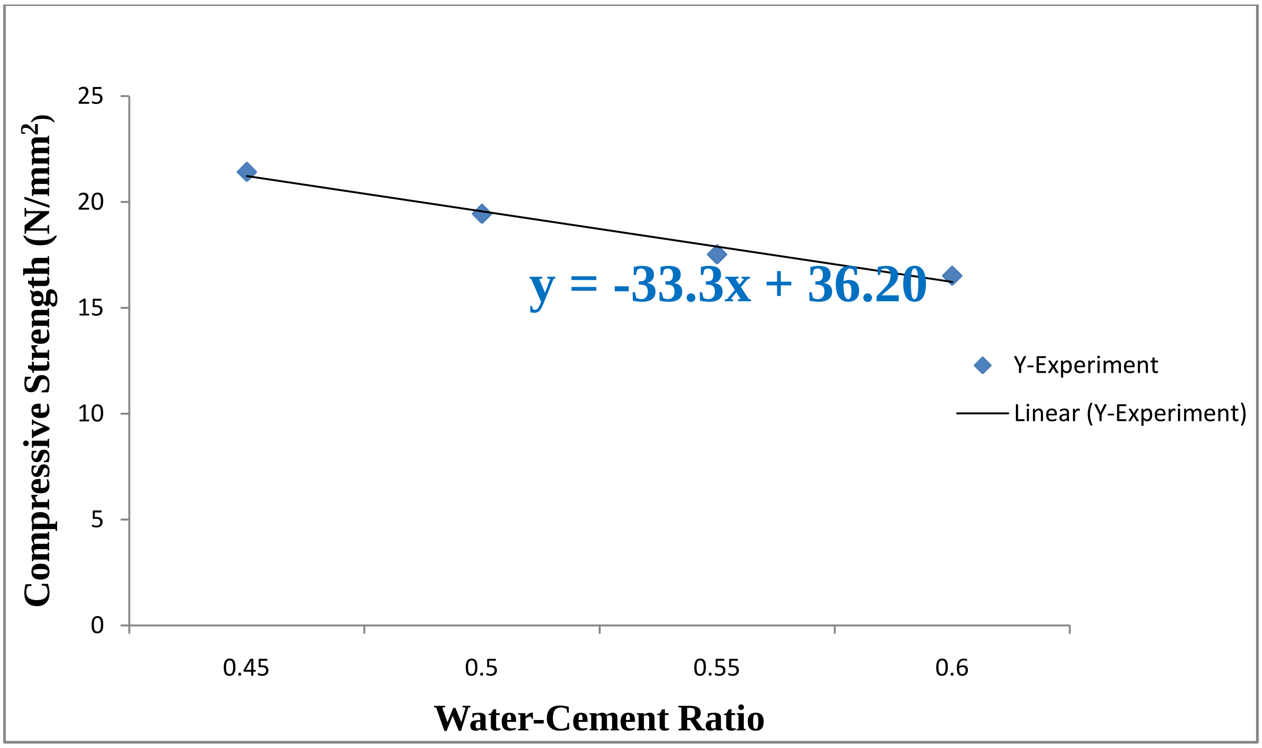


Figure E.2 Compressive Strength verses *W/C for 1:1.5:4.5

*Water-cement ratio

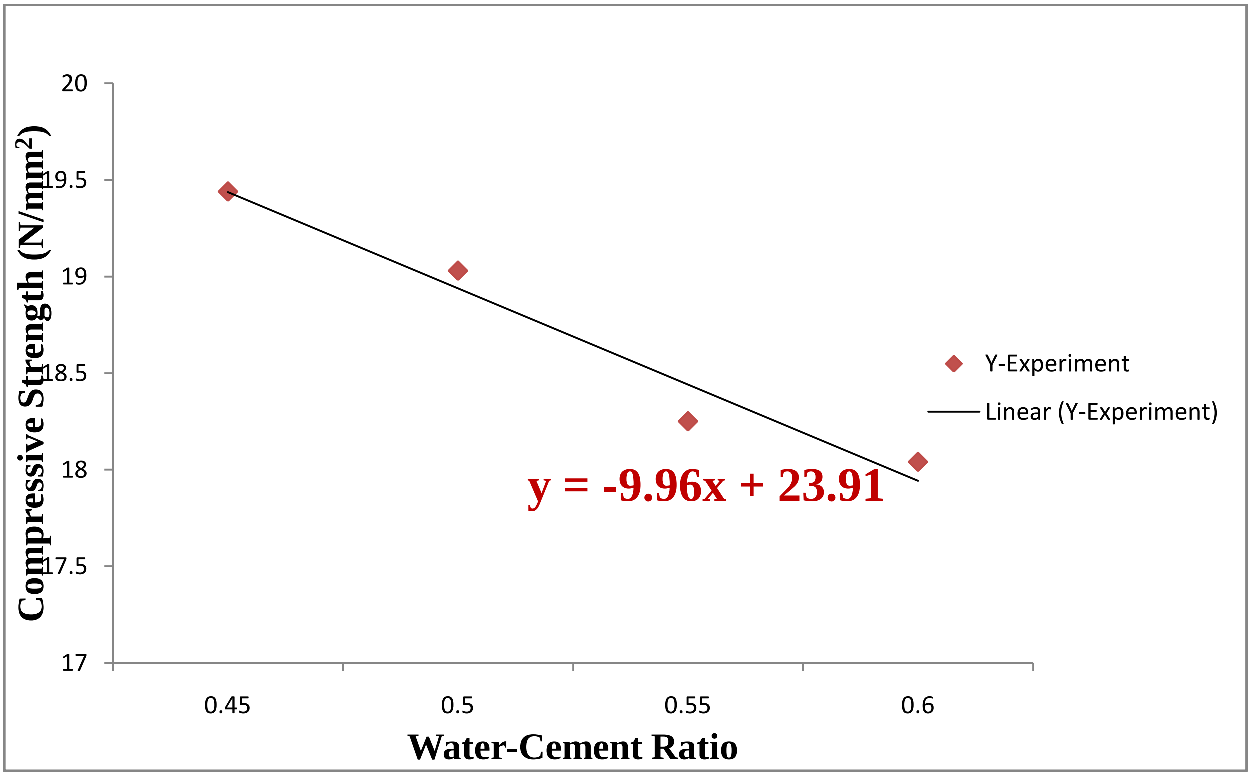


Figure E.3 Compressive Strength verses W/C for 1:1.75:4.25

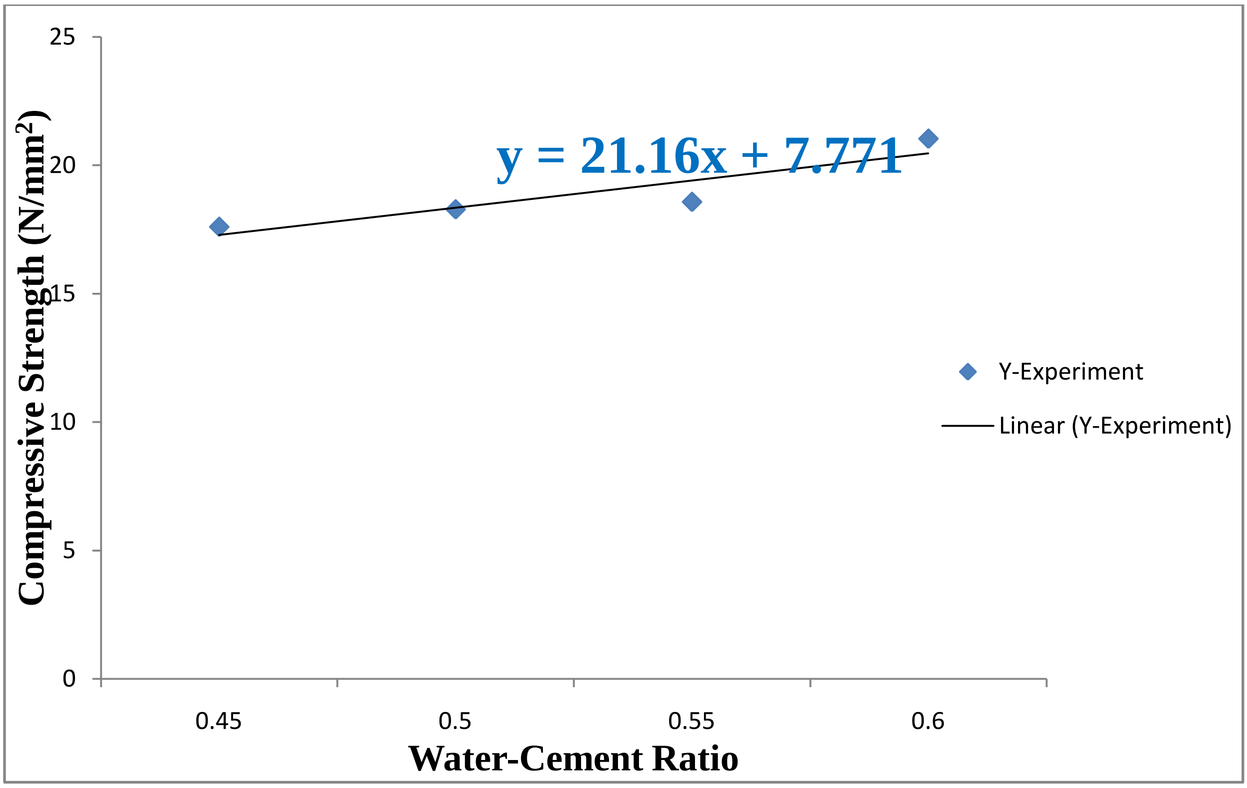


Figure E.4 Compressive Strength verses W/C for 1:2:4

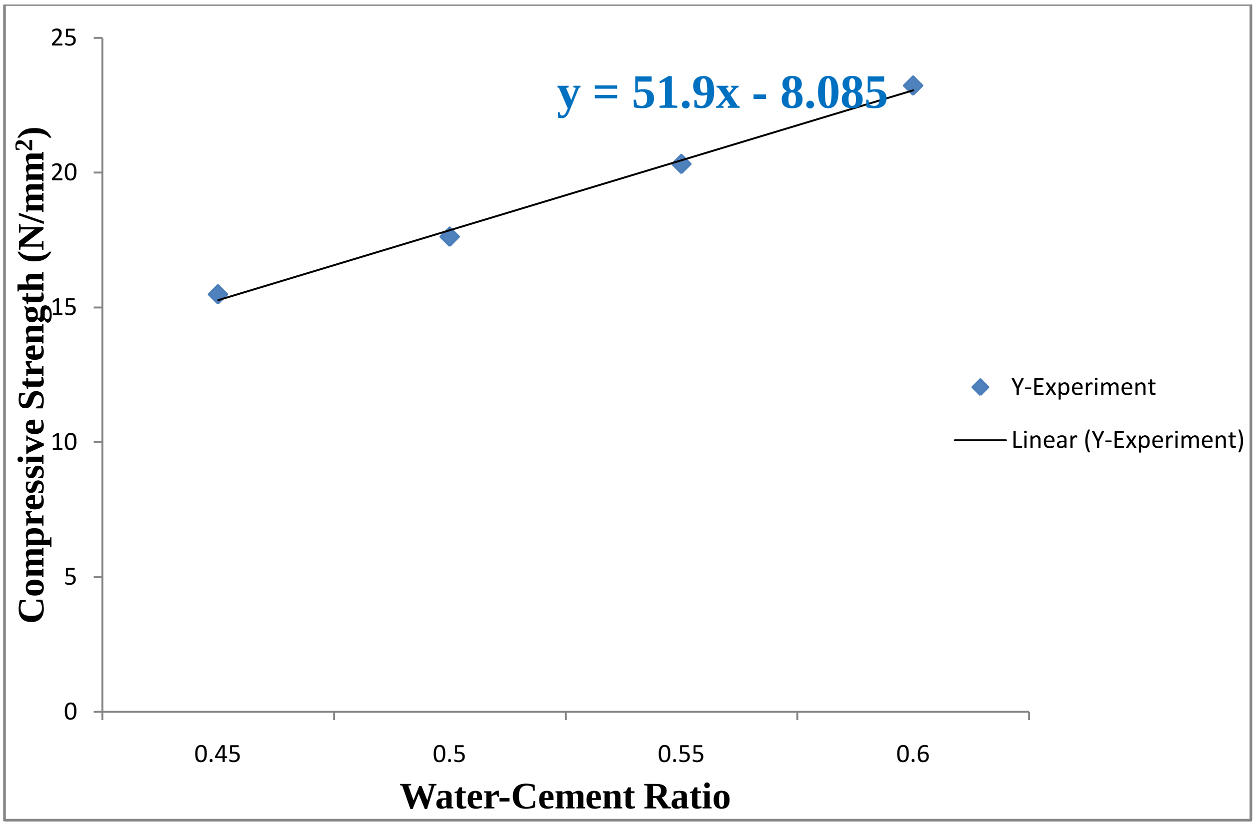


