CIV5017Z: DISSERTATION FOR MASTER OF ENGINEERING DEGREE

A review on the efficiency of different supplementary cementitious materials as a partial replacement for Portland cement in concrete

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ABSTRACT

The effects of global warming and climate change are important and have attracted the attention of many researchers. Global warming is a result of the presence of increasing amounts of greenhouse gases in the atmosphere. Carbon dioxide, which is largely emitted into the atmosphere during the manufacture of cement clinker, is one of the greenhouse gases. Hence, researchers have explored the use of some waste materials and naturally occurring minerals as a partial replacement for cement in concrete. These materials are often referred to as supplementary cementitious materials (SCMs). Apart from the potential benefits of these SCMs for the properties of concrete, they also bring about a reduction in the amount of waste in landfill sites, as these wastes can cause land, water, and air pollution, thereby posing threats to human health. However, despite the potential benefits of SCMs in the cement and construction industry, they have not been fully utilized especially in developing countries in Africa. This may be due to low awareness of the potential benefits of SCMs among the stakeholders in the construction industry, and also limited availability. Nevertheless, due to extensive research into the usability of different materials as SCM, various materials are available in the construction market as binder systems. Thus, selecting the appropriate binder system to get the desired result for a particular concrete might be difficult for construction personnel. Hence, this study presents a review of the effects of various SCMs on the mechanical and durability properties of concrete. Six SCMs are reviewed. These SCMs include fly ash, silica fume, which are industry by-products; metakaolin, limestone calcined clay, which are naturally occurring minerals; rice husk ash, which is an agricultural waste material; and limestone-fly ash, which is a combination of an industrial by-product and a naturally occurring material. Firstly, an overview of the mechanical and durability properties of concrete is presented. This includes the presentation of general factors affecting the mechanical and durability properties of concrete. Subsequently, the effect of the various SCMs on mechanical (such as strength, elastic modulus, creep, and shrinkage) and durability properties (freeze-thaw, acid attack, sulphate attack, chloride-induced corrosion, carbonation-induced corrosion, and alkali-silica reaction) of concrete are presented. The review shows that the inclusion of appropriate dosage of these SCMs in concrete or mortar enhances their properties. Certain limitations of these SCMs are also discussed. This study also identifies areas of further research in relation to the properties of concrete produced with the SCMs.

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ABBREVIATIONS

APS: Average Particle Size

ASR: Alkali-silica Reaction

FA: Fly Ash

GGBS: Ground Granulated Blast-furnace Slag

HCP: Hardened Cement Paste

ITZ: Interfacial Transition Zone

LC3: Limestone Calcined Clay Cement

LC2: Limestone Calcined Clay

LFA: Limestone fly ash

MK: Metakaolin

PC: Portland Cement

RH: Relative Humidity

RHA: Rice Husk Ash

SCM: Supplementary Cementitious Material

SF: Silica Fume

W/b: Water-binder ratio

1. INTRODUCTION

1.1 Background

In the construction industry, concrete is known to be the most produced and used material. The demand for concrete will continue to increase vastly, especially in developing countries with a shortage of housing and infrastructure. In 2010, the estimation for housing shortage worldwide was about 428 million units [1]. The housing shortage in South Africa was estimated to be 2.1 million houses in 2019 [2]. Concrete is suitable to meet the huge demand in the housing industry.

According to Hilburg [3], about 4.4 billion tons of concrete are being produced yearly. However, by 2050, the figure is expected to rise to about 5.5 billion tons [3]. Concrete needs to be of adequate and appropriate quality, whether it is lightweight concrete or dense concrete. Consequently, the constituents of concrete are one of the contributing factors to its quality.

In terms of sustainability practice, cement is the most crucial constituent of concrete. Usually, concrete is composed of cement, aggregates, water, and admixtures. The cement industry contributes to the increasing content of carbon dioxide in the atmosphere as the production of cement is responsible for about 5-8% of global carbon dioxide emissions [3]–[6]. This can be attributed to the high energy intensity of the clinkerization process for plain Portland Cement. For instance, between 3.1-3.8 GJ of heat is needed to manufacture one ton of cement clinker [7]. It should be noted that most of the carbon dioxide (CO_2) emission during the clinkerization process is mainly from limestone. Therefore, to reduce the emission of CO_2 to the atmosphere (because it is one of the greenhouse gases contributing to global warming), the amount of cement clinker used in the concrete industry manufacturing needs to be reduced.

Furthermore, greenhouse gases including carbon dioxide, methane, nitrous oxide, water vapor, and other gases do not allow some of the heat on the Earth's planet to escape from the atmosphere to space. The phenomenon is referred to as the greenhouse effect in which its existence has made life comfortable on the Earth planet (without the greenhouse effect, the Earth would have been colder than it is) [8]. However, due to human activities, there are too many greenhouse gases in the atmosphere that keep the

Earth warmer than needed. Hence, there is a need to reduce the emissions of greenhouse gases into the atmosphere.

Over the past decades, the cement industry has explored three major approaches to reduce carbon dioxide emissions into the atmosphere. The first approach is the use of alternative raw materials and alternative fuels. The industry partly replaces cement raw materials with burning of industrial waste such as bottom ash in kiln while using fuels with low carbon content (such as bio-fuels) instead of conventional carbon-based fuels [9]. The second approach is the use of a thermally-efficient modern kiln system to improve the clinkerization process in order to reduce carbon dioxide emission [10]. The third approach, which can regarded as a very effective approach because it accounts for the highest reduction of CO_2 emission, involves using additives as a partial replacement for the cement clinker [11]. The additives are often called supplementary cementitious materials (SCMs).

In addition, many researchers have conducted experiments using different SCMs in the production of concrete with similar performance characteristics to conventional concrete. SCMs are used in the concrete industry either as pozzolans or hydraulic materials. These SCMs can either occur as natural minerals (such as feldspar, quartz, mica, etc.) or have an industrial origin [12]. Fly ash, silica fume, and blast furnace slag are the main industrial SCMs that have been adopted over the past decades [13]–[16]. Agricultural wastes such as rice husk ash, sugarcane bagasse ash, cassava peel ash, corn husk ash, etc., have also been employed as SCMs by many researchers. However, despite the fact that SCM is a well-documented technology in the construction industry over several decades, only few developing countries have adopted it [17]. Therefore, it is necessary to undertake a literature review on different supplementary cementitious materials in order to provide construction personnel in developing countries, especially in Africa, with an insightful discussion on the subject matter.

1.2 Problem Statement

With the increasing emission of carbon dioxide (CO_2) in the atmosphere, which contributes significantly to global warming, the need to reduce CO_2 emission is of paramount importance to scientists and researchers both locally and globally to make the Earth liveable for humanity.

To this end, there has been a shift in the production of plain Portland Cement with the incorporation of supplementary cementitious materials (SCMs) as a partial replacement for cement clinker in order to reduce the intensive energy consumption in the cement industry and to reduce the emission of CO_2 associated with the production of plain Portland Cement. Some of the SCMs that are being investigated in binary, ternary, or quaternary systems include; fly ash, silica fume, rice husk, slag, pulverized fuel ash, volcanic ash, limestone, calcined clay, etc. Despite the potential advantages and benefits of adopting SCMs in construction work, some construction personnel in developing countries, especially in Africa, are yet to adopt SCMs in their construction work fully. This is probably due to low awareness of the potential benefits of SCMs and the limited supply of SCMs that are industry by-products in developing countries. Moreover, due to extensive research into the use of different SCMs in the past few decades, various binder systems (for instance in the European Norm context, CEM I, CEM II, CEM III, CEM IV, CEM V, etc.) are available in the construction market. Hence, keeping up with the current technology or process and selecting appropriate binder systems in relation to concrete's mechanical and durability properties might be difficult for the construction personnel.

Therefore, there is a need to provide a review of the effect of different SCMs on concrete's mechanical and durability properties. It is hoped that this research will encourage the utilization of SCMs in developing countries and probably facilitate the development of standards for their use.

1.3 Research Aim and Objectives

The research aims to foster an increment in the level of adoption of SCMs in the construction industry by conducting a literature review on their potential benefits. The specific objectives of this study include:

i. To provide a brief introduction on concrete's mechanical and durability properties.

ii. To provide a literature review on the effect of different SCMs on concrete's mechanical and durability properties.

iii. To identify the percentage of the cement clinker that can optimally be replaced by the selected SCMs

1.4 Scope and Limitation

This research is a partial fulfilment of an MEng degree at the Department of Civil Engineering, University of Cape Town. This study is a minor dissertation worth 60 credit units.

This research study is limited to an extensive literature review. No experimental or field investigation was carried out. The literature review is limited to the effect of SCMs on the mechanical and durability properties of concrete. The SCMs that were reviewed are relevant to Africa situation, including fly ash (FA), silica fume (SF), metakaolin (MK), rice husk ash (RHA), limestone calcined clay (LC^3), and limestone-fly ash (LFA). Although SF is only available in a few African countries such as South Africa and Libya, however, the material is reviewed in this study due to its relative potential as a high-grade SCM, which needs to be known by the construction personnel.

2. RESEARCH METHODOLOGY

There are two main tasks involved in this study;

- An extensive literature review on the topic. This involves reviewing existing textbooks and research papers on the effect of SCMs on the mechanical and durability properties of concrete.
- Analysing and synthesising knowledge gained from the literature review. This involves the presentation of insightful discussions that can be helpful to decision-makers in the construction industry.

2.1 Literature Search Strategy

A systematic approach was used while searching for literature in various databases.

The following journal databases were specifically prioritized and used to search for the relevant literature relating to this research topic. These journal databases were specifically chosen because they focus on research relating to concrete, cement, green construction, and environmental sustainability, which are the themes of this research.

The journal databases included:

- Cement and Concrete Research
- Journal of Materials Science
- Sustainable Materials Construction and Technology
- Journal of the America Concrete Institute
- Construction and Building Materials
- International Journal of Concrete Structures and Materials
- Journal of Advanced Concrete Technology
- Journal of Green Building
- Journal of Environmental Sustainability

- Journal of Sustainable Construction Materials and Technology

Furthermore, the following general databases were searched for relevant literature with the keywords “Supplementary cementitious materials”, “Mechanical properties of blended concrete”, “Durability properties of blended concrete,” “Performance of SCM concrete,” “Mechanical properties of concrete made with blended cement,” and “Durability properties of concrete made with blended cement.”

The general databases included:

- Science Direct
- Web of Science
- Springer Link
- Open UCT
- Google Scholar

The downloaded texts were screened in three phases. The first phase was by reading the title and abstract of the texts, while the second phase involved scanning through the full text. Subsequently, the texts that passed through the second phase were read carefully to extract the relevant information needed for this study.

3. AN OVERVIEW OF MECHANICAL PROPERTIES OF CONCRETE

3.1 What is concrete?

The Latin word “concretus” which means “to grow together”, has been said to be the origin for the word “concrete.” The oldest concrete was found during the construction of a road in Galilee, Israel, in 1985, which was evidenced to have been made around 7000 BC [18]. Concrete is a very popular and commonly used material because its properties can be modified to meet the requirements of various projects. Concrete is generally made of water, fine aggregate, coarse aggregate, cement, and sometimes admixtures. It should be noted that cementing materials have been widely used in the ancient world for construction purposes. For instance, the ancient Romans mixed lime with volcanic ash to form a pozzolanic cement used in the production of concrete [18].

In terms of strength, concrete can be broadly categorized into two types: normal concrete and high strength concrete. The compressive strength of normal concrete always falls between 20 - 40 MPa, while that of high strength concrete is above 40 MPa [19]. Most construction works adopt the use of normal concrete, while high strength concrete is used in special projects that require a high level of strength. Some of the benefits of using concrete as construction materials are highlighted below:

- Concrete is economical and durable.
- Produced from raw materials available almost everywhere on earth.
- The properties of concrete can be modified to meet the demand for different jobs.
- In relation to other building materials, the compressive strength for reinforced concrete is very high.
- Concrete can be made to form different sizes and shapes.
- Concrete is a non-combustible material; it provides good fire resistance.
- Concrete can be recycled and used in new construction.

3.2 Mechanical properties of concrete

This section briefly discusses some important mechanical properties of concrete such as compressive strength, tensile strength, flexural strength, elastic modulus, shrinkage, and creep. Mechanical properties of concrete are important to study as they show the ability of concrete to withstand stress or deformation.

3.3.1 Strength properties of concrete

According to Bryan [20], the strength of a material can be defined as "the ability of that material to resist stress without failure". The strength of concrete is often regarded as one of the most vital properties of hardened concrete and used as an index of quality for concrete. Concrete strength is time-dependent, and thus, it is mostly specified at a specific age. There are various ways to determine the strength of concrete depending on the type of load applied to the concrete. These tests include compressive test, tensile test, flexural test, torsional test, and shear test. It should be noted that torsional and shear loading can be evaluated in terms of tension, compression, and flexural. Hence, it may be unnecessary to carry out the laboratory procedures for evaluating the two strengths as a means of control during construction [18].

3.3.1.1 Compressive strength

The compressive strength of concrete is termed as its ability to resist compressive loads without failures such as deflection or crack. Whether the signs of failure are visible or not, the maximum stress a concrete sample can withstand is referred to as failure [20]. The way of testing the compressive strength of concrete and the intrinsic properties of such concrete majorly determines the value for the compressive strength. As a result, it is important to standardize the ways of determining the compressive strength. In South Africa, there are four standards that can be used in relation to the determination of compressive strength of concrete: [21]–[24]. SANS 5863 [21] describes the method for evaluating the compressive strength of test specimens (i.e., cylinder, cube, or cubes cut from half-prism). SANS 5860 [22] specifies a method for ensuring that the cast concrete samples are in appropriate shape and dimension. SANS 5861-2 [23] discusses the procedures for sampling freshly mixed concrete, while SANS 5861-3 [24] describes the manufacturing and curing of test samples. According to the standards, it is important to note that all specimens need to be kept in water for at least 48hours before testing.

After testing, the value for the compressive strength of each specimen can be evaluated from the formula presented in equation 1.

$$f_{cc} = \frac{F}{A_c} \quad \text{...(1)}$$

where:

f_{cc} is referred to as the compressive strength, measured in megapascals; F represents the maximum applied load at failure, measured in Newtons; and A_c represents the cross-sectional area of the tested specimen, measured in square millimetres.

Furthermore, a set of three specimens of the same shape and dimension should be tested, and the difference between the highest and lowest results must not exceed 15% of the average value. Otherwise, the tests would be deemed invalid.

3.3.1.2 Tensile strength

Tensile strength is referred to as the maximum load a concrete can withstand under tension. However, due to the uneasiness attached to applying a concentric load on concrete specimens and the high variations in the direct tensile test results, thus; indirect tensile strength tests are mostly adopted [18]. Splitting of elements by the application of line loads (see Figure 3.1) is one of the indirect tensile tests used in South Africa. SANS 6253 [25] describes the testing apparatus, how to test the specimen in the testing machine, and how to calculate the value of the tensile strength of the tested specimen for the splitting tensile strength test. It should be noted that the splitting tensile test is relatively easy to carry out than other tension tests [18]. Moreover, the tensile strength of concrete is about one-tenth of its compressive strength [18], [20].

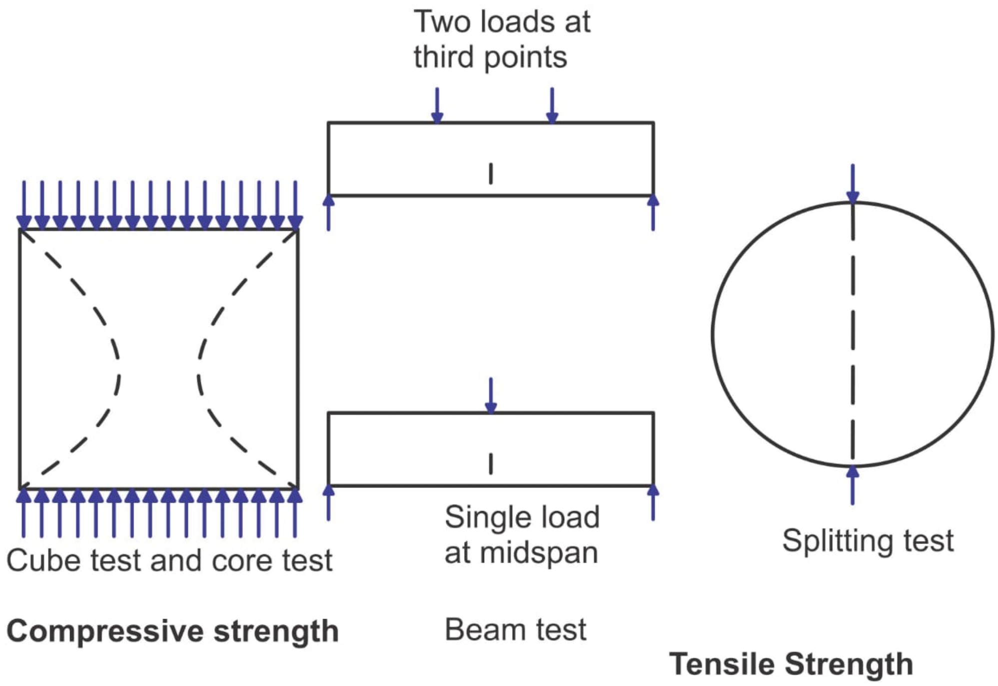

Figure 3. 1: Diagrammatic representation of compressive and tensile strengths [20]

3.3.1.3 Flexural strength

Flexural strength of concrete, also known as modulus of rupture, is termed as the ability of the concrete to resist failure in bending. Common elements that are loaded in flexure or bending include beams, slabs, road pavements, etc. Flexural strength is essential in concrete road design as road slabs often fail in bending and not compression. The volume and size of the coarse aggregates used in making concrete have a significant effect on the concrete's flexural strength [18]. The flexural strength of concrete is about 10-20% of its compressive strength [18]. Due to the fact that the degree to which factors such as curing conditions, type, and the dimension of aggregates, cement/water ratio, mix proportions, and age affects the three categories of concrete's strength (flexural strength, tensile splitting test, and compressive strength) differs; therefore, no absolutely correct relationship can be established between them [20]. However, an approximate relationship between flexural strength, tensile splitting test, and compressive strength is presented in Figure 3.2. It should be noted that 1 MPa is equal to 1 N/mm^2 as the strength presented in Figure 3.2 is measured in N/mm^2. Moreover, SANS [26] describes the testing apparatus, procedures (i.e., how to place the specimen in the testing machine and the rate of loading), and calculation of results.