Figure E.5 Compressive Strength verses W/C for 1:2.25:3.75

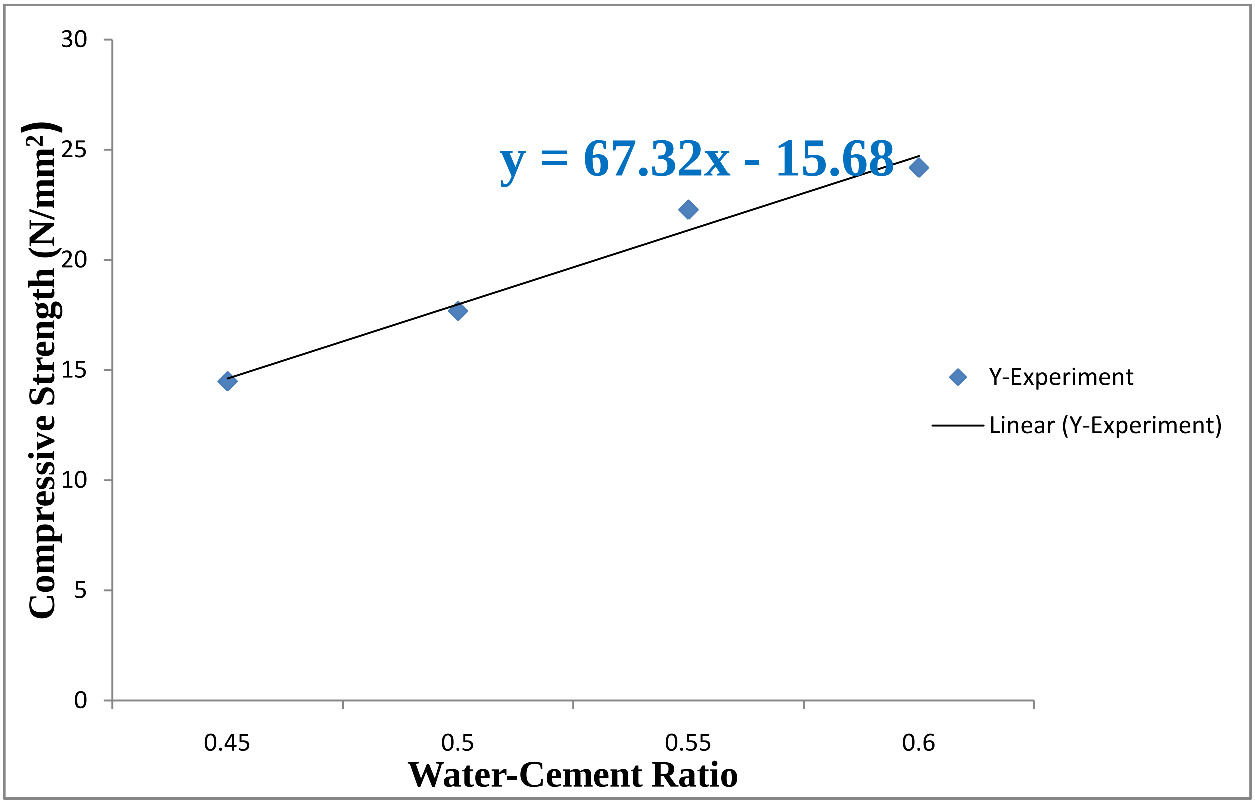


Figure E.6 Compressive Strength verses W/C for 1:2.5:3.5

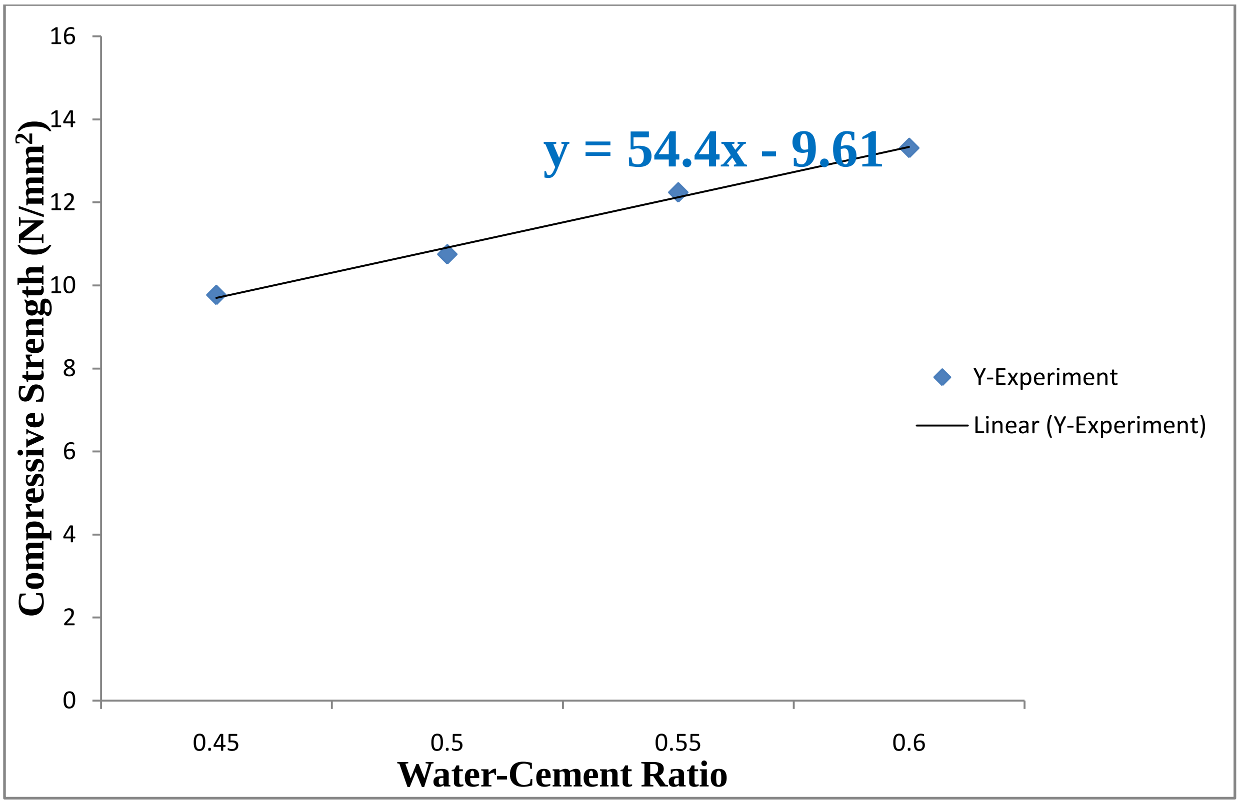


Figure E.7 Compressive Strength verses W/C for 1:2.75:3.25

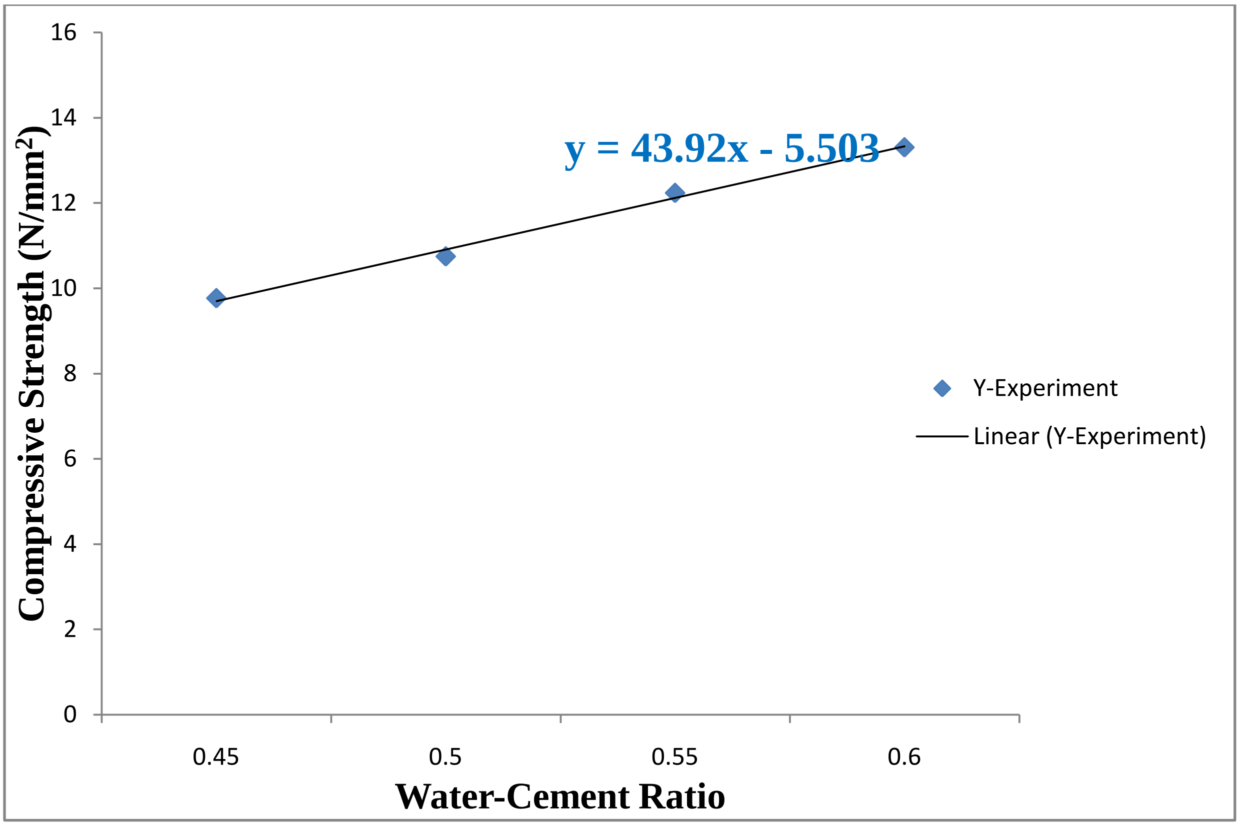


Figure E.8 Compressive Strength verses W/C for 1:3:3

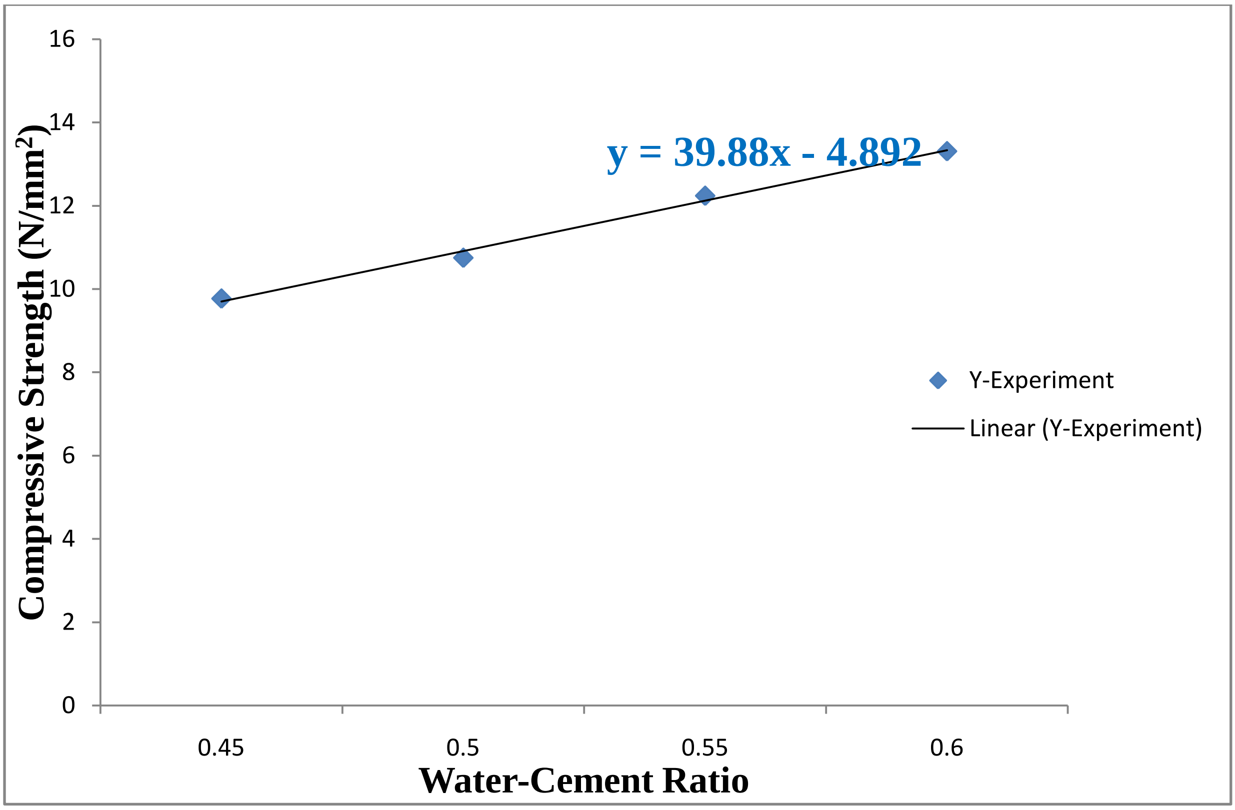