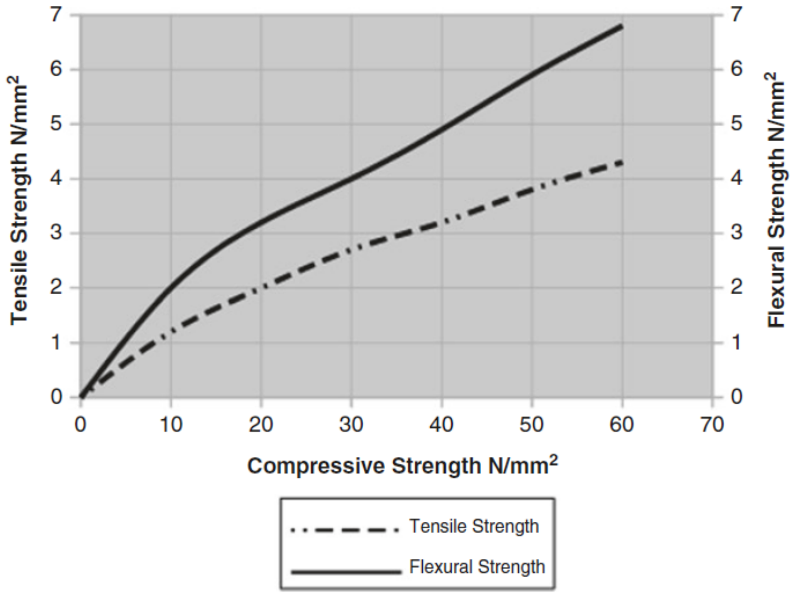

Figure 3. 2: An approximate relationship between flexural, tensile, and compressive strength [18].

3.3.1.4 Factors affecting the strength of concrete

Some of the factors that influence the strength of concrete are briefly discussed below.

- **Heterogeneity:** Concrete is said to be a heterogeneous material as its hardened form consists of hardened cement paste (HCP) and aggregates. Thus, concrete can be said to be composed of two phases: HCP and aggregate phases. When concrete is loaded, there tends to be a concentration of stresses at the interface between the two phases, which mostly weakens the concrete.

- **Porosity:** No matter how well compacted and cured concrete is, some unreacted water of the fresh concrete will create some pores in the concrete. The porosity of concrete should be reduced as much as possible because there is an inverse relationship between the strength of concrete and porosity.

- **Aggregate:** It is no doubt that aggregate is the strongest constituent of concrete. It should be noted that the crushing value for an aggregate only has a significant effect on concrete with very high strength [20].

- **Compaction:** It is essential that concrete is well and fully compacted after placing in order to avoid having porous concrete with a defect such as honeycombing, which can reduce the strength of the concrete. It was reported that about 30% of concrete's strength could be lost due to the presence of only 5% of air void [18].

- **Curing:** Curing plays a vital role in the strength development of concrete. Concrete needs to be moist until it develops sufficient strength. Proper curing also contributes to the production of concrete with better surface hardness and good abrasion resistance.

- **Cementitious material:** There are various pieces of evidence from previous researches that the type of cement used in the making of concrete can influence the strength of such concrete significantly [7], [14], [17]. In general, the cement type that gives higher strength at an early age (i.e 24 hours) compared to plain Portland cement is referred to as rapid hardening cement (RHC) such as SF. The major difference between the plain Portland cement and RHC is the higher composition of C_3S in RHC.

- **Supplementary cementitious materials (SCMs):** Some SCMs reduce the strength of concrete during their early age due to their slow hydration reaction while some, due to their faster rate of hydration, improves the strength of concrete at any age. FA concrete is a typical example of the former, while SF concrete is a typical example of the latter [17], [18].

3.3.2 Concrete deformation

Concrete deforms when subjected to external stress. The initial deformation (Figure 3.3) is termed as an elastic strain which is then followed by an increase in strain (creep). Concrete structures can also deform due to thermal strain and shrinkage. According to Alexander & Beushausen [27], factors affecting concrete deformation can be grouped into intrinsic and extrinsic factors. Some of the intrinsic factors include the age of paste, water/cement ratio, temperature, admixtures, degree of hydration, moisture content, nature of interfacial transition zone, cement extenders, etc. Age of loading, rate & time of drying, duration of load, relative humidity & temperature, and the level of the applied stress are among the extrinsic factors affecting concrete deformation [27].

3.3.2.1 Elastic modulus

The elastic modulus of concrete shows its ability to resist elastic deformation when subjected to stress. It is also known as the modulus of elasticity. In simple terms, the modulus of elasticity is defined as the ratio of applied stress to the corresponding axial strain. The unit of elastic modulus is the same as that of stress (MPa) since the strain is dimensionless. Unlike steel, concrete does not obey Hooke's law as its stress-strain relationship is non-linear; this can be attributed to the microcracking that do occur at the interface between cement paste and aggregate [27], [28]. Although each aggregate should have its own specific compressive strength-elastic modulus relationship, SANS 10100-1 [29] presents an equation to estimate elastic modulus of concrete based on multiplying its compressive strength by a constant value and an aggregate stiffness factor. Moreover, the elastic modulus of concrete can be determined experimentally in two ways: static tests [30], [31] or dynamic tests [32], [33].

The major factors that affect the elastic modulus of concrete are highlighted below:

- **Stiffness of the cement paste:** There is a direct relationship between stiffness of the cement paste and elastic modulus; that is, the stiffer the paste, the higher the elastic modulus.

- **Stiffness of the aggregate:** The aggregate phase is usually the phase with the highest stiffness in concrete. Thus, the stiffer the phase, the higher the elastic modulus of the concrete [27].

- **Interfacial Transition Zone (ITZ):** The difference in elastic modulus of concrete with similar compressive strength but different aggregate stiffness may be partially attributed to the ITZ between the aggregate and cement paste. Thus, the denser the ITZ, the higher the modulus of elasticity [27], [34].

- **Supplementary Cementitious Materials (SCMs):** Some SCMs materials such as GGBS and FA lower the early age of elastic modulus due to their slow hydration process, while condensed silica fume (CSF) slightly improves the modulus of elasticity of concrete due to early densification of the ITZ [27].

3.3.2.2 Creep

Creep, which is time-dependent and occurs at all stress levels, can be defined as an increase in strain (deformation) of concrete that is subjected to constant load (Figure 3.3). It should be noted that creep is an important factor in the serviceability of concrete structures as it may be several times larger than the elastic strain [28]. Creep can be said to be advantageous to concrete structures as it facilitates the relief of stresses in concrete because of differential structural movement. However, its negative effects include prestress loss, cracking of concrete structures due to excessive deflections, buckling of long columns, and instability of shell structures [27].

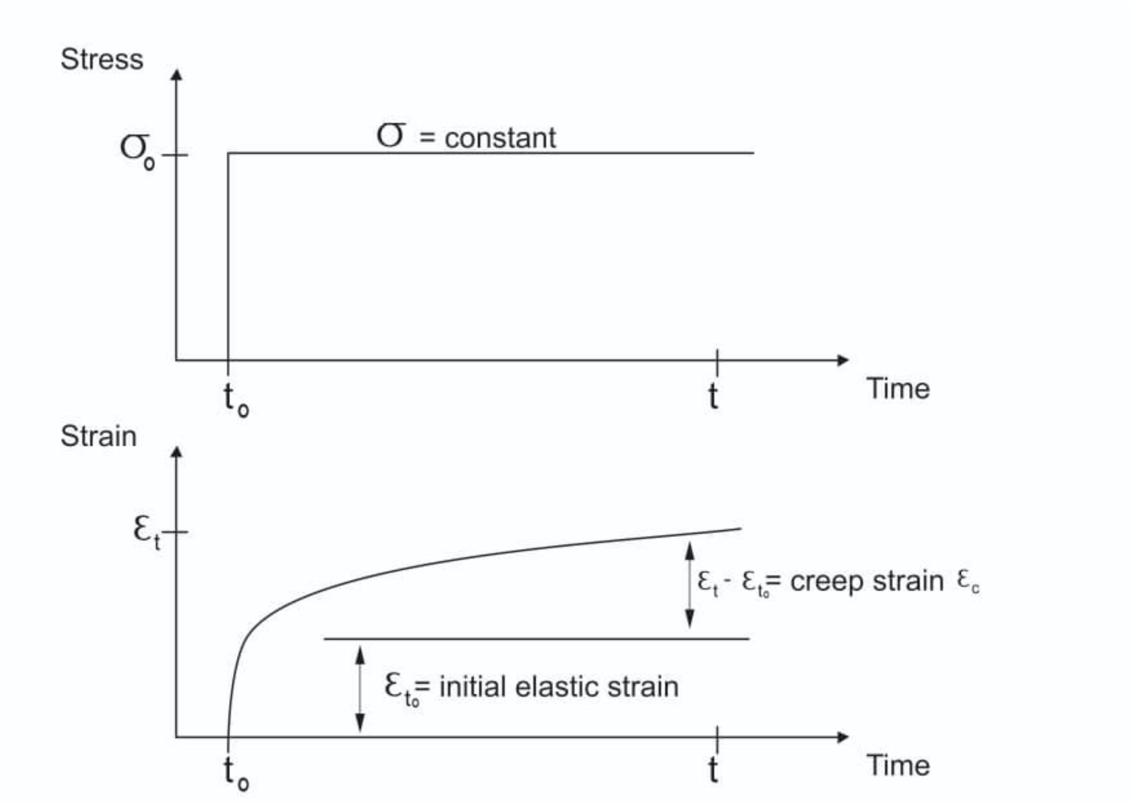

Figure 3. 3: Diagrammatic representation of creep [27]

Specific creep is an important parameter in the study of creep, which can be used to compare the creep behaviour of concrete subjected to different levels of stress. Specific creep (C_c) is defined by the equation 2, where ε_c represents the creep strain and σ represents the sustained load. Creep coefficient is another useful comparative parameter which is the ratio of creep strain to the elastic strain.

$$C_c = \frac{\varepsilon_c}{\sigma} \qquad \text{-------------} \qquad (2)$$

Some of the factors affecting the creep of concrete are briefly discussed below:

- **Water/binder ratio:** The water/binder (w/b) ratio is one of the factors that control the strength and stiffness of cement paste; therefore, it also has an indirect effect on the creep of cement paste. Hence, an increase in the w/b ratio will cause an increase in creep.

- **Moisture content:** Since creep depends largely on the presence of moisture in the concrete, the lesser the moisture content, the lesser the creep. This could be explained by the fact that concrete that is allowed to dry before subjecting them to any loading will creep less because there would be no additional component of drying creep [27].

- **Cement type:** The constituents of cement and its fineness also influence the creep of cement paste. Cement that contains a high amount of C_3S, due to the fast setting of the cement, and low amount of C_3A tend to produce cement paste with less creep [27]. Extremely fine cement has also been found to produce cement paste with higher early creep [15].

- **Aggregate:** Aggregates with lower elastic modulus offer less restraint to the creep of the cement paste, therefore, causing higher creep on the concrete. According to [27], [28], the creep of concrete containing aggregates with a high modulus of elasticity can be as low as one-fourth of the concrete with aggregates of low elastic modulus.

- **Time:** As indicated earlier, creep is time-dependent. Neville [15] stated that creep was still measurable on specimens of 30 years of age. However, it should be noted that about 80% of the creep that will occur after 20 years of concrete manufacture may occur after one to two years, depending on the member size [27].

- **Cementitious materials:** Some supplementary cementitious materials such as fly ash and silica fume have been found to give a low creep factor in relation to concrete made with plain cement[35], [36].

According to Brooks [28], ASTM C512 [37] is the only approved standard test for the determination of creep of concrete subjected to uniaxial compression. Moreover, due to the fact that there are many factors influencing the creep of concrete, it may be very difficult to have a unified test approach that will take all relevant factors into account and be suitable for all conditions. Thus, estimation of creep is at best imperfect [27]. To this end, SANS 10100-1 [29], adapted from BS 8110-2 [38], presents an equation for estimating the final creep strain of concrete. Equation 3 is presented below:

Final creep strain = elastic strain × creep factor…………………………………..(3)

Creep factor can be generally defined as the ratio of creep strain to the initial strain after a short loading period. The specified values of creep factors vary for different codes, as they depend on the age of loading [39]

3.3.2.3 Shrinkage

As mentioned earlier, shrinkage occurs when there is a volume change of concrete. It can occur when concrete is in its fresh and hardened states. It should be noted that shrinkage is of two forms under drying conditions: autogenous shrinkage and drying shrinkage.

Autogenous Shrinkage

Immediately after the setting of concrete, the reduction in the volume of cement paste that occurs due to the hydration process is known as autogenous shrinkage, which is mostly about 8% of the total cement paste volume [18]. The influence of autogenous shrinkage on concrete with a w/b ratio greater than 0.4 is usually minimal and may be negligible except for mass concrete structures [15]. When the w/b ratio is less than 0.35, proper attention should be given to autogenous shrinkage, especially when in situ concrete (i.e., new) is cast against hardened concrete, as it can be very high at this stage [40]. Apart from the w/b ratio, other factors that can influence the rate of autogenous shrinkage include water content in the cement paste, chemical composition of cement, time, temperature, etc. Moreover, Neville [15] indicated that concrete made with a high amount of fly ash tends to have less autogenous shrinkage.

Drying shrinkage

The loss of water from the concrete to the environment, which results in a decrease in the volume of hardened concrete, is referred to as drying shrinkage. When concrete is in drying condition, water is first lost from its voids, followed by capillary Portland cement gel, which results in drying shrinkage [18]. Drying shrinkage can cause cracking on the concrete surface, which may be of an appreciable size that may be undesirable from an aesthetics and durability point of view. It should be noted that drying shrinkage cracks (see Figure 3.4) can occur randomly at any location on the surface of the concrete; so far, there is a restraint to the movement of shrinkage. Furthermore, drying shrinkage can cause deflections in flexural members. For instance, in singly reinforced concrete, deflections due to shrinkage may be responsible for up to 25% of the total deflection of beams [27].

Figure 3. 4: Drying shrinkage cracking [41].

Some of the factors affecting drying shrinkage are briefly discussed below:

- **Structure of the HCP:** The structure of the HCP in concrete has a great influence on the shrinkage of concrete structures. HCP comprises small gel pores and bigger capillary pores [27]. The HCP structure is mainly affected by the w/b ratio and the rate of hydration reaction. More volume of hydration products will be formed when the rate of hydration reaction is fast, which can be a result of a low w/b ratio. When subjected to drying conditions, the free water attributed to the bigger capillary pores will be lost first, followed by the gel water attributed to the small gel pores, which make the paste shrink. It should be noted that HCP with a lower w/b ratio will be stiffer, therefore, tends to contract less.

- **Aggregate:** Concrete shrinkage tends to reduce when there is an increase in concentration and stiffness of aggregate. As indicated by Alexander & Beushausen [27] in their book, concrete with about 65-70% aggregate volume concentration may only exhibit one-fifth of HCP shrinkage.

- **Cement composition:** The amount of gypsum and C_3A in cement also have an effect on the shrinkage of HCP. According to Neville [15], cement deficient in gypsum tends to increase the shrinkage of HCP. Also, cement with high content of C_3A tends to reduce the shrinkage of HCP.