Figure E.9 Compressive Strength verses W/C for 1:3.25:2.75

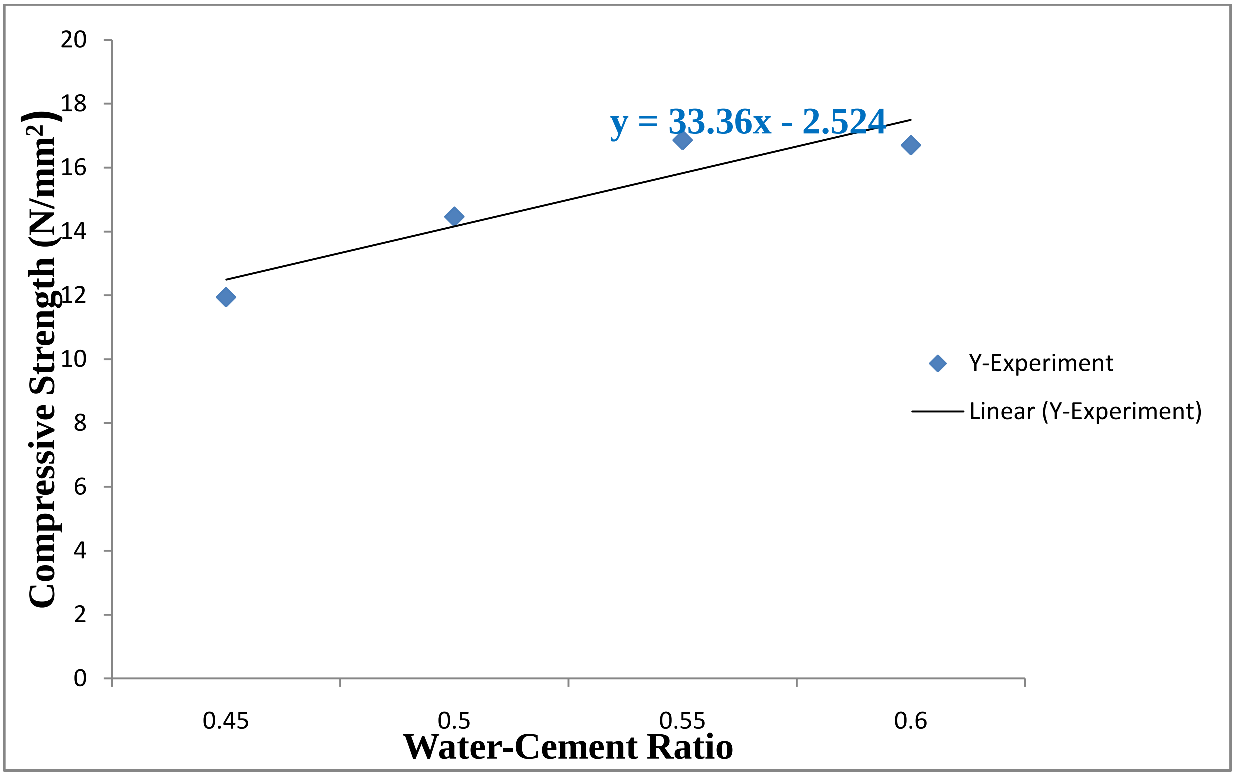


Figure E.10 Compressive Strength verses W/C for 1:3.5:2.5

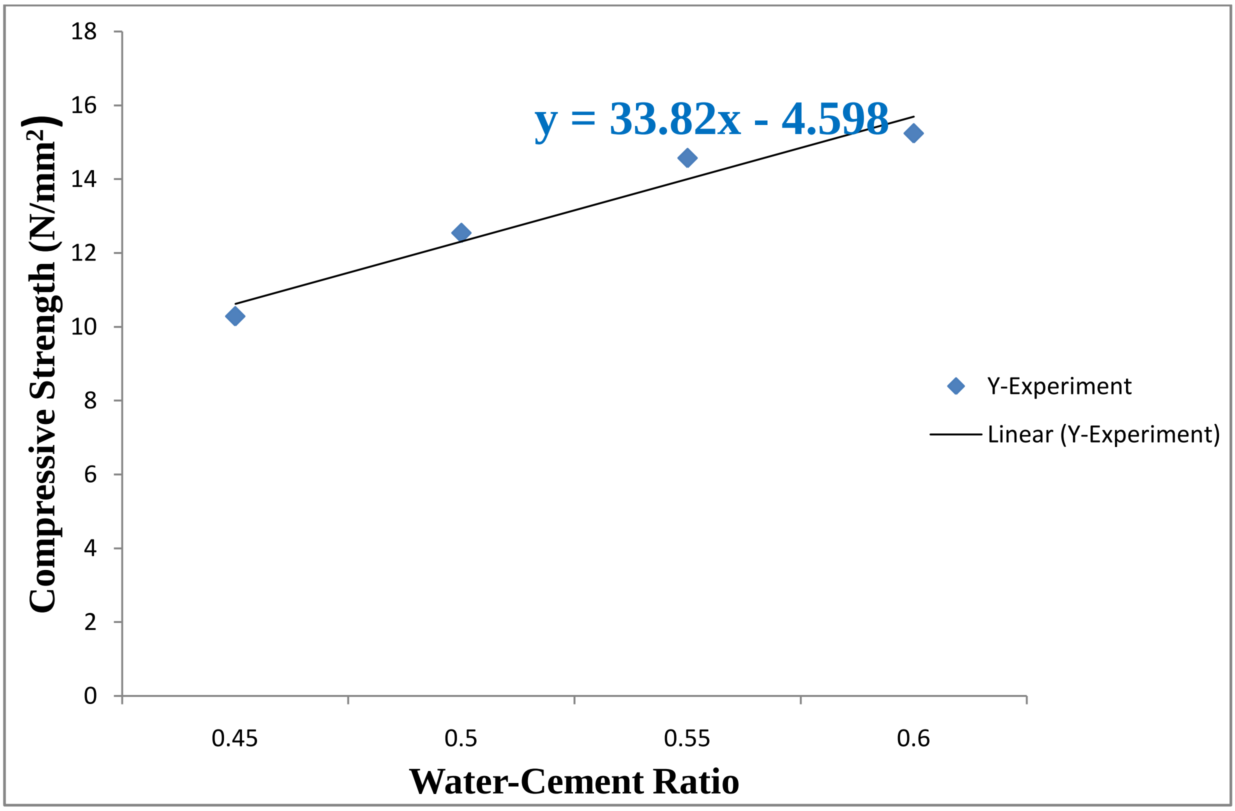


Figure E.11 Compressive Strength verses W/C for 1:3.75:2.25

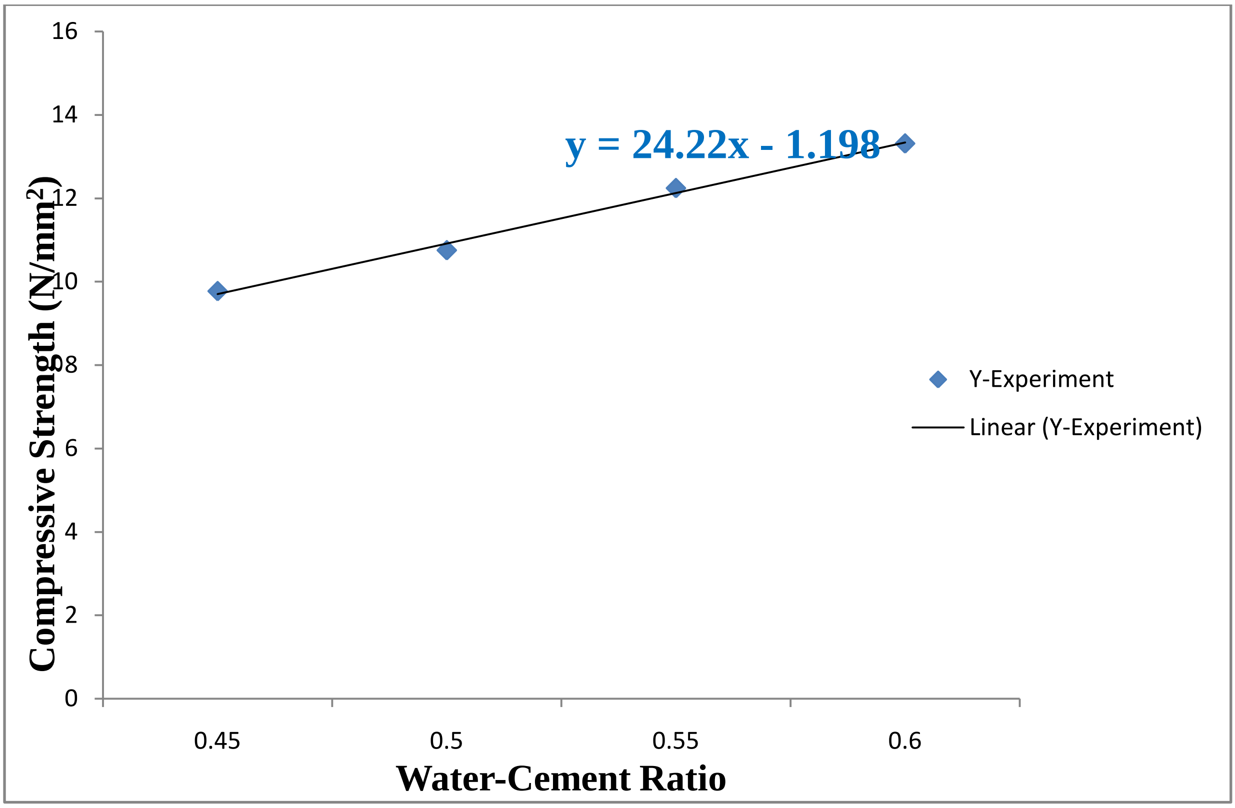


Figure E.12 Compressive Strength verses W/C for 1:4:2

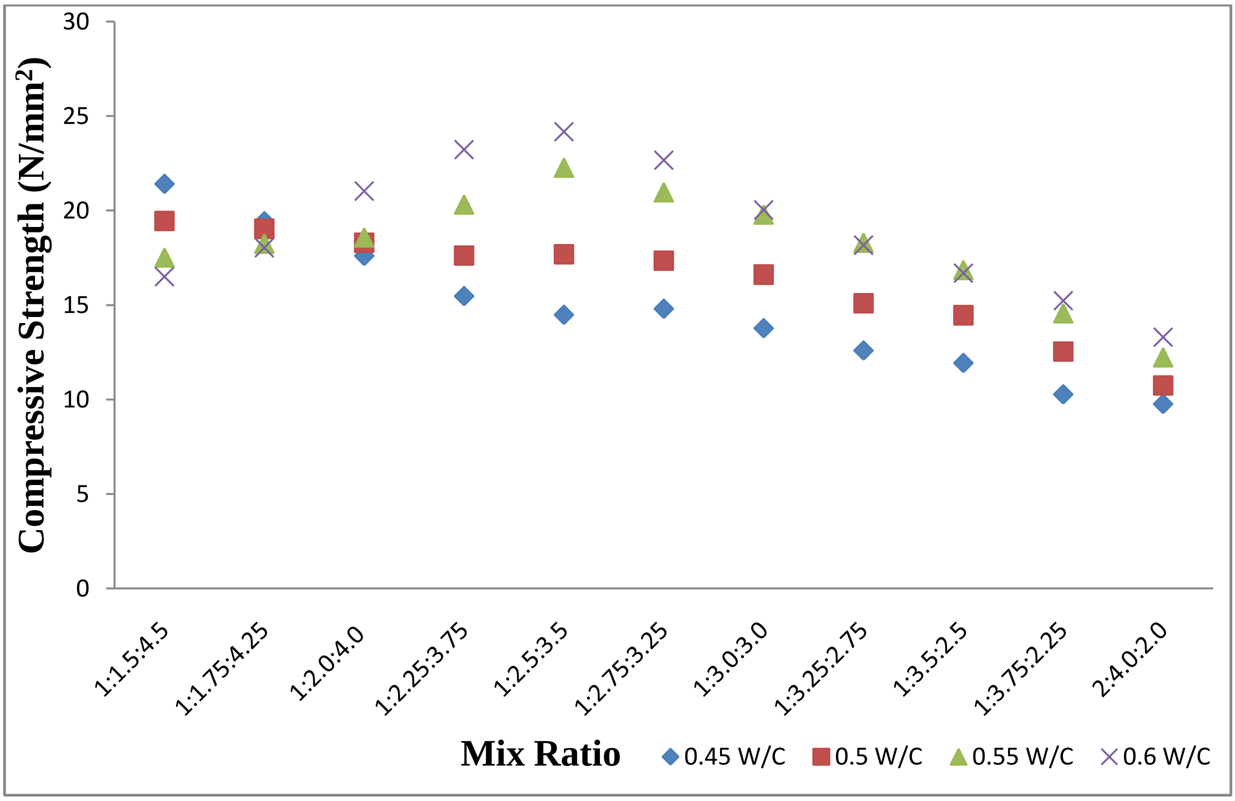