- **Supplementary cementitious materials:** A study carried out by Alexander [42] indicated that the use of ground granulated blast furnace slag (GGBS) as an SCM caused a reduction of shrinkage strains in sealed specimens. Furthermore, according to Guneyisi et al.[43], the incorporation of MK in Portland cement concrete resulted in lower drying shrinkage compared to the control specimen (plain PC concrete).

Other factors that influence drying shrinkage include the shape and size of the concrete member (structure with large shape and size tends to shrink less due to their capability of retaining internal moisture for long periods), relative humidity (drying shrinkage reduces with an increase in relative humidity [27]), temperature (elevated temperature tends to increase drying shrinkage of concrete), time (prolonged curing time can reduce shrinkage due to slow drying of the moisture content of the concrete), etc. Shrinkage strain can be estimated using a graphical method presented in SANS 10100-1 [29] adapted from BS 8110-2 [38].

3.4 Summary

This chapter starts with a brief definition, history, and benefits of using concrete as a construction material. The benefits of using concrete include the fact that concrete is economical, durable, non-combustible and can be modified to meet the demand for different applications. Subsequently, the strength and deformation properties of concrete are briefly discussed. It should be noted that the strength of the concrete is mostly regarded as one of the essential properties of concrete and often times used as an index of quality for concrete. Compressive strength, tensile strength, and flexural strength are the discussed strength properties of concrete.

As such, it is important to understand the factors that affect the strength of concrete. Importantly, it was noted that some SCMs improve the strength properties of concrete at all ages, while some only improve the strength properties of concrete at a later age due to their slow hydration reaction. Another important factor that influences the strength of concrete is porosity. No matter how well compacted and cured concrete is, the concrete would still have some unreacted water, which are responsible for creating some pores in the concrete, thereby reducing the concrete's strength. More so, concrete are also influenced by type of aggregate, compaction, curing, and the type of binder used in making of the concrete.

Elastic modulus, shrinkage and creep behaviour of concrete are discussed as the deformation properties of concrete. The elastic modulus, which represents the ability of the concrete to resist elastic deformation, is affected by various factors. These factors are discussed in this chapter as they need to be considered while manufacturing the concrete so as to avoid unmatured deformation of concrete structures. The stiffness of the aggregate and ITZ are very important factors. The former represents the phase with the highest stiffness in the concrete; thus, the stiffer the phase, the higher the elastic modulus. Also, concrete specimens with denser ITZ between cement paste and aggregate have been found to have a higher elastic modulus. Besides, blended concrete consisting of some SCMs such as SF can improve the elastic modulus of concrete.

Creep, as a time-dependent property, can be affected by various factors, including moisture content. If the concrete is loaded before drying, this means that the creep will tend to be higher compared to when the concrete is dry, as the component of drying creep will be added in the former situation. Furthermore, autogenous and drying shrinkage are discussed. The structure of HCP is the most important factor affecting drying shrinkage as it directly relates to the rate of cement hydration.

4. AN OVERVIEW OF DURABILITY PROPERTIES OF CONCRETE

According to Ballim et al. [44], the durability of a concrete structure can be defined as "the ability of a structure or component to withstand the design environment over the design life, without undue loss of serviceability." The durability of concrete structure is related to its material performance in a given environment over its expected service life. For instance, two concrete structures with similar material properties may deteriorate at a different rate if they are exposed to different environmental conditions.

Many old concrete structures have performed well over many decades, which shows the inherent durability of concrete. In relation to this inherent durability property of concrete, some structural engineers are of the view that durability is not a major problem in concrete structures [44]. However, this is not always the case, as some newer infrastructures deteriorate earlier than expected [18]. For instance, Surahyo [18] reported a case of 8 bridges in the United Kingdom whose repair cost (due to deterioration) was almost six times their initial cost of construction.

Therefore, in order to achieve a durable concrete structure, the designer of such structure needs to know the acceptable performance criteria of a concrete structure in a given environment. It is also essential to assess the actual quality of the built structure (by carrying out various tests) in order to ascertain that the concrete structure has been constructed according to the design specifications. This chapter presents an overview of some important durability properties of concrete: free-thaw resistance, sulphate attack resistance, acid attack resistance, resistance to chloride-induced corrosion, resistance to carbonation-induced corrosion, and Alkali-silica reaction (ASR). As with every other research, this study is limited to the above-mentioned durability properties and do not include other durability properties such as fire resistance, abrasion, erosion, permeability, etc.

4.1 Freeze-thaw resistance

The phenomenon of freeze-thaw occurs when water molecules of saturated concrete freeze upon temperature drop. Frozen water brings about a 9% increase in volume, which causes distress to the concrete. Upon the rise of the temperature in a warmer period, thawing occurs. Since the water molecules will melt away during the thawing process, tiny cracks can surface on the concrete. When the colder period comes again, the tiny cracks are filled with water molecules, and further expansion will occur, which causes more distress in the concrete. Hence, repeated cycles of freezing (during the cold period) and thawing (during the warm period) can cause scaling, cracking, spalling, and loss of concrete strength [18]. This phenomenon, freezing & thawing, do occur in some Africa countries such as South Africa, Algeria, Egypt, etc. [45], [46]. For instance, the average temperature variation in South Africa ranges from -2^{o}C to 26^{o}C in the winter period, which lasts for about 5 months [47]. Deterioration due to freezing and thawing is common in horizontal and vertical surfaces that are exposed to water.

The test methods for measuring the resistance of concrete to freezing and thawing can be grouped into two: direct and indirect methods. For direct methods, concrete samples are subjected to artificial freezing and thawing environmental loads, and the internal structural damage of the concrete will be assessed. The indirect methods involve measuring the property that affects frost resistance, such as permeation, porosity, electrical resistivity, etc. [48].

4.1.1 Factors affecting freeze-thaw resistance

- **Entrained air**: The pressure due to the frozen water in concrete can be relieved with the aid of air entrainment as the presence of air voids will allow concrete to expand on freezing without causing large internal stresses [18], [44]. It should be noted that a moderate amount of air should be entrained in concrete as little entrained air may not be sufficient to provide adequate resistance to freezing and thawing, while too much-entrained air will significantly affect the strength of the concrete. To this end, American Concrete Institute (ACI 301, 2016) [49] presents some recommendations for the air content of concrete based on the exposure condition and the aggregate size employed in making such concrete, as shown in Table 4.1.

- **Water/binder ratio**: The water/binder ratio of concrete influences the degree of resistance of concrete to freezing and thawing. ACI-201 (2016) and BS 8110-

2 (1985) [38], [50] recommend that the maximum water/binder ratio of concrete exposed to freezing and thawing should be 0.5 and 0.55, respectively.

- **Aggregate**: The selection of aggregate in an environment where freezing and thawing take place is essential to have durable concrete. For instance, some aggregates tend to absorb too much water, which makes it difficult to accommodate the expansion during freezing [18]. Therefore, aggregates that are susceptible to frost action should not be used in the manufacture of concrete structures that are exposed to freezing and thawing. Typical suitable aggregates, which are less absorbable and low-shrinking, in this environment would consist of dolomite, basalt, granite, and quartz [18].

- **Supplementary cementitious materials**: Various SCMs have been investigated in terms of their resistance to freezing and thawing. According to research conducted by Reiterman et al. [51], the optimum replacement of cement, which gave better performance compared to Portland cement, by ceramic powder, fly ash, and blast furnace slag are 12.5%, 37.5%, and 50%, respectively, by weight of cement to give adequate resistance to freezing and thawing. The better performance of the blended concrete may be due to their denser microstructure enhanced by the pozzolanic activity of the SCMs.

Table 4.1: Total air content for concrete exposed to freezing and thawing cycles [49]

Nominal maximum aggregate size (in.)	**Air content (%)**	
	Exposure Classes F2 and F3	Exposure Class F1
3/8	7.5	6.0
1/2	7.0	5.5

Nominal maximum aggregate size (in.)	**Air content (%)**	
	Exposure Classes F2 and F3	Exposure Class F1
3/4	6.0	5.0
1	6.0	4.5
1 – ½	5.5	4.5

4.2 Sulphate attack resistance

Sulphate attack is one of the main durability problems of concrete in sulphate-rich environments. The reaction of some hydration products in cement paste with sulphate can cause the expansion of concrete [52]. It should be noted that all sulphates are potentially undesirable to concrete. In particular, sodium, magnesium, and calcium sulphates are very dangerous to concrete and occur widely in clayed soil, seawater, and groundwater [18]. Sulphates in solution form react with tricalcium aluminate and calcium hydroxide of cement paste to produce ettringite (calcium sulpho-aluminate) and gypsum (calcium sulphate), respectively (equation 4 and 5).

$$C_3A+3CaSO_4+32H_2O \rightarrow 3CaO.Al_2O_3.\ 3CaSO_4.32H_2O \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(4)$$

$$Ca(OH)_2 + SO^{2-}{}_4 + 2H_2O \rightarrow CaSO_4\,.2H_2O + 2OH^- \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(5)$$

Gypsum and ettringite are expansive products that occupy more volume than the compounds they replace so as to deteriorate the concrete structure [18], [44], [52]. Usually, concrete attacked by sulphate does show a whitish damage appearance at the corners and edges of the concrete, which is then followed by cracking and spalling. The permeability of concrete increases when it cracks, thus facilitating the ingress of deleterious substances into the concrete.

Concrete specimens are normally stored in sulphate solutions such as magnesium sulphate or sodium sulphate to test for the resistance of the concrete against sulphate attack. The effect of exposing the concrete to sulphate solution can be estimated by measuring the change in dimension of the concrete, loss of strength & weight and change in the dynamic modulus of elasticity [52].

4.2.1 Factors affecting sulphate attack resistance

Some of the essential factors influencing the resistance of concrete to sulphate attack are briefly highlighted below.

- **Type of sulphate**: The rate of deterioration of concrete exposed to sulphate solution depends on the type of the sulphate. According to Surahyo and Zongjin [18], [52], magnesium sulphates cause the most damage to concrete amongst the sulphates followed by sodium sulphates and then calcium sulphate. Similar ionic radii and the same valence electrons between Mg^{2+} and Ca^{2+} makes magnesium sulphate to give the most severe attack on concrete.

- **Sulphate concentration**: The concentration of sulphate in the sulphate solution to which the concrete is exposed also determines the rate of concrete deterioration. A study conducted by Maslehuddin [53] showed that the rate of deterioration of concrete under sulphate attack increases with an increase in sulphate concentration. Sulphate concentration of 1% or below may have no significant effect on the concrete, only if the concrete is well-compacted [18].

- **Type of cement**: Since the expansive reaction of sulphate attack only occurs with the presence of calcium hydroxide and calcium aluminate in hydrated cement paste, therefore, the chemical composition of cement plays an essential role in determining the resistance of mortar or concrete to sulphate attack. Hence, Sulphate Resisting Portland Cement with low content of tricalcium aluminate is an excellent binder to use in manufacturing concrete that is exposed to sulphate attack [54].

- **Supplementary cementitious materials**: Due to the pozzolanic activity of some SCMs (such as FA, SF, and MK), which consumes calcium hydroxide in the cement paste; the inclusion of FA, SF, and MK in the making of concrete have been reported to increase the resistance of concrete to the attack of sulphates [54].

4.3 Acid attack resistance

The attack of concrete by acid is due to the alkaline nature of concrete. An acid reacts with the alkaline component of cement paste, which lowers the alkalinity degree of concrete or result in a complete neutralization reaction. The loss of alkalinity of concrete is disastrous and thus causes the concrete matrix to disintegrate [44]. Most times, the reaction of an acid with calcium hydroxide, the alkaline substance in concrete, produces water-soluble calcium compounds, which are often leached away [15]. This leaching increases the porosity of the concrete, which leads to damage of the concrete matrix and also reduces the compressive strength of such concrete [18].

The sources of acid that attack concrete include but are not limited to sewage that forms sulphuric acid, mine & industrial water that contains acid, clay soil that contains iron sulphide which forms sulphuric acid, CO_2 and SO_2 present in the atmosphere also turn to carbonic and sulphuric acids respectively [18]. Moreover, due to the presence of sulphate ions in sulphuric acid, it has been recognised as the most severe acid on concrete structures [55].

4.3.1 Factors affecting acid attack resistance

Below are some of the important factors affecting the acid attack resistance of concrete structures.

- **The pH of the attacking acid**: One of the important factors that determine the rate of deterioration of concrete is the pH of the attacking acid. The lesser the pH of the acid, the more the severity of the attack on the concrete. According to Surahyo [18], acid of less than 6.5 pH attacks concrete moderately while acid of pH less than 4.5 causes severe deterioration to concrete.

- **Temperature**: The rate of acid attack on concrete increases with an increase in the temperature of the environment. Research conducted by Mahmoodian & Alani [56] showed that there is a significant loss of mass in concrete exposed to an acidic environment at various temperatures (10^{o}C, 20^{o}C, and 30^{o}C). It was found that the loss in mass of concrete was most significant at 30^{o}C. Furthermore, a study conducted by Omosebi et al. [57] indicated that temperature is one of the most vital factors that affect concrete in an acidic environment.

- **Supplementary cementitious materials**: Due to the pozzolanic activity of many SCMs and influence on the hydration mechanism, different SCMs have been found to improve the resistance of concrete to acid attack. Concrete made with blended cement containing silica fume and metakaolin showed greater resistance to acid attack than plain concrete [58]. Similarly, the incorporation of slag and metakaolin into the making of concrete gives better resistance to acid attack than plain concrete, as reported by [59].

4.4 Corrosion of embedded steel reinforcement

Corrosion can be referred to as the process that occurs when steel returns back to its natural state (i.e., iron oxide) after being subjected to reactive environments. It is an electrochemical process that comprises the movement of electrons between anodic and cathodic sites on the reinforcing bar (see Figure 4.1) [60]. The tendency of metal or steel to return back to their stable state can be regarded as the main cause of corrosion. Four processes need to occur before corrosion can occur [60]:

Anodic process: This occurs at the anodic site where the electrons are liberated in the metallic state to form iron ions (equation 6), which are hydrolysed to give acidity (equation 7).

$$Fe \rightarrow Fe^{2+} + 2e^{-} \quad \ldots\ldots (6)$$

$$Fe^{2+} + 2H_2O \rightarrow Fe(OH)_2 + 2H^{+} \quad \ldots\ldots (7)$$

Cathodic process: This occurs at the cathodic site where consumption of the electrons produced at the anode takes place. This reduction reaction produces alkalinity (equation 8)

$$O_2+2H_2O+4e \rightarrow 4OH \ldots\ldots(8)$$

Electrolyte: The alkaline pore solution in concrete serves as an electrolyte that conducts electric current flow from anode to cathode.

Metallic path: To complete the circuit, there must be movement of electrons within the embedded steel from their production site (anode) to their site of consumption (cathode).

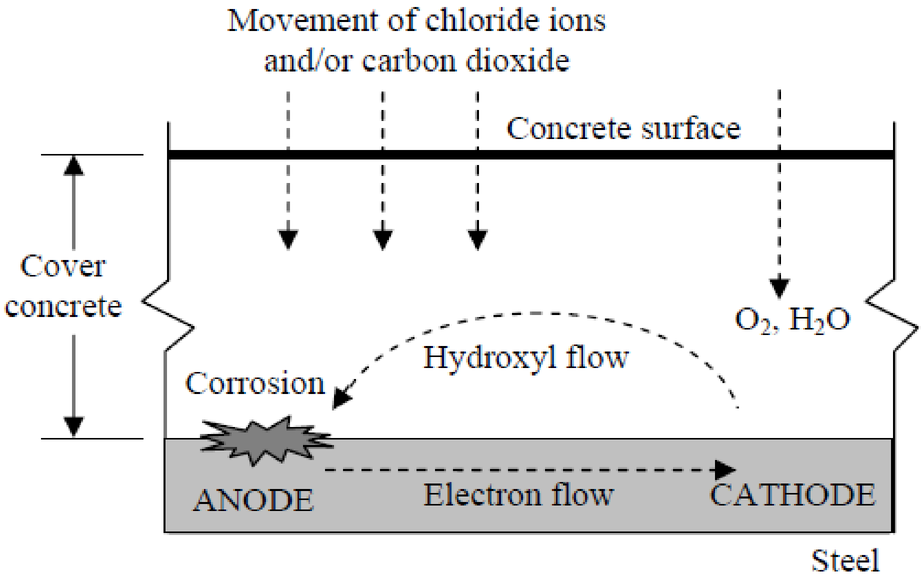

Figure 4. 1: A diagrammatic representation of corrosion of embedded steel [60]

It should be noted that embedded reinforcing steel in concrete is normally protected by a passive film of iron oxide due to the high alkalinity of concrete. Furthermore, corrosion of embedded steel can either be macrocell or microcell. It is termed macrocell when the anodic and cathodic sites are clearly separated from each other while regarded as microcell when the sites are adjacent to each other. According to Otieno et al., [60], macrocell corrosion often occurs in chloride-induced corrosion, while microcell corrosion is common in carbonation-induced corrosion.

4.4.1 Resistance to chloride-induced corrosion

Reinforcement corrosion is the most severe durability issue in reinforced concrete structures, as indicated by Zongjin [52]. Due to the high alkalinity nature of concrete, the embedded reinforcement in concrete is usually protected by the formation of a thin protective film of iron oxide on its surface. This phenomenon is regarded as passivity. The passivity of the embedded reinforcement can be lost when the thin film is destroyed by the ingress of chloride ions [61]. Reinforcement corrosion takes place as soon as the passivity is lost under the presence of oxygen and water. Corrosion of embedded reinforcing steel leads to the formation of rust which increases the volume of concrete. Due to the increase in the volume of the concrete, tensile stresses are created, which can result in cracking and delamination, and subsequently spalling.

Penetration of chloride ions into the concrete occurs through diffusion, capillary absorption, or hydrostatic pressure. Most researchers have employed Fick's second law of diffusion to predict the concentration of chlorides in concrete. [62]. The resistance of concrete to the ingress of chloride ions by diffusion can be estimated by performing a chloride conductivity test. The procedures for the chloride conductivity test are explained in [63]. Figure 4.1 shows a diagrammatic representation of chloride-induced corrosion.

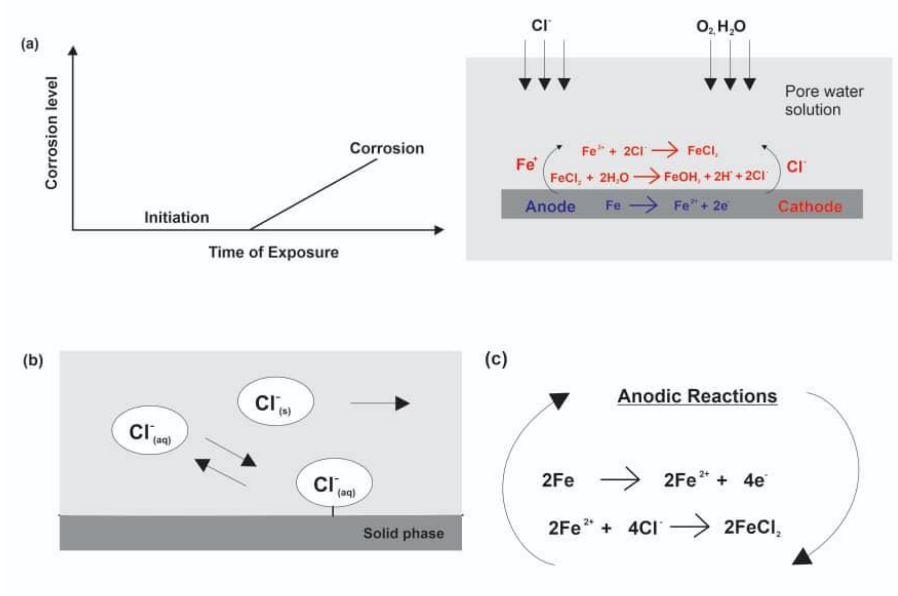

Figure 4. 2: A diagrammatic representation of chloride-induced corrosion [64]

Factors affecting the resistance of concrete to chloride-induced corrosion

Some of the vital factors that influence the rate at which concrete resists chloride attack are briefly highlighted below.

- **Permeability and porosity of concrete**: Without adequate chloride content at the surface of the embedded steel, chloride-induced corrosion will not take place. Therefore, the porosity and permeability of concrete are vital to determining the rate of deterioration of concrete due to chloride attack. The porosity of concrete is influenced by the water/binder ratio and aggregate-binder ratio. Similarly, the permeability of concrete to a large extent depends on the effectiveness of curing methods or procedures adopted after placing the concrete and the degree of compaction of such concrete during placement [65].

- **Environmental factors**: Other main causes of deterioration of reinforced concrete structures in relation to chloride attacks are the environmental factors (i.e., exposure condition and temperature). For instance, the rate of chloride ingress into concrete in a marine environment will be faster than concrete in a non-marine environment. Mazer et al. [66] examined the penetration of chloride under three different exposure conditions: underwater, tidal variation, and atmosphere at four different temperatures (15^{o}C, 20^{o}C, 25^{o}C, and 30^{o}C) over a period of 18 months. They found that the concrete samples in the underwater region indicated the highest concentration of chlorides. More so, they observed that an increase in temperature leads to an increase in the concentration of chlorides.

- **Supplementary cementitious materials**: The effectiveness of incorporating SCMs in the manufacturing of concrete has been investigated by many researchers. Gettu et al. [67] investigated the chloride resistance of concrete made with FA30 (it contains 30% of FA) and LC^3 (limestone calcined clay), and 100% plain concrete. They found out that LC^3 has the highest chloride resistance and then followed by FA30. Moreover, various researchers have also confirmed the effectiveness of concrete consisting of at least 15% of SF in increasing the resistance of the concrete against chloride ingress [68], [69].

4.4.2 Resistance to carbonation-induced corrosion

As indicated earlier, the embedded reinforcing steel is passivated with a thin layer of gamma ferric oxide. However, the passivity can be lost due to the ingress of carbon dioxide into the concrete, which reacts with calcium hydroxide in order to lower the pH of the concrete. This phenomenon is referred to as carbonation. The pH of newly made concrete is about 13. However, the pH reduces to about 8 when carbonation occurs, leaving the embedded reinforcement to be susceptible to corrosion since the thin protective film is being destroyed [52]. It is important to note that carbonation itself is not an issue in concrete; it becomes an issue when carbon dioxide reaches the surface of the embedded steel to depassivate it. Figure 4.3 shows a diagrammatic representation of carbonation-induced corrosion.

The rate at which carbonation causes reinforcement corrosion is mostly expressed as the rate of carbonation penetration which is measured in micrometres (µm) per year. Corrosion rate below 2µm/year can be considered harmless to reinforced concrete [52]. Moreover, according to Bertolini et al. [70], the corrosion rate of 50-100 µm/year and values above 100µm/year are considered to be high and very high, respectively. It should be noted that carbonation can also cause shrinkage of concrete, which may be responsible for one-third of the total shrinkage of concrete [18]. The depth of carbonation can be determined by treating concrete with phenolphthalein indicator solution. The carbonated portion of the concrete will remain unchanged, while the uncarbonated portion will turn pink [18], [70].

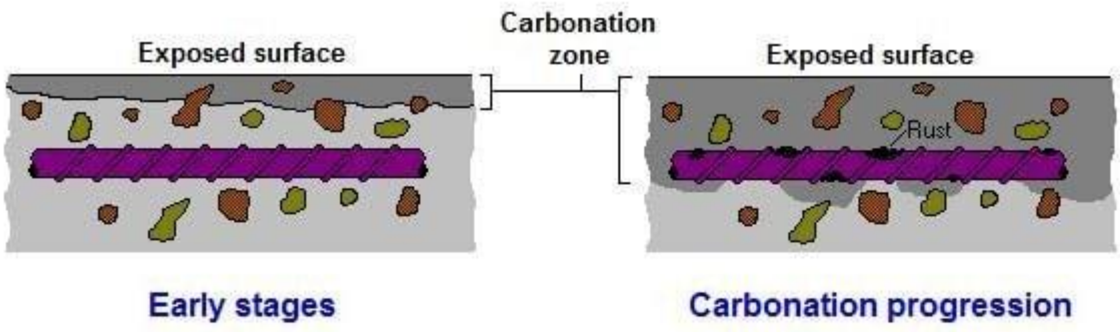

Figure 4. 3: A diagrammatic representation of carbonation induced corrosion [71]

Factors affecting the resistance of concrete to carbonation-induced corrosion

Some of the factors influencing the rate of deterioration of concrete due to carbonation-induced corrosion are briefly discussed below.

- **Humidity:** Humidity is one of the environmental factors that affect the carbonation of concrete. The concrete pores facilitate the diffusion of carbon dioxide into the concrete. The rate of diffusion tends to be slower when the concrete pores are filled with water. According to Bertolini et al. [70], in water-saturated concrete, the diffusion rate of carbon dioxide will decrease with an increase in humidity till the diffusion rate becomes zero. It should be noted that the carbonation rate of a concrete structure may differ at different parts of the structure if they are under different exposure conditions. 60-70% humidity was reported as the most critical humidity values for promoting carbonation [70].

- **Temperature:** An increase in temperature has been found to increase the rate of concrete carbonation [70], [72]. Chen et al. [72] investigated the effect of temperature (10^{o}C, 20^{o}C, and 30^{o}C) on the carbonation depth of concrete. Their result showed that there is a linear relationship between the carbonation depth and temperature. An increase in temperature increases the carbonation depth.

- **The concentration of carbon dioxide:** Another factor that influences the rate of deterioration of concrete due to carbonation is the concentration of carbon dioxide. More carbon dioxide will penetrate into concrete as the concentration of carbon dioxide increases in the atmosphere. The concentration of carbon dioxide (in the atmosphere) in the rural and urban environment is about 0.03% and 0.1%, respectively [70]. This indicates that concrete structures in an urban environment may experience more carbonation than the ones in rural areas.

- **Supplementary cementitious materials:** Although many researchers agree that the incorporation of SCMs in the making of concrete tends to increase the corrosion rate of such concrete due to the depletion of calcium hydroxide in the pozzolanic reaction [73], [74], however, the advantage of SCMs in reducing the permeability of concrete cannot also be overlooked. The study conducted by

Gonen and Yazicioglu [74] indicated that the concrete made with 10% of SF had very little effect on the carbonation of the concrete relative to the plain concrete, while concrete made with 15% of FA has a negative impact on carbonation. In addition, the result of a study conducted by Gettu et al. [67] showed that concrete made with LC^3 and FA30 showed lower resistance to carbonation compared to plain concrete in the long run.

4.5 Alkali-silica reaction

The reaction between the alkalis present in concrete and reactive siliceous components of aggregates to form an expansive gel, alkali-silica gel, is referred to as an alkali-silica reaction (ASR). The gels swell as they attract water by osmosis or absorption from their surrounding or the cement paste. The concrete will crack when the tensile stress due to the swelling is greater than the tensile strength of the concrete [75]. Figure 4.4 shows a typical crack caused by ASR expansion. In South Africa, deterioration of concrete structures due to ASR was first noticed in the 1970s in the Cape Peninsula and environs [75]. The reactive silica aggregates that caused the deterioration are from the Malmesbury Group [75]. Quartz, dacitic, opal (amorphous), tridymite, and amongst others are typical rock materials that consist of reactive silica [18], [52]. Petrographic examination of aggregates can be conducted before concreting. The petrographic examination will indicate the amount of reactive silica present in such aggregates.

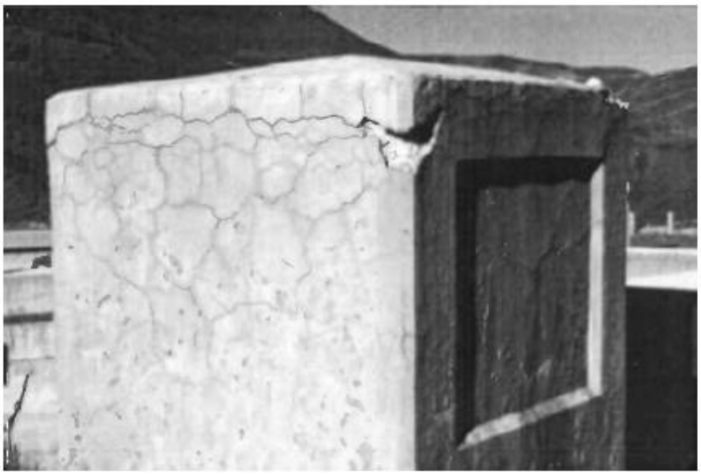

Figure 4. 4: Random cracking in a parapet wall caused by ASR [76].

4.5.1 Factors affecting alkali-silica reaction

For ASR to occur, there must be the presence of aggregates that contains reactive silica, pore solution that contains alkalis, and environmental conditions (such as sufficient moisture content, temperature, and climate) to support the reaction. These three factors are briefly discussed below, together with the influence of SCMs on ASR.

- **Alkalis**: For ASR to occur, there must be sufficient alkalis present in the pore solution of concrete. The main source of alkalis is the cement used in making the concrete. Alkalis usually occur in cement in the form of mixed salt $(Na, K)_2SO_4$ or neutral sulphates (Na_2SO_4 & K_2SO_4). The pH of concrete made with high-alkali cement ranges between 13-14, while that of low-alkali cement ranges between 12.5-12.9 [75]. This means that concrete made with high-alkali cement is more susceptible to ASR than the one made with low-alkali cement, provided that all other conditions are equal.

- **Type of aggregate**: As much as the amount of reactive aggregates in concrete is crucial for the occurrence of ASR, the reactivity of the reactive aggregates also makes a great contribution to the occurrence of ASR. For instance, only 2% of opal in sand caused ASR on some concrete structures, while about 20% (minimum) of Malmesbury metasediment in the sand caused ASR on some concrete structures [75]. Furthermore, it is important to note that aggregates obtained from the same geographical area or quarry can differ in reactivity, as they can contain different amounts of reactive silica, which may be due to their mode of formation. Therefore, it is important to determine the reactivity of aggregates prior to using them either from their service records or through laboratory testing.

- **Environmental factors**: Even if the concrete is made up of reactive aggregates and high-alkali cement, expansive gels will not be deleterious in such concrete unless in the presence of sufficient moisture. Apart from the fact that water promotes the expansion of concrete after the formation of the alkali-silica gel, it also serves as an agent of transportation for various reactive substances [77]. Hence, preventing concrete structures from water can reduce the expansion of concrete due to ASR. Other environmental conditions that influence ASR include temperature, relative humidity, and climate (winter season, due to high amount of rainfall was reported to favour ASR expansion due to availability of moisture in the concrete) [75], [77].

- **Supplementary cementitious materials**: Numerous studies have been conducted to show that various SCMs can mitigate the expansion of concrete due to ASR at different degrees [78], [79]. Concrete made with SF and MK is more efficient in mitigating the negative effect of ASR on concrete compared to plain concrete and FA concrete as per the series of the experiment conducted by Shehata & Thomas [80]. The better performance of SF and MK concrete can be attributed to their higher pozzolanic activity, which promotes denser microstructure, refined pore matrix and alkali binding ability. Meesak & Sujjavanich [81] investigated the effectiveness of incorporating three different SCMs in manufacturing concrete so as to mitigate ASR. Their results showed that concrete made with natural zeolite showed the lowest expansion due to ASR, followed by the concrete made with brick powder and then glass powder.

4.6 Summary

This chapter presents an overview of the durability properties of concrete. Freeze-thaw resistance, sulphate attack resistance, acid attack resistance, corrosion resistance, and ASR are briefly discussed. It should be noted that in order to achieve durable concrete, the designer of such a structure must be aware of the acceptable performance criteria of a concrete structure in a given area. For instance, the probability of chloride-induced corrosion in a marine environment may be higher than the probability of carbonation-induced corrosion in such an area.

Among the factors affecting the freeze-thaw resistance of concrete are entrapped air and the type of aggregate. For concrete to expand freely without causing large internal stresses, moderate air needs to be entrapped in it. Moreover, aggregates with low-shrinking and swelling potential should be employed to accommodate expansion during freezing.

Gypsum and ettringite are the most common expansive products that result from sulphate attacks of concrete. The rate of formation of these products in concrete depends on various factors, including the type of sulphate attacking the concrete, sulphate concentration, type of cement and the use of SCMs. Generally, SCMs consume portlandite in the cement matrix, thereby increasing the resistance of the blended concrete to sulphate attack since less portlandite will be available for the reaction.

Concrete can also be damaged when an acid reacts with the alkaline component of the cement paste, which often form calcium compounds that are leached away. This phenomenon is aggravated if the pH of the acid is low and when the temperature is high. However, blended concrete incorporating SCMs can reduce the attack due to their improved pore structure.

In the chapter, the mechanism of concrete corrosion was explained. For corrosion to occur, there must be electron transfer from the anodic site to the cathodic site on the steel through an electrolyte (i.e., pore solution in the concrete) in the presence of sufficient water and oxygen. The corrosion can either be chloride-induced or carbonation-induced. Without adequate chloride content and carbon dioxide at the surface of the embedded steel, corrosion will not occur. Hence, permeability and porosity of concrete are important factors to determine their resistance to corrosion. In addition, less permeable and denser concrete can be produced with the incorporation of SCMs.

It is also noted in the chapter that expansive gel due to ASR will not be deleterious even when reactive aggregates react with the alkalis in the cement paste, but until the environment is favourable (i.e., availability of adequate moisture content). Due to the pozzolanic activity of SCMs, their incorporation in the manufacture of concrete can considerably reduce the expansion due to ASR.