Figure E.13 Mix Ratio verses Compressive Strength for W/C 0.45, 0.5, 0.55, 0.6

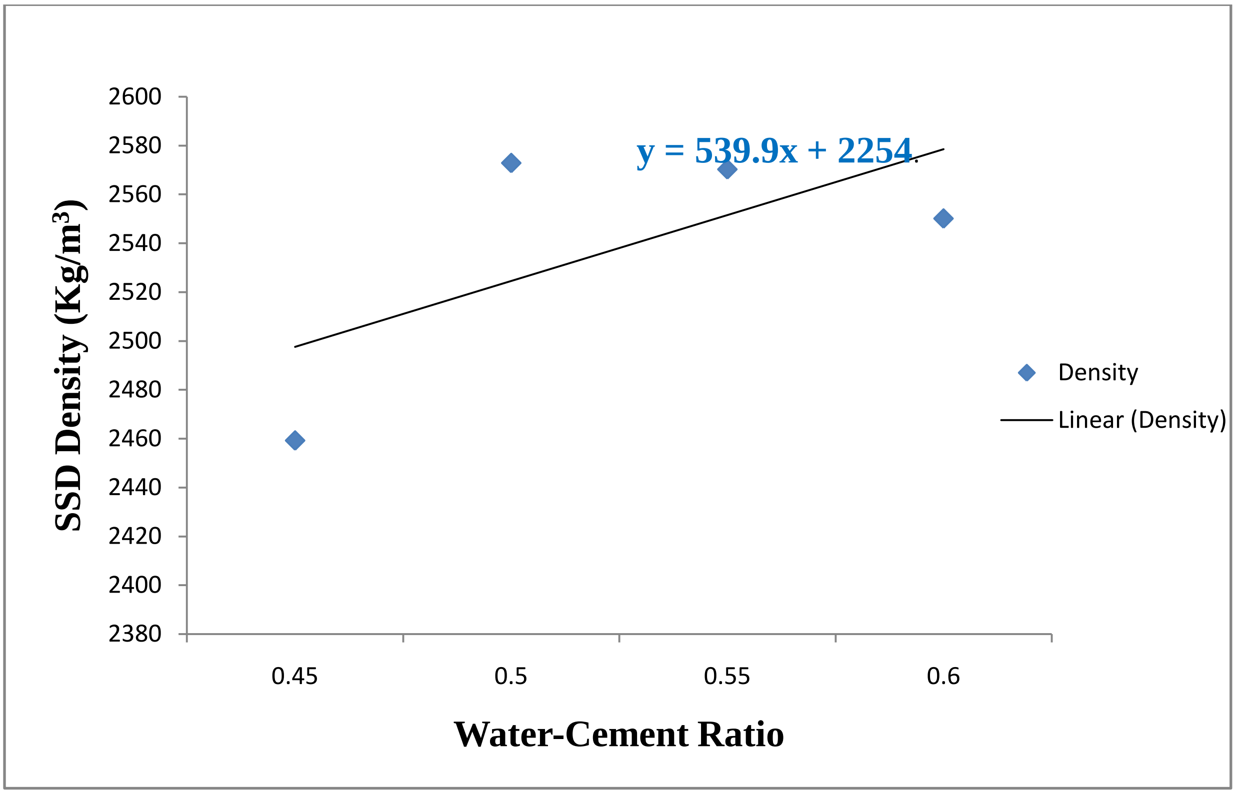


Figure E.14 Water-Cement Ratio verses SSD Density for 1:1.5:4.5

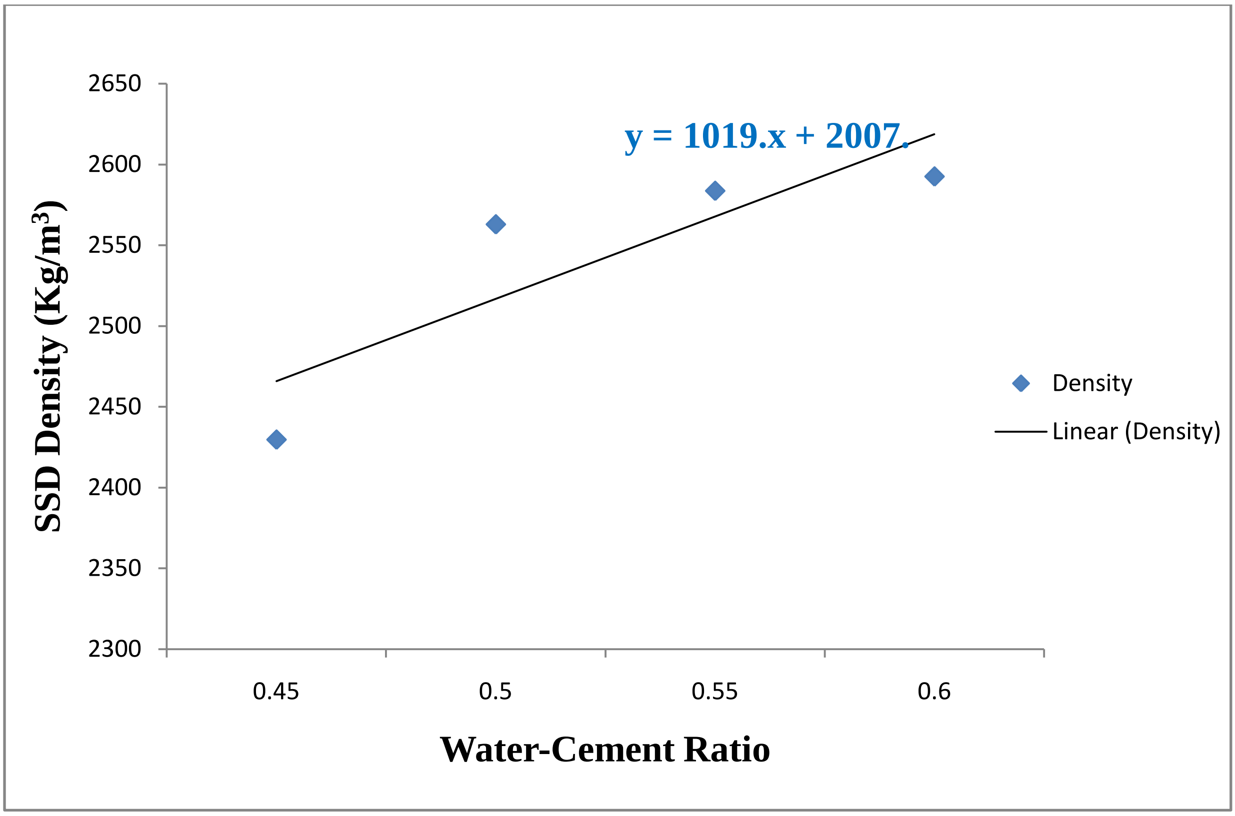


Figure E.15 Water-Cement Ratio verses SSD Density for 1:1.75:4.25

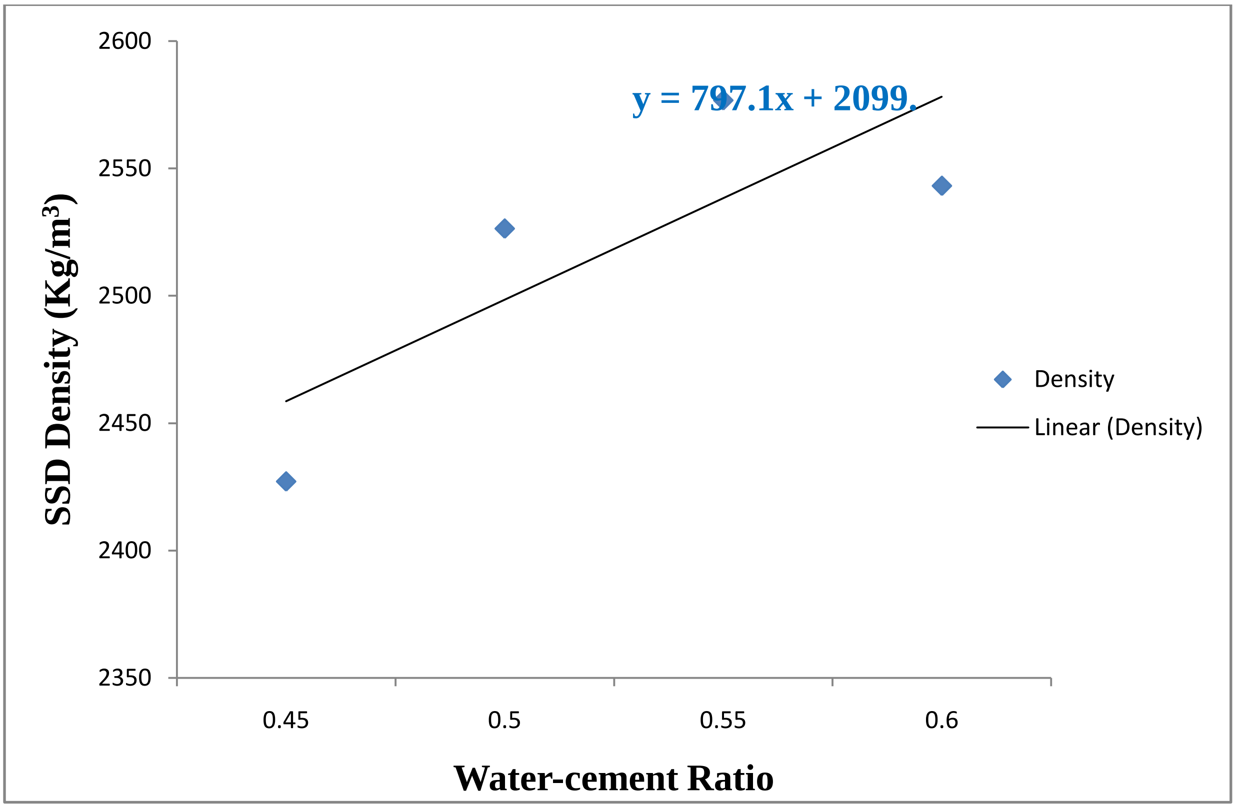


Figure E.16 Water-Cement Ratio verses SSD Density for 1:2:4