5. EFFECTS OF BINARY BLENDED CEMENT ON CONCRETE PROPERTIES

Binary blended cements are cements that contain plain Portland cement and one supplementary cementitious material (SCM). In this study, in terms of binary blended cement, the SCMs that are reviewed include: fly ash (FA), silica fume (SF), metakaolin (MK), and rice husk ash (RHA). It should be noted that the available SCMs in the construction industry are more than these four, however, they are beyond the scope of this study. However, the availability of some of the other non-discussed SCMs in this study are relatively low compared to FA, MK, and RHA. For instance, the availability of GGBS across the globe is about 5-10% of the cement produced [82]. The effects of these four SCMs in relation to the mechanical and durability properties of concrete are presented in this chapter.

5.1 Fly Ash

Fly ash (FA) is a by-product of an industrial process of combusting pulverized coal in electric power plants. The dust collection system of the power plants majorly determines the size of FA. Usually, FA is finer than plain Portland cement as the diameter of FA particles ranges between 1 μm – 150 μm [78]. When coal undergoes combustion, FA takes about 75-85% of the total coal ash while the remaining 15-25% are collected as boiler slag or bottom ash [78]. FA majorly consists of alumina, silica, and oxides of iron and calcium. Due to its fineness, amorphous character, and mineralogical composition, FA is regarded as a pozzolanic material. It should be noted that fly ash produced from different power plants may have different properties due to several factors such as type of furnace, degree of pulverization, mineralogical composition of the coal, etc. The leading countries in the production of FA are China, the United States of America, and India, as they are responsible for more than 75% of the world's production [83]. In Africa, fly ash is being produced in many countries such as South Africa, Tunisia, Egypt, Nigeria, Ethiopia, Cameroon, and Ghana [35], [84]–[91]. Moreover, due to the fact that FA is a by-product, it is relatively cheaper than Portland cement [92].

5.1.1 Mechanical properties of fly ash concrete

Strength: Concrete containing 20% & 30% of FA was reported to have a compressive strength of 7.24 MPa and 9.91 MPa, respectively, lesser than the control mix after 3 days of curing. However, the compressive strength of the concrete made with 20% & 30% FA were 4 MPa & 8 MPa higher after a year than that of the control mix, respectively [93]. Moreover, lightweight concrete composing 20% FA showed comparable compressive strength with the control concrete after 7 days; however, the strength increased after 28 days of curing (see Figure 5.1) [94].

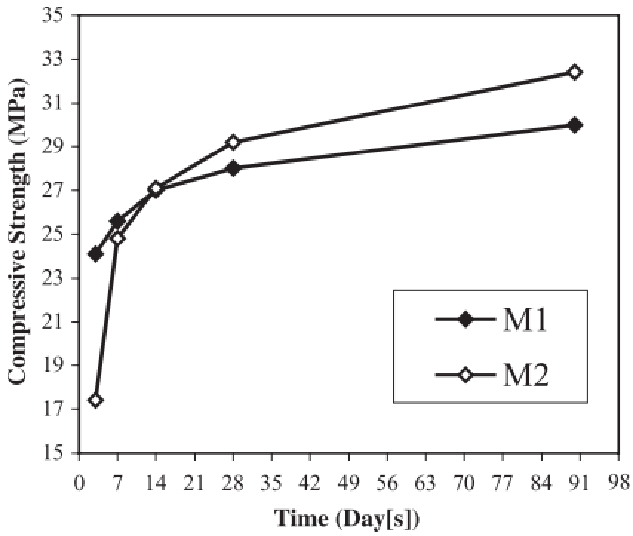

Figure 5. 1: Compressive strength of normal (M1) and FA concrete (M2) [94]

Besides, a study showed that the incorporation of 10% of FA in the manufacture of lightweight concrete resulted in higher tensile strength (6-10%) after 28 days compared to the reference concrete; although a replacement of 20-30% of cement by FA will reduce the tensile strength of the blended concrete by 20% in relation to that of the normal concrete [95]. In another study, it was observed that concrete samples containing up to 60% FA reduced the flexural strength of concrete after 7 & 28 days of curing. However, compared to the control mix, the flexural strength of the concrete samples consisting of 10, 20, and 30% of FA increased by 5, 10, and 2.5%, respectively [96].

The slow strength development of FA concrete can be attributed to its slow hydration reaction while at later ages FA concrete showed higher strength due to the reaction of FA with free lime in the cement paste, which resulted in the formation of additional cementitious materials

Elastic modulus: The compressive strength and elastic modulus of normal concrete and concrete containing 20% FA were investigated [97]. They found that elastic modulus is directly proportional to compressive strength, and the inclusion of FA in concrete reduces the elastic modulus at an early age, whereas it has little or no effect at later ages. 0.59 was adopted for the w/b ratio. From their results, the control concrete has a young modulus of 31 GPa & 35 GPa at 10 and 100 days, respectively, while the FA concrete has a modulus of 22 GPa & 35 GPa at 10 and 100 days, respectively. In a similar investigation, the addition of 30% of FA in concrete with a w/b ratio of 0.65 reduced the elastic modulus of normal concrete by 1.2%, while concrete made with 30% FA with 0.50 w/b ratio reduced the elastic modulus by 1.8% at 28 days [98].

Creep: Hashmi et al. [99] investigated the creep of concrete beams of size 100 × 150 × 1800 mm. The beams were cast with plain cement and 25 - 60% of FA. It was noted that the development of creep deflection was in the range of 60-65% for FA concrete, while it was in the range of 70-75% for plain concrete from day 1 to 180 days. The lower development of creep in FA concrete can be attributed to the higher strength development of FA concrete at later ages. In another investigation, tensile strength creep test was performed on plain concrete and 20% FA concrete. It was observed that the creep strain & specific creep for the two types of concrete samples were similar at all ages ranging from day 1 to 14 days for a stress/strength ratio of 30% [100].

Shrinkage: A laboratory experiment was conducted by Atis [101] to investigate the strength & shrinkage properties of concrete made with 100% of Portland cement, 50% and 70% FA by weight of cement. The w/b ratio for the concrete samples ranges from 0.28 to 0.34. Their results showed that concrete with 70% FA has the lowest shrinkage strain at 6 months of drying time (294 microstrain), while the control mix has the highest drying shrinkage value of 554 microstrain. However, a contrast behaviour was reported by Kate & Murnal [102] in their research. They found that the inclusion of 10, 25, 40, and 70 % of FA in concrete gives 32.1, 35.0, 43.9 and 81.7% more shrinkage strain compared to plain concrete under the same environmental conditions (at 28 days). It could be inferred from the previous studies that the inclusion of FA in concrete may increase or reduce the shrinkage of concrete, depending on the pore sizes refinement, water/binder ratio and cement content.

5.1.2 Durability properties of fly ash concrete

Freeze-thaw resistance: Islam et al. [103] conducted a study that aimed to evaluate the effects of FA on three grades of concrete exposed to plain & seawater over 360 cycles, it was shown that the concrete samples containing 30 and 40% of FA have their compressive strength to be 27-34% higher in seawater and 16-20% higher in plain water compared to the control concrete. Although seawater is not advisable for curing reinforced concrete as it may accelerate reinforcement corrosion, however, the better performance of the plain concrete exposed to sea water can be attributed to the presence of calcium chloride in the sea water, which promotes the formation of a denser pore structure of the concrete [104]. Similar results were obtained by [105] in their study. They found that air-entrained & non-air entrained concrete of 0.46 w/b composing of 20% FA produced surface scaling results of 12.5 and 12% lower than that of the normal concrete, respectively. It is also noted that air-entrained concrete performed better than the non-air entrained concrete since the entrained air provides a medium for the water to escape upon freezing.

Sulphate attack resistance: Dhiyaneshwaran et al. [106] made 6 different mixes of self-compacting concrete with 0, 10, 20, 30, 40, and 50% of FA. The concrete samples were immersed in 5% sodium sulphate solution for 28, 56 & 90 days, and their sulphate resistance was examined through their weight loss. It was observed that the replacement of cement up to 40% by FA increases the sulphate resistance of concrete at all ages (see Figure 5.2, the blue, red, and green bars show the average reduction in weight at 28, 56, and 90 days, respectively). This is similar to the results obtained by Srinivas & Rao [107], where they immersed normal & FA concrete in a solution of magnesium and sodium sulphate solution for 15, 45, 75 & 105 days. They found that the loss in compressive strength & weight of FA concrete is lesser than that of normal concrete. For instance, the loss in compressive strength for FA concrete immersed in $MgSO_4$ ranges between 3.44-12.52%, whereas that of normal concrete ranges between 5.58-18.2% [107].

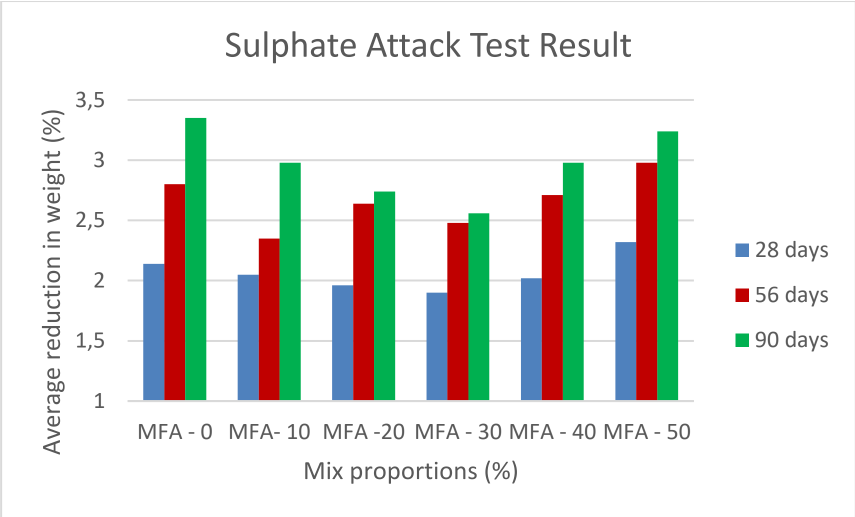

Figure 5. 2: Weight reduction due to sulphate attack [106]

Acid attack resistance: The effect of incorporating FA in the manufacture of steam-cured concrete in relation to acid attack resistance was investigated by Aydin et al. [108]. After 60 days of immersion in 5% sulphuric acid, the weight loss percentages dropped from 5% (normal concrete) to 3.3% (70 % FA concrete). The better resistance of FA concrete can be attributed to its denser pore structure, which inhibit the movement of acid into the concrete. However, concrete with more than 30% FA showed greater strength loss than the normal concrete.

A study was conducted by Dhiyaneshwaran et al. [106] to evaluate the durability properties such as acid attack resistance of concrete samples made with 0, 10, 20, 30, 40, and 50% of FA. After curing for 28 days, the specimens were exposed to 1% sulphuric acid for 28, 56 & 90 days. The concrete with 30% FA exhibited the lowest percentage weight loss.

Chloride-induced corrosion: An investigation was carried out by Choi et al. [109] on normal concrete and concrete cylindrical samples containing 20% of

FA to evaluate their resistance to chloride-induced corrosion. Three normal & FA concrete samples were cast with w/b of 0.5, 0.35, and 0.31, respectively. After immersing the concrete samples into 3.5% NaCl solution for 140 and 250 days, the investigation results showed FA concrete with 0.35 and 0.31 w/b have higher resistance to chloride-induced corrosion than the normal concrete at both ages. For instance, the concrete resistance at 140 and 250 days for normal concrete with 0.35 w/b were 7262 ohm-cm^2 and 11,023 ohm-cm^{2,} respectively, while that of FA concrete were 10,162 ohm-cm^2 and 13542 ohm-cm^2, respectively.

In another study [110], concrete samples were prepared with 100% plain cement and FA content up to 60% of the weight of cement. Their investigation showed that concrete made with 40% of FA showed better resistance to chloride-ion penetration as their total charged passed were 2775 and 885 coulombs, respectively, compared to 3443 and 2916 coulombs for the normal concrete at 28 and 90 days of testing, respectively. However, concrete with more than 40% of FA showed lower resistance compared to the normal concrete [110]. This is because the combination of 60% PC and 40% FA will produce blended concrete of lower strength and low-quality pore structure, as the content of the cement clinker is relatively low. The reactivity of the clinker phase is often greater than that of the pozzolanic phase [111].

Carbonation-induced corrosion: The carbonation resistance of reinforced concrete samples made with 0, 20, 35, and 45% of FA was investigated by Bouzoubaa et al. [112]. After curing for 10 days, the samples were exposed to an environment of 3% CO_2, 65% relative humidity, and 23^{o}C for 140 days. The results showed that carbonation increases with an increase in the content of FA. This indicates that concrete with 50% FA has the highest carbonation depth. A similar experiment was conducted by Kumar et al. [113] as they evaluated carbonation resistance of concrete samples containing 0-70% of FA at 14, 28, 42, 56, and 70 days. It is observed from their results that carbonation resistance of concrete decreases with an increase in exposure time and FA content. For instance, the carbonation depth of concrete without FA at 70th day of exposure to 4.0% CO_2 environment (55% relative humidity and 20°C) was 11mm, while that of concrete containing 50% FA was 18mm.

Generally, the decrease in carbonation resistance of FA concrete can be attributed to the reduction of calcium hydroxide in concrete as the FA content increases, which subsequently lowers the alkalinity of concrete.

Alkali-silica reaction: In the investigation of Shehata & Thomas [114], 19 concrete prisms were cast with 0, 15, 20, 25, 30, 40, 45, 50, and 60% of FA content (by weight of cement). The alkali content of the specimens was raised to 1.25% Na_2O. After two years, the expansion of the prisms was evaluated. It was found that concrete prisms made with 50 and 60% FA showed insignificant expansion (less than the expansion limit of 0.04%) as expansion in excess of 0.04% was assumed to cause damage (i.e., cracks).

Thomas et al. [115] cast 45 concrete specimens with reactive aggregates and different levels of high-alkali cement to assess the effectiveness of incorporating FA in concrete in order to reduce expansion due to Alkali-silica reaction. The concrete samples were exposed to an outdoor exposure site for 18 years. It was observed that all the concrete specimens with 100% plain cement indicated significant expansion and cracking within 5-10 years of exposure, whereas concrete containing 25 and 40% FA exhibited very little expansion, which is not detrimental to the concrete samples over the period of 18 years. The effectiveness of FA concrete in reducing expansion due to ASR can be attributed to the reduction of pH of the concrete due to alkali binding [116]. Alkali binding capacity refers to the ability of concrete to restrict the movement of alkalis to participate in any reaction. Thus, the availability of alkalis for ASR reaction would be significantly reduced. It should be noted that the alkali binding capacity of blended concrete is influenced by the pozzolanic reaction, as hydrates with low Ca/Si are produced [117].

5.2 Silica Fume

Silica fume (SF) is a by-product from the manufacture of ferrosilicon and silicon alloys in electric arc furnaces. When high-purity quartz is subjected to about 2000°C, the quartz is reduced to silicon and produces silicon dioxide vapor. Silica fume is formed when the silicon dioxide vapor oxidizes and then condenses at a lower temperature [118]. Silica fume particles are usually spherical in shape and are ultrafine materials with an average mean size of 0.15 µm [118]. It should be noted that the content of silicon dioxide in silica fume depends on the alloy that is being manufactured. For instance, the production of silicon alloy results in the production of SF with about 74% - 84% of SiO_2 content, while the production of 50% ferrosilicon alloy produces SF of about 74% - 84% SiO_2 content [119]. The global production of SF was estimated to be 1.5 Mt per year in 2016 [83]. Furthermore, China is the highest producer of silica fume in the world. In Africa, countries such as South Africa, Egypt, and Libya are producers of silica fume [120]–[123]. Although SF can be more costly than PC, but its numerous benefits over PC cannot be overlooked [124].

5.2.1 Mechanical properties of SF concrete

Strength: In terms of w/b ratio, two concrete mixes (i.e., 0.35 and 0.3) were employed in casting normal concrete and concrete containing 10 and 15% SF by weight of cement. For the two mixes, the compressive strength of SF concrete samples was higher than the control mix. For instance, after 28 and 56 days of casting of concrete made with 0.3 w/b, the compressive strength of normal concrete, 10% SF concrete and 15% SF concrete were 70 & 77 MPa, 73 & 79 MPa, and 82 & 85 MPa, respectively [125]. A similar investigation was conducted by Koul et al. [126] as they replaced 10 and 15% of cement by SF to determine its compressive strength and that of the control mix. According to their results, after 7 and 14 days of casting, concrete with 10 and 15% SF was 38 & 42.5% and 30.3 & 46.7% higher than the control mix, respectively.

Concrete samples containing 0-25% (at an interval of 5% increment) of SF were made with different w/b ratio of 0.26, 0.30, 0.34, 0.38. The concrete samples were moulded into cylinders of size 150 × 300mm. The results showed that the tensile strength of concrete reduces with an increase in the w/b ratio regardless of the content of SF. After 28 days of curing, the tensile strength was optimum at a 15% replacement level for all the concrete mixes. The addition of more than

15% of SF in concrete either reduces the tensile strength or shows no difference as compared with that of 15% [127].