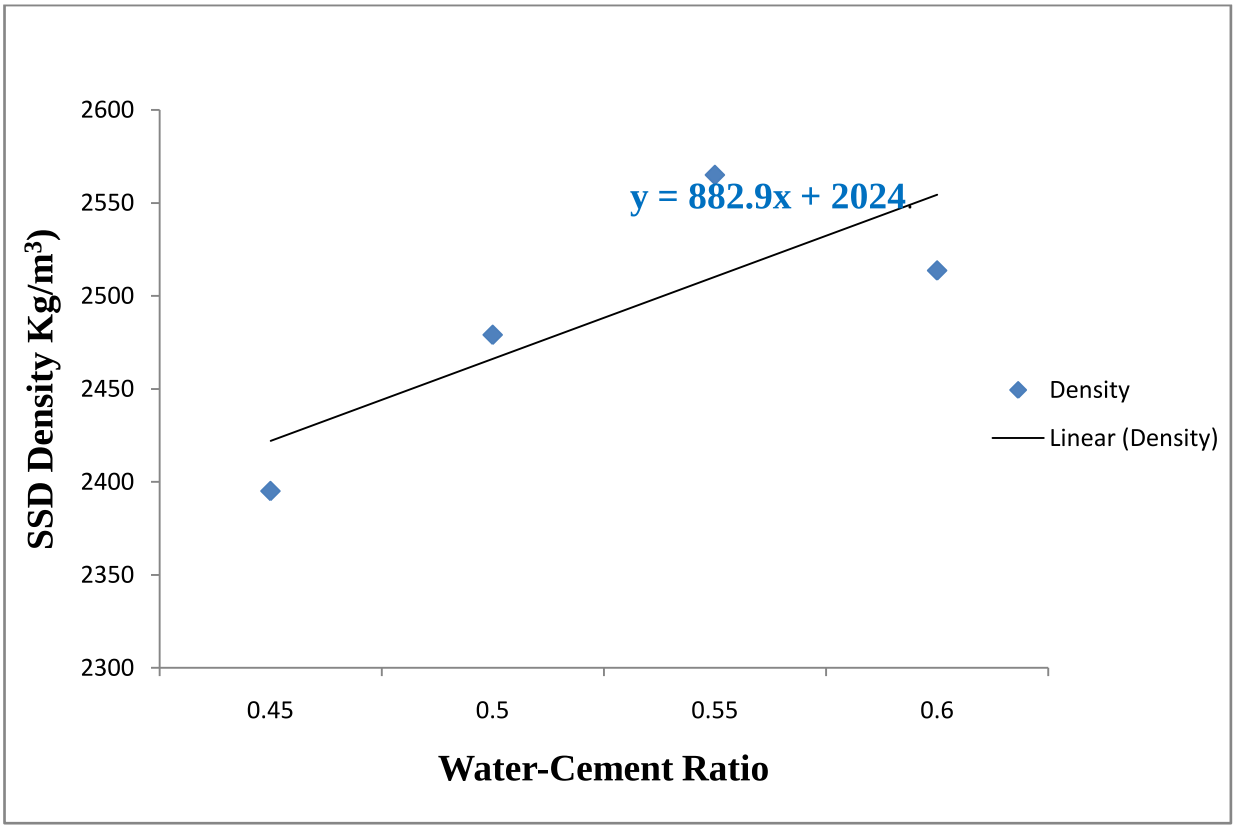


Figure E.17 Water-Cement Ratio verses SSD Density for 1:2.25:3.75

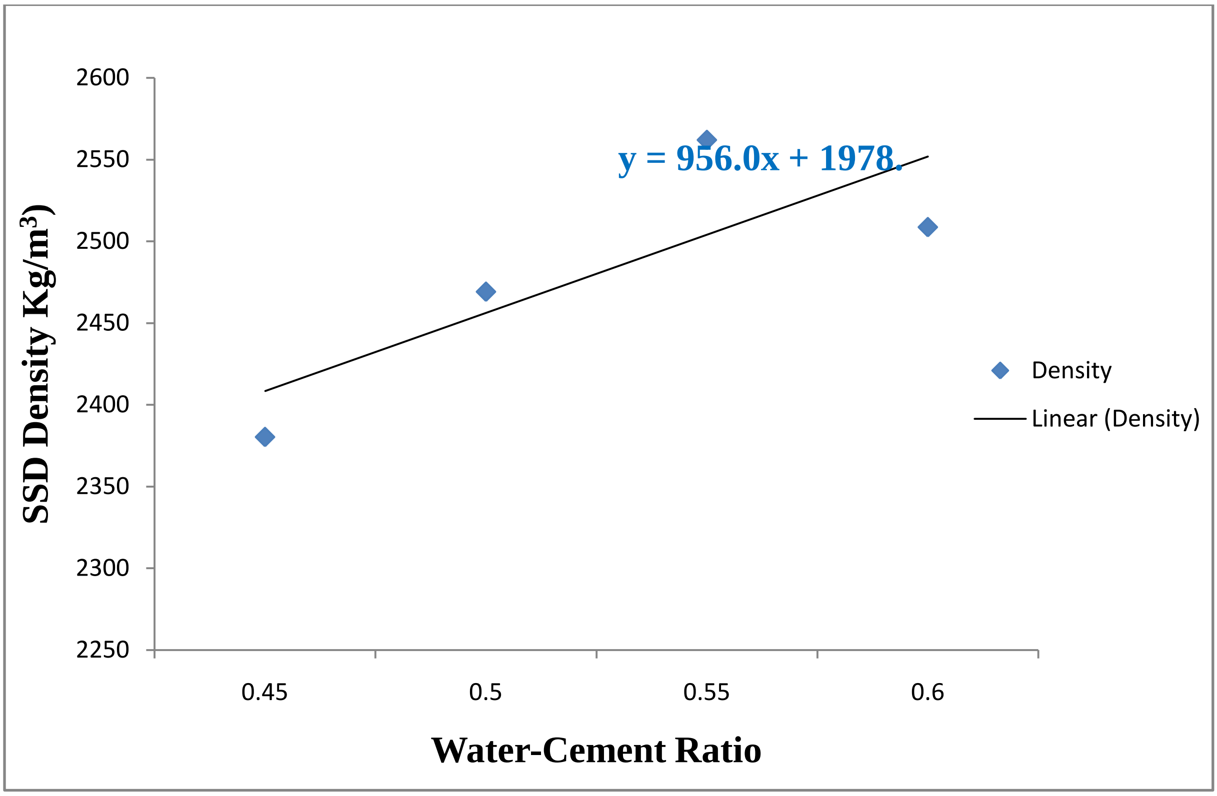


Figure E.18 Water-Cement Ratio verses SSD Density for 1:2.5:3.5

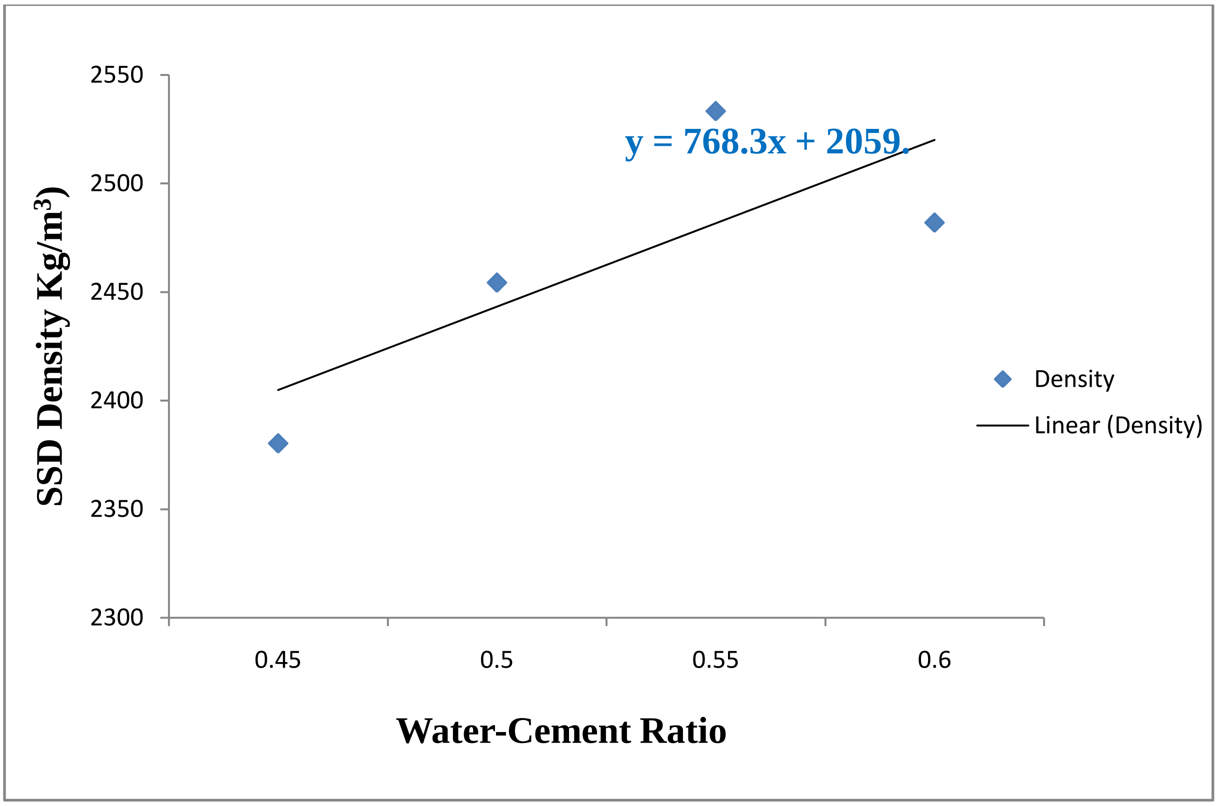


Figure E.19 Water-Cement Ratio verses SSD Density for 1:2.75:3.25

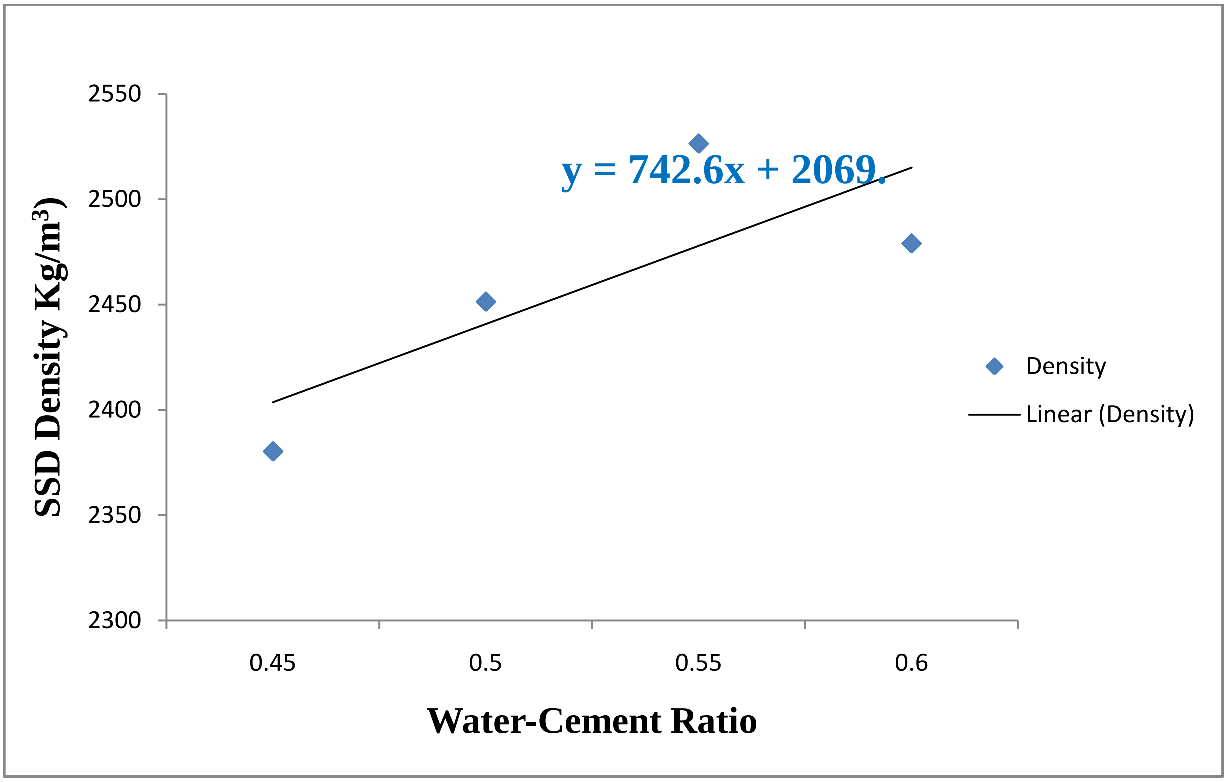


Figure E.20 Water-Cement Ratio verses SSD Density for 1:3:3

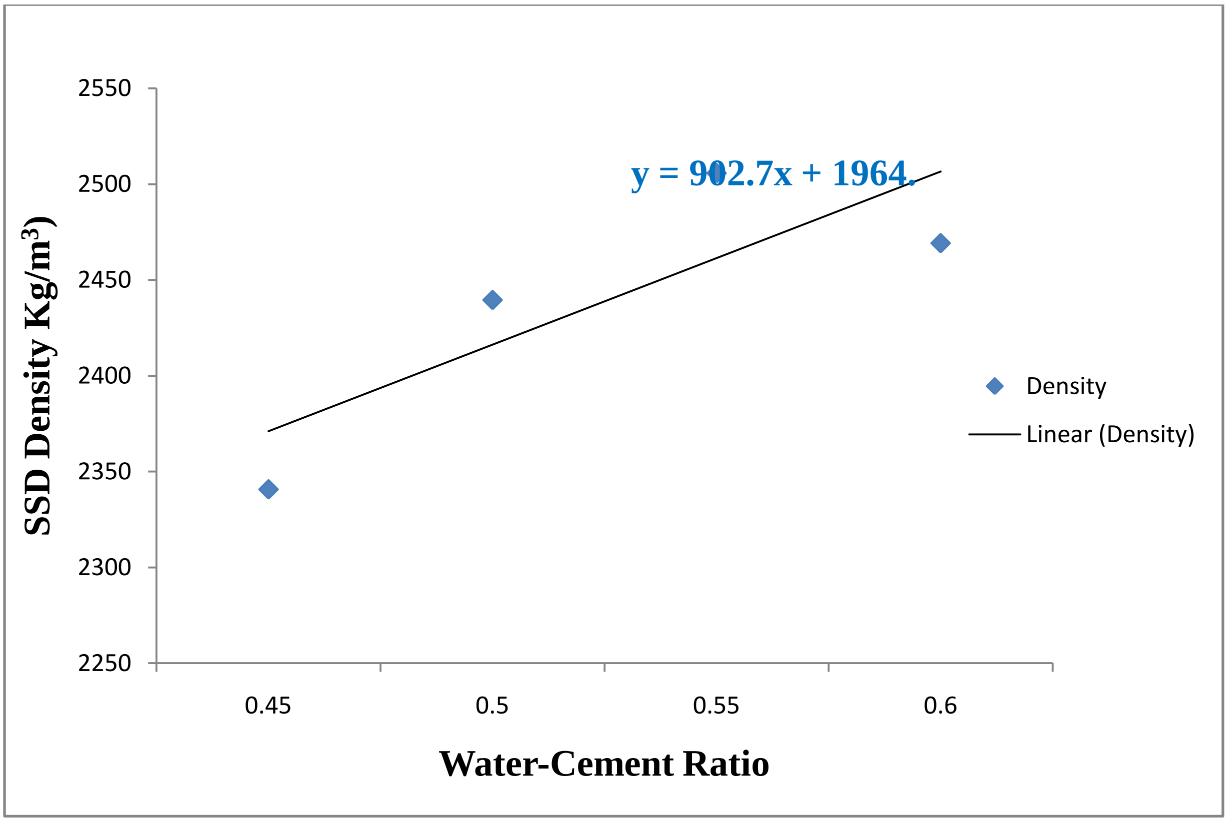


Figure E.21 Water-Cement Ratio verses SSD Density for 1:3.25:2.75

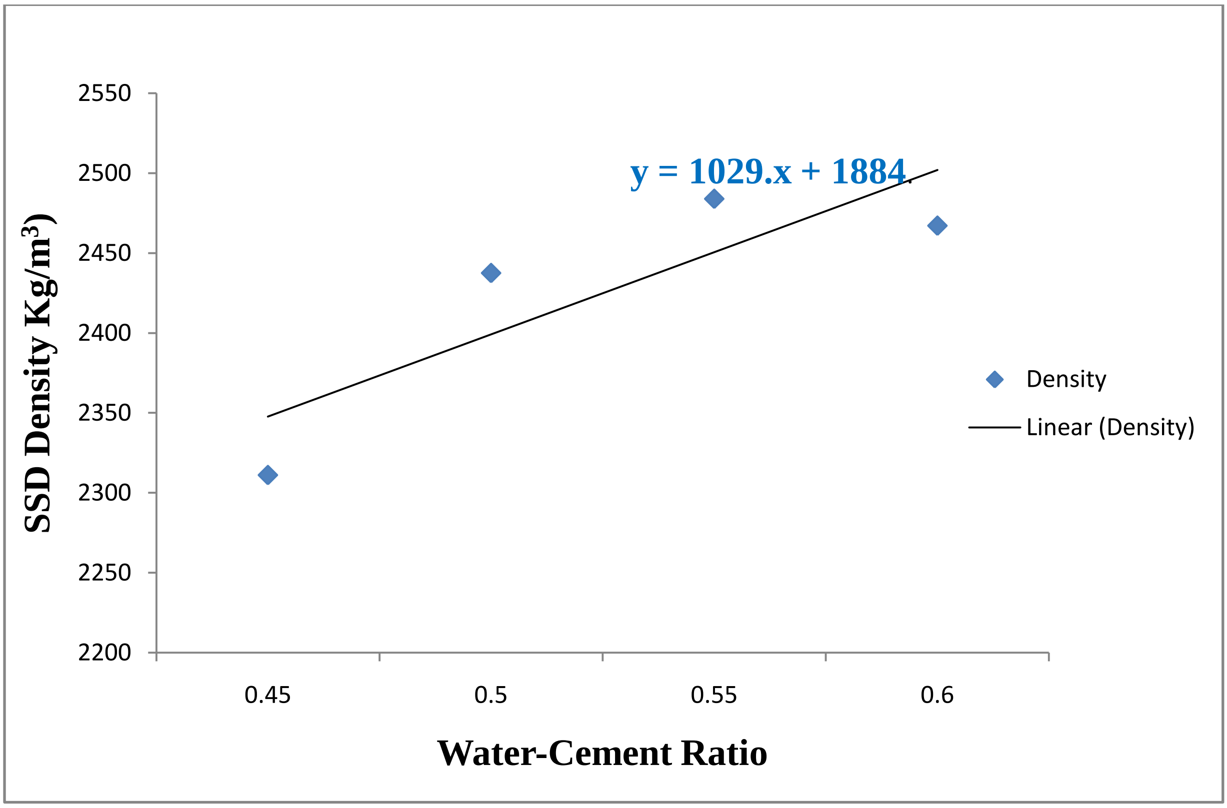


Figure E.22 Water-Cement Ratio verses SSD Density for 1:3.5:2.5

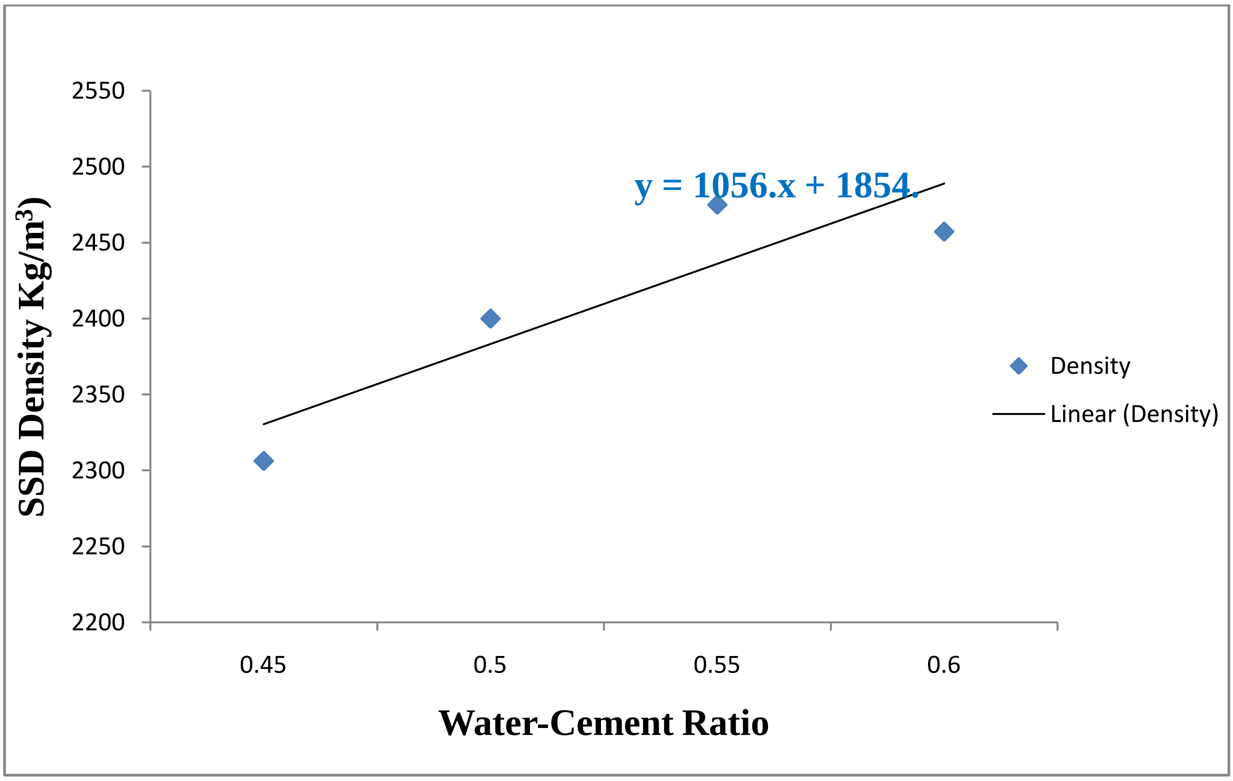


Figure E.23 Water-Cement Ratio verses SSD Density for 1:3.75:2.25