In the same vein, concrete prisms were cast with PC and SF with the replacement level of 3, 6, 9, 12, and 15% to investigate their flexural strengths. The w/b ratio was kept constant at 0.36 throughout the investigation. The results revealed that flexural strength of concrete with 3, 6, 9, 12, and 15% SF were 2.5, 7.4, 12.7, 14.5, and 11.3%, respectively, higher than the normal concrete. Their research indicates that the optimum percentage of replacement of cement with SF is 12% in relation to the flexural strength of concrete [128].

Elastic modulus: Deformation properties such as elastic modulus, shrinkage, and creep of concrete with and without SF were studied by Mazloom et al. [129]. Four cylindrical concrete samples were cast, with each consisting of 0, 6, 10, and 15% SF by weight of cement. The secant modulus of elasticity of the samples was evaluated after loading them on the 7^{th} and 28^{th} day of curing. The stress/strength ratio ranged between 0.14 – 0.22. It was observed that the modulus of elasticity of concrete increases with an increase in the content of SF. The 15% SF concrete had a modulus of elasticity that was 9.7, 6.9, and 3.9% higher than the concrete with 0, 6, and 10% SF, respectively.

Four different types of concrete, with each containing PC; PC & SF; and PC, & ground pumice (GP) were cast to investigate their elastic modulus and compressive strength [130]. From the results of the elastic modulus test, it was observed that concrete containing 15% of SF exhibited the highest elastic modulus amongst every other mix. The control specimen showed an elastic modulus of 29.69 GPa which was about 31% lesser than that of the 15% SF concrete (see Figure 5.3).

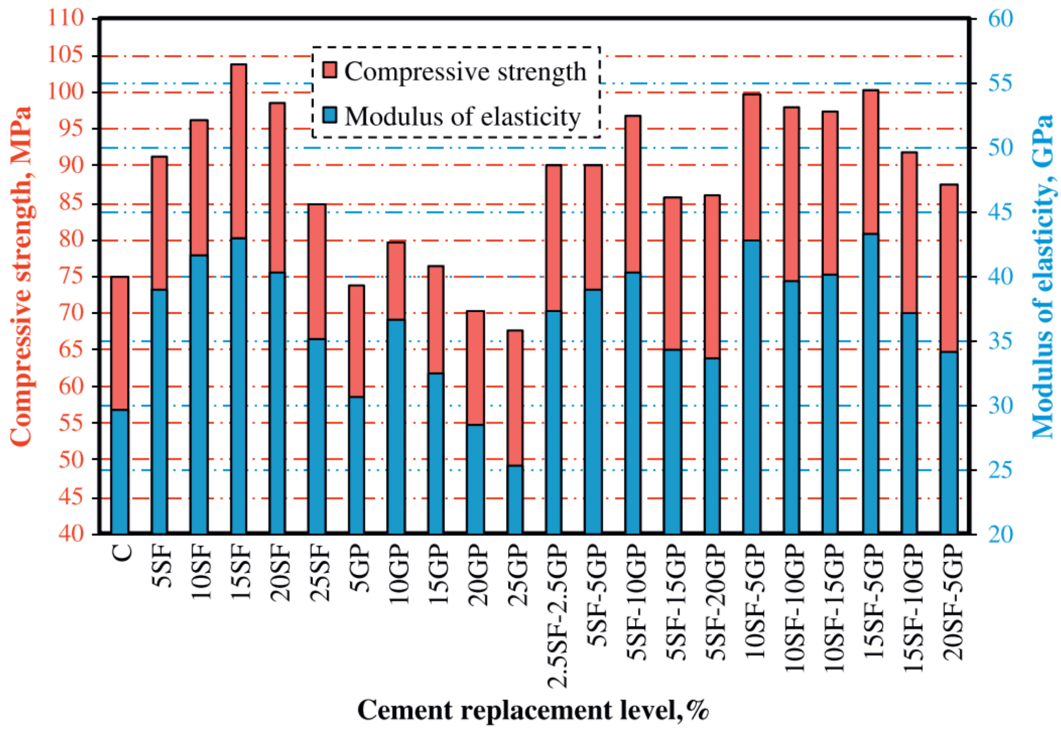

Figure 5. 3: Compressive strength and modulus of elasticity of SF and other blended concrete [130]

Creep: Mazloom et al. [129] studied the effect of creep on normal concrete and concrete containing 6-15% SF by weight of cement. Four concrete specimens of 80 × 270mm were cast and subjected to a sustained load of 10 MPa for 7 and 28 days. The results of their investigation indicated that the inclusion of SF in concrete reduces the creep of such concrete. For instance, the creep strain of the normal concrete reduces by 14.3, 22.9, and 29.9% on adding 6, 10, and 15% SF, respectively to it after 7 days of loading.

Furthermore, deformations of concrete in terms of shrinkage and creep were evaluated by Al-Khaja [36]. Normal concrete and concrete containing SF with a percentage replacement level of 10% were cast at a w/b ratio of 0.38. The concrete prisms were cured in water for 2 days and subsequently subjected to a sustained load. The stress/strength ratio of 0.33 was adopted for the creep test. The concrete prisms were loaded for up till 33 days. Based on their results, the inclusion of SF in the manufacture of the concrete prisms reduced the creep strain by 3 and 18.5% at 14 and 33 days, respectively.

Shrinkage: Total shrinkage that comprises of autogenous and drying shrinkage of normal and SF concrete specimens were evaluated [129]. The concrete samples that were tested were made with 0, 6, 10, and 15% SF. 0.35 was adopted as the w/b ratio. The specimens were tested after 7 days of casting. The results showed that the autogenous shrinkage increases significantly with the addition of SF such that the autogenous shrinkage was 198, 231, 264, and 297μm for concrete containing 0, 6, 10, and 15% SF. The higher increase in the autogenous shrinkage of the SF concrete can be attributed to the refinement of pore size distribution, which subsequently lead to increased capillary tension and thus, more contraction of the cement paste. However, the drying shrinkage reduces over a long period of time so that the total shrinkage of the normal concrete was comparable with that of the SF concrete samples after 587 days.

After curing 12 concrete prisms in water for 28 days, the drying shrinkage of the concrete prisms was evaluated by Carette & Malhotra [131] by placing them in a dry air environment with 50% RH and 23^{o}C for up to 84 days. The concrete prisms were made with 0, 5, 10, 15, 20, and 30% of SF and a w/b ratio of 0.4. Based on the recorded data, the drying shrinkage of the normal and the blended concrete were comparable, as shown in Figure 5.4. For example, the drying shrinkage of the normal concrete was 330μm while that of concrete with 15, 20, and 30% SF were 316, 330, and 323 μm, respectively. The low diffusivity of SF due to its refined pore structure could have reduced the rate of water loss, thereby explaining the better or comparable performance of SF concrete in terms of drying shrinkage compared to the normal concrete.

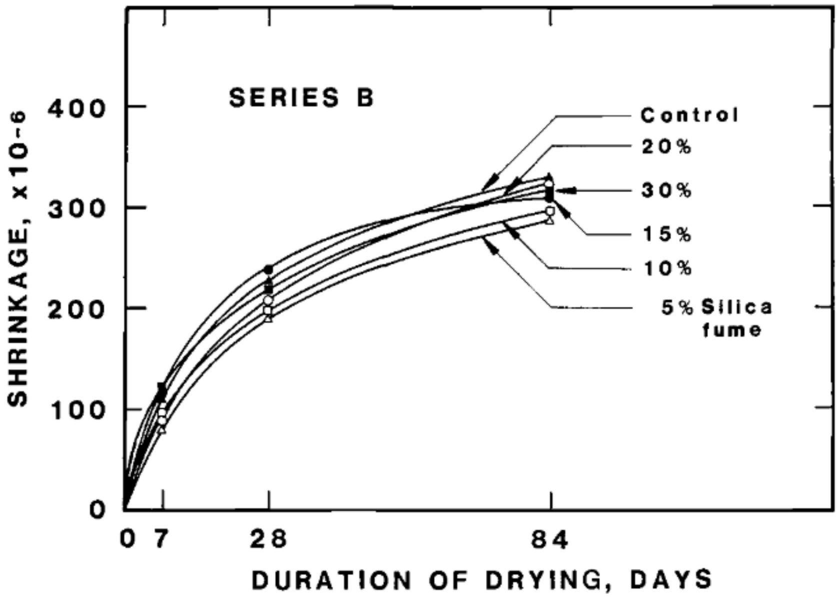

Figure 5. 4: Drying shrinkage of normal and SF concrete [131].

5.2.2 Durability properties of SF concrete

Freeze-thaw resistance: Yin et al. [132] studied the effects of adding SF to concrete in relation to the concrete's resistance to freeze-thaw cycles. The concrete mixes were made with 0, 6, 9, and 12% SF by weight of cement. Concrete prisms of 100 × 100 × 400 mm were cast, cured, and subjected to freezing-thawing environment up until 350 cycles. Subsequently, the relative dynamic elastic modulus of the samples was evaluated. The results indicated that concrete containing 9% SF exhibited the highest relative dynamic modulus, which was 35.4, 3.1, and 8.5% higher than those containing 0, 6, and 12% SF.

In another similar research, the freeze-thaw resistance of ultra-high performance concrete containing SF and other SCMs, which were separately added, was investigated by Lu et al. [133]. The percentage replacement of cement with SF were: 2, 4, 6, 8, 10, 12 13, and 14%. The w/b ratio was kept constant at 0.19 for all the concrete mixes. All the samples were subjected to up to 300 cycles. It was observed that the mass-loss rate reduces as the SF content increases such that the mass loss rate of the normal concrete was 0.6% while that of concrete incorporating 14% SF was 0.4%. The positive effect of SF on concrete's resistance to free-thaw actions can be attributed to its capillary pore filling capabilities, which thus provide a denser pore structure.

Sulphate attack resistance: The effects of incorporating SF and nano-silica on sulphate attack resistance of concrete were studied separately and synergically by Li et al. [134]. In the study, PC was replaced by SF at a replacement level of 5%. With an increment interval of 0.05, the w/b ratio ranged between 0.30 – 0.45. Six concrete cubes were prepared for each concrete mix. Three of the cubes were cured in water for 26 days and subsequently air-dried for the next 2 days before placing them in a sulphate attack machine (15 hours in 5% sodium sulphate solution and 9 hours drying daily) for 90 days. The other three cubes were cured in water for 28 days before they were air-dried for 90 days. The difference in compressive strength of the two types of cubes was evaluated. The results showed that the normal concrete exhibited the highest loss in compressive strength such that its compressive strength loss was 18% higher than that of SF concrete with a w/b ratio of 0.4.

A similar investigation by Lee et al. [135] also confirmed the effectiveness of SF in enhancing sulphate attack resistance of concrete. In their research, concrete samples were prepared with 0, 5, 10, and 15% of SF by weight of cement. The specimens were cured at normal room temperature for 7 days in which half of them (in numbers) were immersed in a 5% solution of sodium and magnesium sulphate for up till 510 days. Compressive strength loss and visual examination were adopted to assess the sulphate attack resistance of the specimens. It was observed that the compressive strength loss of all the samples containing SF (exposed to Na_2SO_4) was lesser than that of the normal concrete at all ages. For instance, the compressive strength of specimens containing 0, 5, 10, and 15% SF were 21.4, 61.6, 58.1, and 69.0 MPa at 510 days.

Acid attack: Durability investigation on acid attack resistance of concrete with and without SF was investigated [136]. Concrete specimens were manufactured with 0, 5, 10, 15, and 20% SF and cured for 28 days in water. Subsequently, the concrete cubes were placed in a solution of 5% HCl for 30 days. The weight loss and compressive strength of the cubes were evaluated. The results showed that concrete containing 10% SF exhibited the lowest weight loss and compressive strength loss (see Figure 5.4).

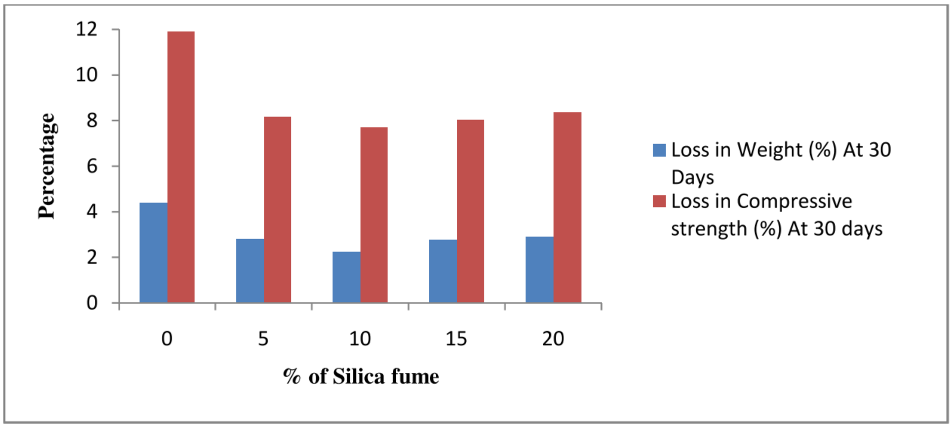

Figure 5. 5: Percentage loss in weight and compressive strength (%) [136]

Furthermore, concrete cubes were cast with two different binder compositions, 100PC and 95PC & 5%SF, to investigate the resistance of such concrete cubes to acid attack [137]. River and manufactured sand were employed at different proportions as fine aggregate. The concrete cubes were immersed separately in a solution of 1% HCl and H_2SO_4 for 28 days. It was observed from the data reported that SF concrete with 45% manufactured sand showed the lowest loss in weight and compressive strength. For instance, the compressive strength loss for the normal concrete was 18.95%, while that of SF concrete of the same aggregate proportion was 6.67%. It was also noted that the weight loss and compressive strength loss of concrete exposed to H_2SO_4 were more than those of HCl.

Chloride-induced corrosion: Jung et al. [138] studied the effect of chloride-induced corrosion on concrete with and without SF. The mixes were made with 0 and 10% of SF. The mixing water for the production of the concrete samples contained 0.2- 3.0% of NaCl by weight of the total binder for each mix. The samples were cured in a polythene bag for 28 days to ensure chlorides are not leached out. The samples were subjected to a wetting & drying condition for 20 cyclic times. Each cycle took 7 days (3 days for wetting and 4 days for drying). Subsequently, the corrosion potential evaluation results stated the corrosion potential of the normal mix to be between -300 to – 583mV while that of the SF concrete ranged from -275 to – 450mV, indicating the effectiveness of SF concrete in mitigating chloride-induced corrosion.

A study on the electrical resistivity of concrete containing 100% PC and 90% PC & 10% SF was conducted by Manera et al. [139]. The w/b ratio was kept constant at 0.6 for both mixes. Before embedding steel bars in the concrete, they were immersed in a solution of saturated calcium hydroxide for 15 days in order to enhance passivity. After casting and curing the specimens for 3 months, a conductivity metre was employed to measure the electrical conductance of the specimens. The resistivity values for each specimen were determined from the measured data. The resistivity values of the normal concrete was 15 ohms while that of the SF concrete was 70 ohms at 50^{o}C. Concrete with higher resistivity showed higher resistance to chloride ingress. Pore structure refinement of SF concrete can be said to be the major factor contributing to its effectiveness in mitigating chloride induced corrosion.

Carbonation-induced corrosion: Blended binder systems were made with SF & PC and MK & PC to investigate carbonation resistance of concrete [140]. The percentage of replacement of cement by SF ranged from 0-25%. The w/b ratio adopted in the research ranged between 0.45 - 0.79. The concrete samples were subjected to an accelerated carbonation condition (4% CO_2, RH 55%, and 20^{o}C) for up to 20 weeks. Based on the reported data, carbonation depth increases with an increase in w/b ratio regardless of the composition of the binder system. It was also observed that concrete containing 10% or less SF content exhibited almost the same carbonation depth as that of the normal concrete for concrete mix with a w/b ratio of 0.52 or less. The negative effect of portlandite consumption on carbonation resistance of the SF concrete and the positive effect of the refined pore structure of SF concrete due to the pozzolanic reactions seem to be of the same magnitude in this investigation, as SF concrete exhibited comparable carbonation depth with that of the normal concrete.

Furthermore, PC was replaced by 5, 10, and 15% SF by weight of binder to investigate the carbonation resistance of normal and SF concrete [141]. The fine aggregates employed in the study were also separately replaced by 5, 10, and 15% SF by weight of total aggregates. After casting the concrete specimens, they were placed in a laboratory environment for stabilization of internal humidity for 30 days. Subsequently, they were placed in an environment of 3% CO_2 at RH of 100% and a temperature of 25°C for 100 days. The carbonation depth was determined with the aid of the phenolphthalein indicator. According to their results, the carbonation depth increases a little with an increase in the content of SF in the binder system while it decreases with an increase of SF content in the aggregate system. Based on previous studies, the carbonation resistance of SF concrete containing 10% or less SF content is always influenced by the w/b ratio rather than the effect of portlandite consumption caused by the inclusion of SF. This indicates that concrete with a low w/b ratio (low porosity) and SF content of 10% or less give comparable resistance to carbonation in relation to the normal concrete.