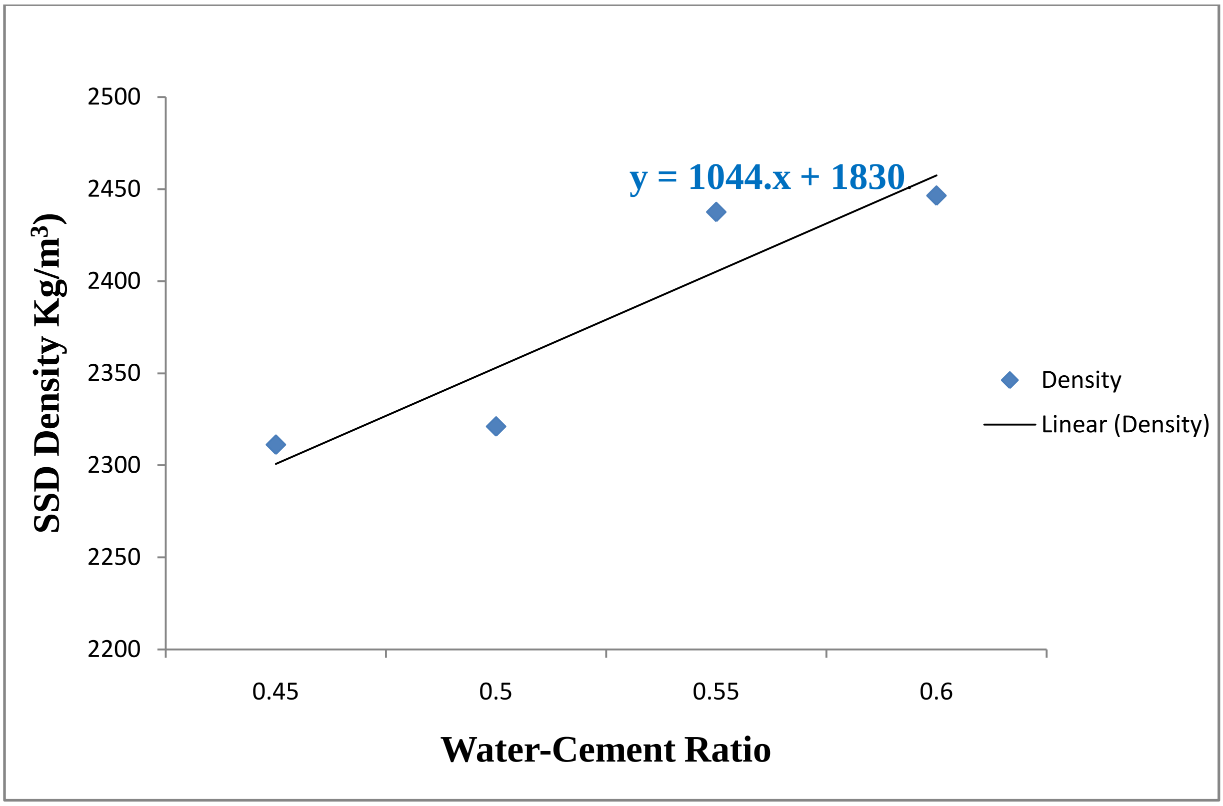


Figure E.24 Water-Cement Ratio verses SSD Density for 1:4:2

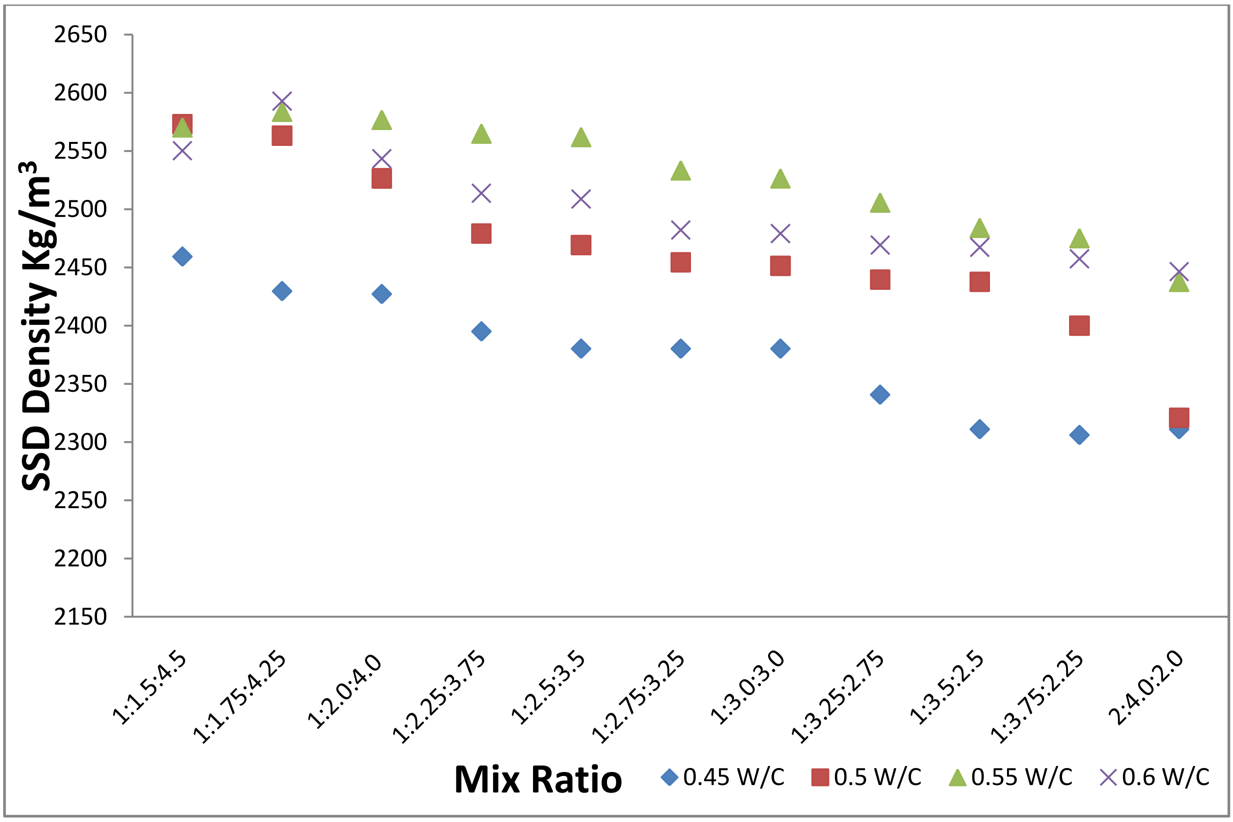


Figure E.25 Mix Ratio verses SSD Density for W/C 0.45, 0.5, 0.55, 0.6

E2. Results Empirical Models

Table E.1 show the SSD density, the experimental compressive strength results and the empirical model compressive strength values.

Table E.2 shows the mix ratio, the experimental compressive strength and the empirical model compressive strength. The table shows that the empirical model developed herein could predict the compressive strength or the SSD density of sandstone concrete when the water-cement ratio is known and vice versa.

Table E.3 show the mix ratio, the experimental SSD density and the empirical model SSD density.

Table E.1 Density, Experimental and Model Compressive Strength

Mix Ratio	Density(Kg/m^3)	Compressive Strength (N/mm^2)	
		Experiment	Empirical Model
0.45:1:3.75:2.25	2306.17	10.28	10.79361
0.45:1:4:2	2311.11	9.77	10.95663
0.45:1:3.5:2.5	2311.11	11.94	10.95663
0.5:1:4:2	2321	10.75	11.2830
0.45:1:3.25:2.75	2340.74	12.6	11.93442
0.45:13:3	2380.25	13.78	13.23825
0.45:1:2.5:3.5	2380.25	14.49	13.23825
0.45:1:2.75:3.25	2380.25	14.81	13.23825
0.45:1:2.25:3.75	2395.06	15.48	13.72698
0.5:1:3.75:2.25	2400	12.54	13.8900
0.45:1:2:4	2427.14	17.61	14.78562
0.45:1:1.75:4.25	2429.63	19.44	14.86779
0.6:1:4:2	2437.53	12.24	15.12849
0.5:1:3.5:2.5	2437.53	14.46	15.12849
0.5:1:3.25:2.75	2439.51	15.1	15.19383
0.6:1:4:2	2446.42	13.31	15.42186
0.5:1:3:3	2451.36	16.61	15.58488
0.5:1:2.75:3.25	2454.32	17.35	15.68256
0.6:1:3.75:2.25	2457.28	15.24	15.78024
0.45:1:1.5:4.5	2459.26	21.42	15.84558
0.6:1:3.5:2.5	2467.16	16.7	16.10628
0.5:1:2.5:3.5	2469.13	17.68	16.17129
0.6:1:3.25:2.75	2469.14	18.18	16.17162
0.55:1:3.75:2.25	2475.06	14.57	16.36698
0.5:1:2.25:3.75	2479.01	17.62	16.49733
0.6:1:3:3	2479.01	20.04	16.49733
0.6:1:2.75:3.25	2481.97	22.67	16.59501
0.55:1:3.5:2.5	2483.97	16.86	16.66101
0.55:1:3.25:2.75	2505.68	18.3	17.37744
0.6:1:2.5:3.5	2508.64	24.18	17.47512
0.6:1:2.25:3.75	2513.58	23.23	17.63814
0.5:1:2:4	2526.42	18.29	18.06186
0.55:1:3:3	2526.42	19.79	18.06186
0.55:1:2.75:3.25	2533.33	20.97	18.28989
0.6:1:1:2:4	2543.21	21.04	18.61593
0.6:1:1.5:4.5	2550.12	16.51	18.84396
0.55:1:2.5:3.5	2561.98	22.27	19.23534
0.5:1:1.75:4.25	2562.96	19.03	19.26768
0.55:1:2.25:3.75	2564.94	20.32	19.33302
0.55:1:1.5:4.5	2570.25	17.52	19.50825
0.5:1:1.5:4.5	2572.84	19.44	19.59372
0.55:1:2:4	2576.79	18.58	19.72407
0.55:1:1.75:4.25	2583.7	18.25	19.9521
0.6:1:1.75:4.25	2592.59	18.04	20.24547

Table E.2 Mix Ratio, Experimental and Model Compressive Strength

W/C Ratio/Mix Ratio	Compressive Strength (N/mm^2)	
	Experiment	Empirical Model
Mix 1:1.5:4.5		
0.45	21.42	21.215
0.5	19.44	19.55
0.55	17.52	17.885
0.6	16.51	16.22
Mix 1:1.75:4.25		
0.45	19.44	19.428
0.5	19.03	18.93
0.55	18.25	18.432
0.6	18.04	17.934
Mix 1:2:4		
0.45	17.61	17.293
0.5	18.29	18.351
0.55	18.58	19.409
0.6	21.04	20.467
Mix 1:2.25:3.75		
0.45	15.48	15.27
0.5	17.62	17.865
0.55	20.32	20.46
0.6	23.23	23.055
Mix 1:2.5:3.5		
0.45	14.49	14.614
0.5	17.68	17.98
0.55	22.27	21.346
0.6	24.18	24.712
Mix 1:2.75:3.25		
0.45	14.81	14.87
0.5	17.35	17.59
0.55	20.97	20.31
0.6	22.67	23.03
Mix 1:3:3		
0.45	13.78	14.261
0.5	16.61	16.457
0.55	19.79	18.653
0.6	20.04	20.849

Mix 1:3.25:2.75		
0.45	12.6	13.054
0.5	15.1	15.048
0.55	18.3	17.042
0.6	18.18	19.036
Mix 1:3.5:2.5		
0.45	11.94	12.488
0.5	14.46	14.156
0.55	16.86	15.824
0.6	16.7	17.492
Mix 1:3.75:2.25		
0.45	10.28	10.621
0.5	12.54	12.312
0.55	14.57	14.003
0.6	15.24	15.694
Mix 1:4:2		
0.45	9.77	9.701
0.5	10.75	10.912
0.55	12.24	12.123
0.6	13.31	13.334

Table E.3 Mix Ratio, Experimental and Model SSD Density

W/C Ratio/Mix Ratio	SSD Density (Kg/m^3)	
	Experiment	Empirical Model
Mix 1:1.5:4.5		
0.45	2459.26	2496.955
0.5	2572.84	2523.95
0.55	2570.25	2550.945
0.6	2550.12	2577.94
Mix 1:1.75:4.25		
0.45	2429.63	2465.55
0.5	2562.96	2516.5
0.55	2583.7	2567.45
0.6	2592.59	2618.4
Mix 1:2:4		
0.45	2427.14	2457.695
0.5	2526.42	2497.55
0.55	2576.79	2537.405
0.6	2543.21	2577.26
Mix 1:2.25:3.75		
0.45	2395.06	2421.305
0.5	2479.01	2465.45
0.55	2564.94	2509.595
0.6	2513.58	2553.74
Mix 1:2.5:3.5		
0.45	2380.25	2408.2
0.5	2469.13	2456
0.55	2561.98	2503.8
0.6	2508.64	2551.6
Mix 1:2.75:3.25		
0.45	2380.25	2404.735
0.5	2454.32	2443.15
0.55	2533.33	2481.565
0.6	2481.97	2519.98
Mix 1:3:3		
0.45	2380.25	2403.17
0.5	2451.36	2440.3
0.55	2526.42	2477.43
0.6	2479.01	2514.56

Mix 1:3.25:2.75		
0.45	2340.74	2370.215
0.5	2439.51	2415.35
0.55	2505.68	2460.485
0.6	2469.14	2505.62
Mix 1:3.5:2.5		
0.45	2311.11	2347.05
0.5	2437.53	2398.5
0.55	2483.97	2449.95
0.6	2467.16	2501.4
Mix 1:3.75:2.25		
0.45	2306.17	2329.2
0.5	2400	2382
0.55	2475.06	2434.8
0.6	2457.28	2487.6
Mix 1:4:2		
0.45	2311.11	2299.8
0.5	2321	2352
0.55	2437.53	2404.2
0.6	2446.42	2456.4

E.3 Discussion of Empirical Model Results

The empirical models derived from the experimental results (Table 4.2 and 4.3) are presented in form of graphs. Figure 4.3 shows a graph of saturated surface dry (SSD) density against compressive strength, which produced equation of trending line.

Where;

Y = the compressive strength
X = the saturated surface dry density.

The optimum compressive strength was found to be 24.18N/mm^2. This corresponds to SSD density of 2508.64Kg/m^3.

Figures E.2 - E.12 show graphs of water-cement ratio versus compressive strength for solid materials mix ratios 1:1.5:4.5, 1:1.75:4.25, 1:2:4, 1:2.25:3.75, 1:2.5:3.5, 1:2.75:3.25, 1:3:3, 1:3.25:2.75, 1:3.5:2.5, 1:3.75:2.25 and 1:4:2 respectively. It can be observed that a high strength was achieved with w/c of 0.45 in the first two mix ratios. However, in the remaining mix ratios, it can be observed that the higher the w/c the higher the compressive strength. Eleven model equations were produced from the trending lines respectively.

Figure E.13 shows a graph of mix ratio verses compressive strength for W/C 0.45, 0.5, 0.55, 0.6 respectively. It can be observed for w/c 0.45 and 0.5 that the compressive strength increased as the sand-sandstone ratio decreased and also decreased as the sand-sandstone ratio increased. Mix ratios 1:1.5:4.5 produced concrete cubes of higher compressive strengths. However, a deviation can be observed for w/c 0.55 and 0.6. Here, the increase reached its optimum at mix ratio 1:2.5:3.5 after which the compressive strength started decreasing as a result of the increase in water-cement ratio from 0.45 to 0.6. The high water absorption capacity of sandstone and the increased cement-sand paste is responsible for this shift.

Figures E.14 – E.24 show graphs of water-cement ratio verses SSD density for the eleven mix ratios. The maximum SSD density was achieved with 0.6 w/c ratio. However, w/c of 0.55 produced higher values of SSD density. Eleven model equations were produced from the trending lines.

Figure E.25 shows graph of mix ratio verses SSD density of the concrete cubes for 0.45, 0.5, 0.55 and 0.6 w/c ratios. It can be observed that the SSD density increased with mixes with higher coarse aggregate (sandstone) content. That is, the higher the coarse aggregate content the higher the SSD density. However, comparing the trending line equations of the graphs show that SSD density increased with increase in w/c ratio, reaching its optimum at 0.55w/c and decreased with further increase in w/c ratio.