Alkali-silica reaction: Boddy et al. [142] studied the effect of including SF containing 88.4% of silica in the production of specimens tested for ASR. Cement was replaced at percentage levels of 4, 8, and 12% by SF. For all the concrete mixes, reactive aggregates and high-alkali cement were used. An accelerated mortar bar test was adopted for the investigation of expansion due to ASR. After 24 hours of casting, the bars were heated in water, and their initial lengths were recorded after 2 days. Subsequently, the specimens were then immersed in a solution of 1M NaOH (preheated to about 80°C). The lengths of the specimens were recorded after 4, 6, 8, 10, 12, and 14 days. Although all the SF mixes showed lesser expansion than the normal mix, however, only mortar made with 8 and 12% of SF showed expansion of 0.10% or less, which is the acceptable expansion limit for most standards.

Furthermore, various types of SF (densified, undensified, pelletized, and slurried) were blended with PC to produce specimens used to access the resistance of such mixes to expansion due to ASR by Boddy et al. [143]. Four different mixes were prepared, with each mix containing 0, 4, 8, and 12% SF by weight of cement, respectively. Accelerated mortar bar and concrete prism expansion were conducted to evaluate the resistance to expansion due to ASR. The alkali content of the cement was raised to 1.25% Na_2O_{eq}. The concrete prism expansion test after 1 year revealed that concrete made with 4% densified and pelletized SF showed lower expansion than the normal concrete while the other two types of SF performed showed higher expansion. It should be noted that all expansions exhibited by concrete containing 4% SF exceeded the expansion limit of 0.04%, indicating their unsuitability to mitigate expansion due to ASR. However, all the concrete mixes with 8 and 12% SF showed expansion lesser than that of the normal concrete and were also below the expansion limit of 0.04%

The high reactivity of SF, which facilitates the consumption of portlandite during the hydration process, thereby reducing the pH of the pore solution and decreasing the CaO/SiO_2 ratio, has been attributed to the effectiveness of SF concrete in mitigating expansion due to ASR [144].

5.3 Metakaolin

Metakaolin (MK) is a pozzolanic material that is formed when kaolinitic clay is subjected to a temperature between 650°C – 900°C [83]. This process of heating is regarded as the calcination of kaolin clay. Kaolinite is the mineral present in kaolin clay. Water is being driven off (dehydroxylation) as kaolin clay is subjected to heating to produce MK of irregular structure and high surface area. It is important to note that the manufacture of 1 ton of MK contributes significantly to the reduction of CO_2 in the environment in relation to plain Portland cement production. Production of 1 ton of plain cement releases about 650kg of CO_2 while that of MK releases about 175kg of CO_2 [145]. Proper attention should be given to the calcination process of kaolin clay to avoid over-heating, resulting in the formation of mullite, which is not reactive [78]. Silicon and aluminium oxides are the major chemical constituents of MK [146], [147]. According to a study conducted by [148], there are more than 290 deposits and occurrences of kaolinite clay in Africa, with the possibility of thousands of such deposits in Africa as clays are widespread in the continent. In terms of economic implication, MK cost of production is significantly lesser than that of Portland cement. However, due to limited production plants globally, which can be linked to poor response from the government and the construction industry, MK costs more than PC in the market [147].

5.3.1 Mechanical properties of MK concrete

Strength: In order to produce high-performance concrete, Kim et al. [149] prepared concrete samples with 100% PC and blends of PC and MK at 5, 10, 15, and 20% replacement levels, and their strength properties were investigated. The w/b ratio of all the concrete mixes was 0.25. The compressive strength test was conducted at 1, 3, 7, 28, 56, and 91 days of casting the specimens. For all ages of testing, concrete samples with 10 and 15% MK had the same and highest compressive strength, which, for example, was 6.6, 4, and 5.3% higher than those with 0, 5, and 20% MK at 91 days.

Shehab El-Din et al. [150] investigated on the mechanical properties of concrete incorporating MK. In their study, normal concrete with 100% PC and MK concrete with 10, 15, 20, 30, 40, and 50% MK were prepared and cured. 0.32 w/b ratio was employed for the investigation. The compressive strength of the concrete cubes was evaluated at 3, 7, & 28 days and the results showed that there was an increase in compressive strength with an increase in MK content for all the concrete specimens at 28 days except the concrete with 50% MK which had lower compressive strength compared to the normal concrete. More importantly, 15% replacement level was found to be the optimum dosage for MK concrete as concrete containing 15% MK exhibited the highest compressive strength all at ages of testing. The cubes were also tested for splitting tensile strength at 28 days. According to their results, the tensile strength of the cubes increases with an increase in MK content up to 30% replacement level; any further increment in MK content decreases the tensile strength of the sample relative to the normal mix. It was also noticed that the optimum percentage for replacement of cement with MK was also found at 15% as the tensile strength of concrete with 15% MK was 8.4, 4, 6, 8.1, 19.6, 24.5% higher than those with 5, 10, 20, 30, 40 and 50% MK, respectively.

PC was separately replaced by MK and SF in order to compare their effects on concrete properties [149]. The percentage of replacement of cement with MK were 5, 10, 15, and 20%. All the concrete mixes were cast with a w/b ratio of 0.25. The flexural test results indicated that flexural strength of concrete increases with the inclusion of MK up to 15%; however, it decreases with the 20% replacement level. For instance, at 56 and 91 days of testing, the flexural strength of concrete containing 0, 5, 10, 15 and 20% MK were 7.5 & 7.63MPa, 7.62 & 7.64MPa, 7.65 & 7.7MPa, 7.65 & 7.7MPa, and 7.55 & 7. 62 MPa, respectively.

Elastic modulus: 6 concrete mixes were prepared by Murthy et al. [151] to investigate the influence of MK on the elastic modulus property of concrete. Each of the mixes contained 0, 7.5, 10, 12.5, 15, and 17.5% of MK by weight of the binder, respectively. 3 concrete cylinder samples were made for each mix. All the concrete samples were cast and cured for 28 days before subjecting them to axial compression. Subsequently, their modulus of elasticity was evaluated from the stress-strain relationship. The results showed that concrete with 10% MK had the highest modulus of elasticity as its value was 21, 9.3, 8.6, 1.9, and 33.4% than those with 0, 7.5, 12.5, 15, and 17.5%, respectively.

Mardani-Aghabaglou et al. [152] studied the comparative effects of incorporating MK, FA, and SF in mortar in relation to their elastic modulus. 10% replacement level was adopted for the three SCMs. All the mixtures were cast with a w/b ratio of 0.48. The specimens were subjected to ultrasonic pulse velocity (UPV) tests after 7, 28, 90, 180, and 300 days and their elastic modulus were calculated based on the measured data from the UPV test. It was observed from the results that the MK specimen showed higher elastic modulus than the normal concrete and the FA concrete at all ages of testing.

Creep: Creep behaviour of concrete containing 0, 5, 10, and 15% MK by weight of cement was examined by [153]. W/b ratio was kept at 0.28 throughout the investigation. The cylindrical concrete specimens of size 267 × 76 mm were cast and cured for 28 days at normal room temperature. After curing, the cylindrical specimens were subjected to a sustained load corresponding to 0.2 stress/strength for up to 200 days. Based on the results, basic and total creep of concrete reduces with an increase in cement replacement by MK. For example, the basic and total creep, at 100 days of the normal concrete was reduced by 57.7 & 50 % and 59 & 55.8% for concrete with 10 and 15% MK, respectively (see Figure 5.5).

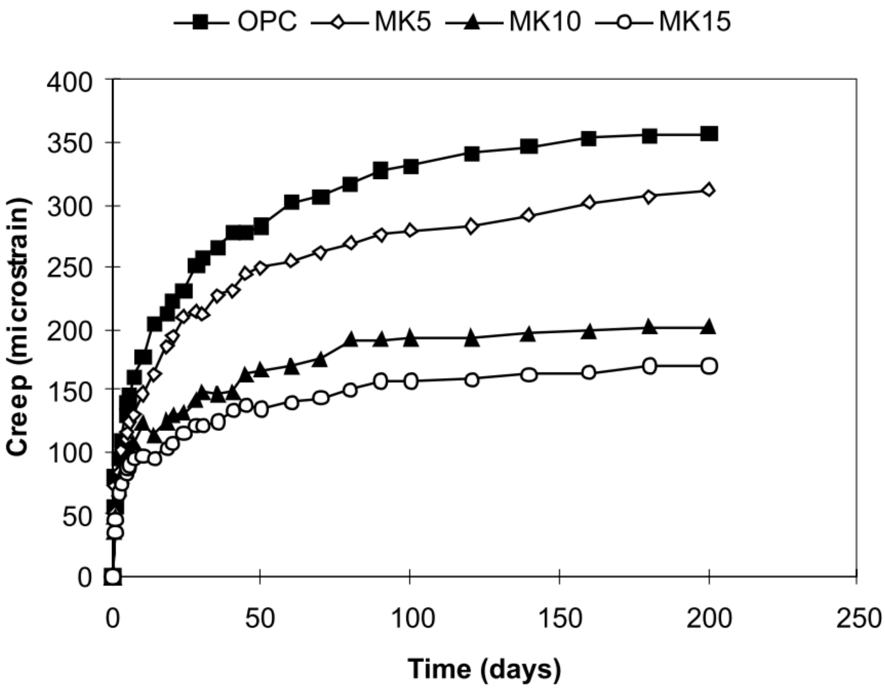

Figure 5. 6: Effect of MK on creep of blended concrete [153]

Furthermore, an experiment was conducted by Vaishnavi [154] to investigate the effect of incorporating MK in concrete in terms of the concrete's creep and shrinkage behaviour. The MK concrete samples were made by partially replacing PC at replacement levels of 5, 10, 20, and 30%. The specimens were subjected to a creep test after curing them for 28 days. The creep test was conducted for up to 200 days. The results revealed that creep strain reduces as the content of MK increases, an occurrence that can be attributed to the less availability of water in MK concrete due to high pozzolanic reaction. The creep strain of concrete with 15% MK was about 64.3% lesser than that of the normal mixture at 200 days. Based on the previous studies, the positive influence of MK concrete on creep behaviour can be attributed to the enhanced aggregate-paste interface, denser pore structure, and stronger concrete matrix of MK mixture.

Shrinkage: Autogenous shrinkage of normal and MK cement paste was reported in the study of Gleize et al. [155]. 4 MK replacement levels of 5, 10, 15, and 20% were employed for the study. The cement pastes were prepared at two w/b ratios of 0.3 and 0.5. The specimens were covered with an aluminium sheet to prevent weight loss immediately after casting. The autogenous shrinkage (change in length) was determined at specified ages (30, 60, 90, 120, and 150 days) relative to 24 hours after casting. It was observed that the autogenous shrinkage of MK specimens, for the mix with a w/b ratio of 0.3, at an early age were slightly higher than that of the plain mix (without MK content) while that of the 0.5 mixes were comparable at an early age. However, for all the specimens, the autogenous shrinkage of MK specimens was lesser than that of the control mix at later ages. Specimens containing 20% MK were found to show the least shrinkage, indicating 20% as the optimum replacement level. For instance, at 150 days for samples with 0.5 w/b, the autogenous shrinkage of samples with 20% MK was 59.4% lesser than that of the plain mix.

Some concrete properties such as drying shrinkage, strength, and permeability were evaluated [43]. The concrete specimens were manufactured with 100%PC, 95% PC & 5% MK, and 85% PC & 15% MK. The w/b ratios adopted were 0.25 and 0.35. For each mix, 4 prisms of size 70 × 70 × 280 mm were cast for the drying shrinkage investigation. The specimens were exposed to drying conditions (23^{o}C and 50% RH) for up to 42 days after curing in water for 1 day. The drying shrinkage strains of the normal and MK concrete were similar at early ages (for about 1 week); however, the difference was clear at later ages. Considering the shrinkage values at 42 days of drying, the values were 673, 452 and 392 microstrain, for samples containing 0, 5 and 15% MK, respectively, showing 15% as the optimum replacement percentage for MK. The weight loss due to drying was also found to follow the same trend. In general, the optimum replacement level of MK could be 15%, in which further increment may show lesser performance compared to that of the optimum dosage. It should also be noted that the reactivity of SCMs is often lower than that of the cement clinker phase; hence, the excessive higher dosage of MK, in this case, can produce undesired final blended concrete [111].

5.3.2 Durability properties of MK concrete

Freeze-thaw resistance: The freeze-thaw resistance of concrete containing MK as a partial replacement for PC was investigated by Nas et al. [156]. The MK replacement levels were 5, 10, and 20%. Throughout the investigation, a 0.6 w/b ratio was adopted. The samples were cured in water for 28 days, after which they were subjected to repeated cycles of freezing and thawing up to 100 times (100 cycles). One complete cycle lasts for 12 hours. The change in mass of each sample was evaluated. Concrete with 10% MK exhibited the least mass loss of 2%, while those of the normal concrete and concrete with 5 and 20% MK were 4, 2.5, and 3%, respectively.

In their research [157], loss in mass and elastic modulus were used as indices of freeze-thaw resistance. This was conducted on concrete containing 0, 10, 12.5, and 15% MK by the weight of cement. Two w/b ratios of 0.47 and 0.37 were used in the research. After curing the specimens, they were subjected to repeated freezing and thawing cycles up to 300 cycles. It was evident from the results that the inclusion of MK increases the resistance of concrete to freeze-thaw action. For instance, the loss in elastic modulus for concrete samples containing 0, 10, 12.5, and 15% MK were 37.6, 33.4, 27.1, and 17.4%, respectively at the 300th cycle, for concrete with a 0.37 w/b ratio.

Sulphate attack resistance: The durability characteristics of self-compacting concrete containing a blend of PC and MK were studied and compared in the study of Kavitha et al. [158]. 100 mm cubes were cast with a 0.38 w/b ratio to test their resistance to sulphate attack. PC was partially replaced with 5, 10, and 15% MK by the weight of binder. The concrete cubes were cured in a water tank for 28 days after casting. Subsequently, they were immersed in a 5% solution of $MgSO_4$ for up to 12 weeks, in which their compressive strength loss and weight loss were recorded weekly. 10% MK replacement level was found as the optimum dosage to mitigate sulphate attack as its concrete showed 13, 6, and 3% less compressive strength loss compared to concrete with 0, 5, and 15% MK, respectively. The weight loss also followed the same trend.

A similar investigation was conducted by Zhang et al. [159], where 9 and 15% of PC were partially replaced by MK to investigate the effect of MK addition on sulphate attack resistance of mortar. 0.4 w/b ratio was used throughout the investigation. The specimens were moist cured for 3 and 7 days before immersing them in a 10% solution of Na_2SO_4 for 28, 56, and 90 days. The loss in compressive and flexural strength was adopted as measures of evaluating sulphate attack resistance. For every age of testing and curing period, the flexural strength loss of mortar made with 15% MK was the least, followed by those with 9% and those without MK content, indicating 15% to be the optimum percentage of replacement with MK. It was also observed that specimens with 7 days of curing period exhibited lower strength loss compared to the specimens cured for 3 days.

Acid attack: Concrete cubes with size 150 mm and concrete cubes with size 100mm were cast to determine the compressive strength loss and weight loss, respectively, of concrete samples when exposed to an acidic environment [160]. The cubes were prepared with 0, 10, 20, and 30% MK. The samples were cured in water for 28 days and subsequently immersed in a water tank of 5% HCl for 28, 60, and 90 days. W/b ratios of 0.3, 0.35, 0.4, and 0.45 were used. Based on their results, concrete with 10% MK had the least weight loss for all the w/b ratios. The concrete with 20% MK showed almost the same weight loss as the normal concrete; however, the concrete with 30% MK showed higher weight loss compared to the normal concrete, indicating that 30% MK will be an unsuitable replacement level against acid attack. This means that the combination of 70% PC and 30% MK will produce undesired blended concrete, as its strength will be significantly and comparatively lesser compared to the normal concrete due to insufficient clinker content to bind the materials together.

A similar trend was observed for the compressive strength loss. The effect of aggressiveness of acidic solution on normal, SF, FA, and MK hardened cement paste (HCP) was studied by Roy et al. [161]. Three different SCMs (FA, SF, and MK) were blended with PC to access their comparative benefits. PC was partially replaced at 15.0, 22.5, and 30.0% by MK. At 95% RH and 38^{o}C, the specimens were cured in water for 28 days. After curing, they were separately immersed in a solution of 1% HCl, 1% H_2SO_4, 1% HNO_3, and 5% H_2SO_4 with the corresponding pH of 1.46, 1.46, 1.40, and 1.03, respectively for 28 days. The weight loss was lesser in all MK HCP compared to that of the normal mix for specimens exposed to 1% HCl, 1% H_2SO_4, and 1% HNO_3. 15% MK concrete sample was also observed to exhibit the lowest weight loss for samples exposed to 1% concentration of the acid solutions. However, all the MK samples exposed to 5% H_2SO_4 for 28 days showed poorer resistance to the acid attack of such a high concentration compared to the normal specimen, indicating that the pH of acidic solution significantly affects the performance of MK, FA, and SF mixtures.

Chloride-induced corrosion: The chloride-induced corrosion of reinforced concrete samples manufactured with a w/b ratio of 0.5 was investigated [162]. The concrete samples were prepared with 0, 5, 10, and 15% MK. Reinforcing bars of 12 mm diameter were embedded in the concrete specimens. The specimens were immersed in a solution of 2.5% NaCl for 10 days (wetting) and then dried for 20 days (drying). The test was done repeatedly for 10 cycles of wetting and drying. The time to corrosion initiation of the embedded bars for each of the concrete mix was evaluated via the half-cell potential measurement. 136, 144, 156, and 158 days were the estimated period to corrosion initiation for concrete containing 0, 5, 10, and 15% MK. This showed that the inclusion of MK increases the concrete’s resistance to corrosion.

Wang et al. [163] carried out an experimental investigation on the influence of including MK in concrete in relation to the chloride diffusion of such concrete. The concrete mixtures were made with 0, 3, 6, and 9 % MK at a w/b ratio of 0.6. The concrete samples were cast and cured for 28 days and subsequently immersed in 3.5% NaCl solution for 90 days. Then, 10g of powder was extracted at various depths of 2, 4, 6, 8, and 10mm with the aid of a driller and powder collector. The total and free chloride were evaluated after placing the powder in a solution of 0.1M nitric acid and water, respectively. It was noticed that total, and free chloride decreases with an increase in MK content and depth. For instance, the total chloride ions at 10 mm depth for concrete with 6 and 9% MK were 28.5 and 33.3%, respectively lesser than that of the normal mix.

Carbonation-induced corrosion: The carbonation resistance and permeability of concrete incorporating 0, 5, 10, and 15% MK were evaluated in the study of Barbhuiya et al. [164]. 0.5 and 0.6 w/b ratio were used to prepare the concrete samples. 50mm cores samples were extracted from a block of 250 × 250 × 100mm size prepared for the permeability test. After drying the specimens for two weeks, they were subjected to accelerated carbonation (5% CO_2, 20^{o}C, and 65% RH) for 3 weeks. Subsequently, the samples were longitudinally broken to apply phenolphthalein on the fractured surface. The carbonation depth results indicated that the carbonation depth of concrete containing 10% MK and concrete without MK (control) were similar. Thus, concrete with 5 and 15% showed greater carbonation depth compared to the control mix. This means that the detrimental effect (for carbonation) of portlandite consumption in MK concrete overcame the beneficial effect of pore refinement for the concrete containing 5 and 15% MK. The reverse was the case for the 10% MK (see Figure 5.6) concrete. Similar positive result of MK concrete on carbonation resistance was observed in the study of Bakera & Alexander [147].

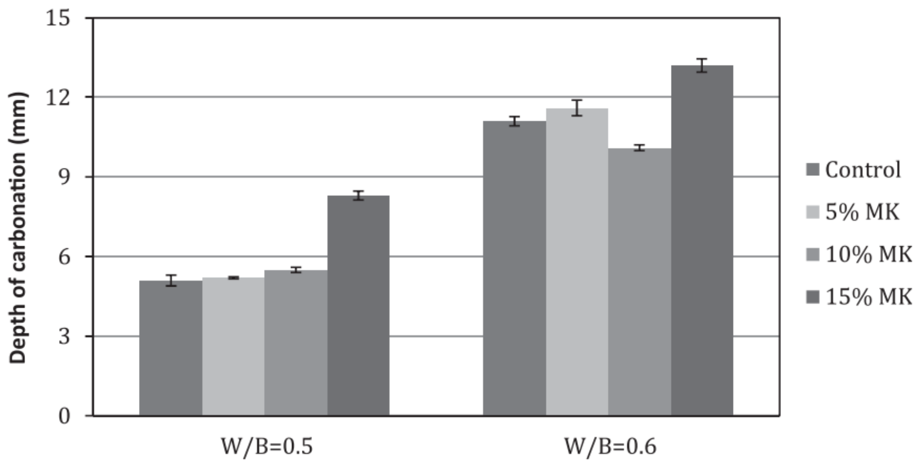

Figure 5. 7: Carbonation depths of MK concrete

The effect of different blends of cement, such as MK-PC, FA-PC, and GGBS-PC , on the carbonation resistance of concrete was investigated [165]. The MK concrete samples were manufactured with a w/b ratio of 0.6 at 15 and 20%

replacement levels. Three concrete prisms of 70 mm thick were prepared for each mix. The concrete samples were cured for 28 days. After the curing period, some specimens were subjected to accelerated carbonation at 4% CO_2, 55% RH, and 20^{o}C for up to 70 days. From the results, the carbonation depth increased as the MK content increased in concrete. This is due to the consumption of portlandite in MK concrete, which results in lower alkalinity in MK concrete than normal mix. However, based on the experimental results obtained by the authors [165], the depth and time for migration of CO_2 to the surface of the embedded steel in concrete were modelled for natural carbonation. The results indicated that the carbonation depth found on the 70th day of accelerated carbonation corresponds to 32 years of exposure under natural conditions. The results also indicated that the carbonation depth does not exceed 30mm after 50 years of exposure. Considering that most structures' service life is 50 years and is always designed with a minimum cover of 50mm, MK inclusion in concrete may be durable in real-life situations in relation to carbonation-induced corrosion.

Alkali-silica reaction: The effect of adding MK to concrete in order to investigate the resistance of such concrete to expansion due to ASR was investigated [166]. The MK concrete was made by partially replacing PC with 5, 10, 15, and 20%. Highly reactive aggregate and cement with 1.25% Na_2O_{eq} (soda content) were employed in the study. The concrete samples were stored in sealed plastic bags after casting so as to maintain an RH of 100%. The expansion of the specimens was measured at a regular interval over a period of two years. Based on the reported data, prism containing 5% MK showed moderate expansion of 0.2% at 2 years while the normal concrete showed significant expansion at about 56 days, exceeding the expansion limit of 0.04%. However, the concrete with 10 and 15% MK showed good durability against expansion due to ASR. They both exhibited expansion values lesser than 0.04% at all ages of testing.

A similar investigation was conducted by Gruber et al. [167]. Concrete prisms of size 75 × 75 × 300mm were cast with 0, 5, 10, 15, and 20% MK as percentage replacement of cement. The mixing water during concreting was dosed with NaOH to increase the alkali of the cement to 1.25% Na_2O_{eq}. The specimens were kept in a condition of RH 100% and 38°C for up till 2 years. The expansion measurement showed that the normal concrete exhibited the largest expansion, exceeding the expansion limit of 0.04% at 3 months. Also, the 5% MK concrete also showed expansion values greater than the limit, while concrete with 10-20% showed an expansion limit lesser than 0.04%. It was observed that concrete with 15 and 20% MK had a similar expansion value at all times.

5.4 Rice Husk Ash

Rice husk ash (RHA) is produced from burning rice husk, an agricultural by-product of rice. The production of 10 metric tons of rice brings about 2 metric tons of rice husk [78]. Rice husk consists of cellulose, lignin, and silica, in which upon combustion, cellulose, and lignin disappear, leaving silica as the major constituent of RHA [168]. In order to produce high-quality RHA, Ganesan et al. [169] recommends that the rice husk is subjected to a temperature between 550°C-700°C for one hour. RH that is partially burnt is usually black in colour, while the one that is completely burnt is either white or grey in colour [78]. Furthermore, the chemical and physical properties of RHA depend on its burning process and duration of burning. For instance, RHA will consist of amorphous silica when RH is subjected to temperature up to 900°C for less than 1 hour, while RHA will consist of crystalline silica when RH is heated at 1000°C for more than 5 minutes [78]. China, Indonesia, and India have the highest productions of rice globally. Importantly, about 40 countries in Africa produce 60% of rice being consumed by their citizens [170], [171], including Nigeria, Egypt, Madagascar, Tanzania, Ghana, South Africa, Mali, Gambia, Liberia, Niger, Rwanda, Guinea, Malawi, Burundi, Mauritania, and Togo [171]. As RHA is a by-product of rice, it is relatively cheaper than PC [171].

5.4.1 Mechanical properties of RHA concrete

Strength: The strength and durability properties of concrete incorporating RHA as a partial replacement of cement were evaluated in the study of Ramasamy [172]. The RHA concrete specimens were made with 5, 10, 15, and 20% RHA by weight of the binder. W/b ratio of 0.35 was used to prepare the high-strength concrete specimens, while 0.43 was adopted for the medium-strength samples. The specimens were subjected to compressive strength testing after curing them for 7, 28, 56, 90, and 180 days. From the results, the compressive strength of the concrete samples increased with the addition of RHA up to 10% replacement level; any further increment reduces the strength of the concrete. This means that less than 90% of cement clinker in the mix will produce fewer calcium silica hydrates and portlandite, leaving the excessive silica from RHA (since little portlandite is available to react with) with nothing to react with. Thereby, no additional hydration products will be formed. The compressive strength of concrete with 5 and 10% RHA were 2.79 and 4.17%, respectively higher than the normal concrete, while those with 15 and 20% were lesser by 9.72 and 26.39%, respectively, compared to the normal mix at 90 days for 0.35 mix.

In the same vein, optimal percentage replacement of cement by RHA that consists of about 87% of silica in relation to strength properties of concrete was investigated by Ganesan et al. [169]. The percentage of replacement by RHA were 5, 10, 15, 20, 25, 30, and 35%. The concrete samples were cast and cured in water for 7, 14, 28, and 90 days before testing them on a compression testing machine. Their results showed that compressive strength increases with age of curing and RHA content up till 30% replacement level. It should be noted that the compressive strength of concrete with 30% RHA was very similar with that of the normal concrete, therefore, 20% RHA replacement level can be chosen as the optimum dosage as it exhibited the maximum strength. For example, the compressive strength of concrete with 0, 20, 25 and 30% RHA were 38.3, 46.0, 43.0, and 38.7 MPa, respectively after 90 days of curing.

Tensile strength of concrete containing 0, 5, 10, 15, and 20% RHA as partial replacement of cement were studied [173]. For each mix, three cylindrical specimens of size 200 × 100 mm were cast and cured for 7, 14, and 28 days before determining their corresponding tensile strength. The average value of tensile strength for the three specimens was determined and taken as the tensile strength of such mix. 10% replacement level was considered as the optimum dosage of RHA as its concrete showed tensile strength value that was 6.25, 4.75, 12.5, and 20% higher than those with 0, 5, 15, and 20% RHA, respectively, after 28 days of curing. The same trend was noticed after 7 and 14 days of curing.

Moreover, three different RHA with different average particle sizes (APS), 31.3, 18.3, and 11.5µm, partially replaced PC at 20% to produce concrete samples [174]. The w/b ratio was kept at 0.53 throughout the investigation. After casting, the specimens were cured in a water tank for 28, 90, and 180 days. The flexural strength results indicated that all the concrete mixes made with RHA, irrespective of their APS, exhibited higher flexural strength than the normal concrete. The results also showed that flexural strength of RHA concrete increased with a decrease in APS such that the flexural strength of RHA concrete made with APS of 31.3, 18.3, and 11.5µm were 5.4, 5.4 and 5.7 MPa and 5.5, 5.7, and 6.1 MPa, after 90 and 180 days, respectively.

Elastic modulus: The effect of RHA and its APS on the elastic modulus of concrete was reported in the study of Habeeb & Fayyadh [174]. Three different RHA with different APS of 31.3, 18.3, and 11.5µm were adopted in the research. The concrete specimens were made with 0 and 20% of RHA by weight of the binder. A constant w/b ratio of 0.53 was used for the investigation. After 24 hours of casting, the specimens were demoulded, cured in water for 28, 90, and 180 days and tested for elastic modulus, respectively. As per the results, the elastic modulus of all the RHA concrete samples was higher than that of the normal mix for all ages of testing. More so, an inversely related relationship was observed between the elastic modulus of the RHA concrete and the APS; that is, the higher the APS, the lower the elastic modulus. At 28 and 180 days, the elastic modulus of RHA concrete with APS of 31.3, 18.3, and 11.5µm were 30.1 & 30.8, 30.2 & 31.4, and 31.7 & 32.9 GPa, respectively while those with 0 RHA were 29.6 and 31.0 GPa, respectively.

The suitability of RHA in making masonry blocks was investigated in the research conducted by Alwani et al. [175]. The masonry blocks of size 390 × 190 × 100mm were cast with a w/b ratio of 0.7. The blocks were prepared with 0, 10, 15, and 20% RHA by the weight of cement. The blocks were covered with burlap for 24 hours after casting, after which they were cured in water for 7 days.

The blocks were tested for modulus of elasticity at 60 days. It was observed that the modulus of elasticity of the masonry blocks increases as the RHA content increases up to 15% replacement level. To elaborate, the masonry block with 15% RHA had elastic modulus that was 17.6, 4.7, and 23.5% higher than those containing 0, 10, and 20% RHA, respectively. Thus, 15% was found as the optimum replacement level for RHA.

Creep: The creep strain of concrete containing ground RHA (GRHA) and natural RHA (NRHA-without processing) was evaluated [176]. Three cylindrical specimens were cast with 0, 15% NRHA, and 15% GRHA content, respectively, and were subjected to a sustained load. 40% of the compressive strength at 28 days of curing was used as the sustained load. Three specimens with the same mixes as mentioned above were also prepared to measure the unloaded strain. The creep strains were evaluated for up to 500 days. The results showed that concrete containing 15% GRHA had creep values lesser than that of the normal mix at all ages of testing, while the sample with 15% NRHA exhibited slightly higher creep strain than the control mix. This shows that it is necessary for RHA to be ground so as to optimize its performance in concrete in relation to creep behaviour.

Moreover, creep of normal and RHA concrete was studied and compared by He et al. [177]. The replacement levels of PC with RHA were 0, 5, 10, 15, and 20%. A superplasticizer was used to enhance the workability of the mixes, and a w/b ratio of 0.4 was used throughout the investigation. The concrete specimens were cast with a mould of 100 × 100 × 300mm and were cured in a controlled room of RH 95% and 20^{o}C for 7 days. A sustained load of 25% of the 28-day compressive strength of the concrete specimens was used for the creep testing. The creep strains of the specimens were recorded up to 200 days. It was observed that the creep strain reduces with an increase in the RHA content of the binder system. For instance, the creep strain at 100 and 200 days of concrete containing 0, 10, 15, and 20% RHA were 700 & 735, 557 & 562, 482 & 497, and 439 & 473 microstrain, respectively.

Shrinkage: The effectiveness of using RHA in mitigating the autogenous shrinkage of ultra-high performance concrete (UHPC) was evaluated [178]. The concrete specimens were prepared using 0, 10, and 20% RHA of APS 5.6μm by weight of cement. The w/b ratio of 0.18 was adopted in the research. Three sealed corrugated moulds of size 28.5 × 440mm were prepared for each sample, and their autogenous shrinkage after 12 hours of final setting up till 28 days were determined. It was observed that the autogenous shrinkage values of the control mix were lesser than those of RHA mixes at early ages, especially after the first 12 hours of setting. However, there was a marked difference in their autogenous shrinkage values at later ages starting from 2 days after setting. 20% was found as the optimum replacement level as its shrinkage value was about 88 and 75% lesser than those with 0 and 10% RHA at the end of the experiment.

Similar results were noticed in the study of de Sensale et al. [179]. The effect of two types of RHA on the autogenous shrinkage values of cement paste was reported in their study. Although the two RHA had the same APS of 8μm, however, one of them was considered non-crystalline (CRHA) while the other was considered crystalline (RRHA) based on X-ray diffraction analysis. The samples were made with 0, 5, and 10% RHA by weight of cement with a constant w/b ratio of 0.3. Three samples were cast for each mix, and their autogenous shrinkage values were evaluated after 24hours of final setting up till 28 days. As per the results, the RRHA samples showed lesser autogenous shrinkage values compared to the normal mix even at early-age of about 1-2 days. This can be attributed to the filler effect of the RRHA samples compared to the hydration effect of the PC samples. Although the autogenous shrinkage values of the CRHA samples were lesser than the normal mix at later ages of about 4 days and above, the CRHA samples showed higher shrinkage values at early ages. This can be attributed to the high pozzolanic effect due to the high silica content of CRHA compared to that of the normal mix. It was concluded that samples with 10% RHA content exhibited the lowest shrinkage values for both CRHA and RRHA samples at 4 days and above (see Figure 5.7).

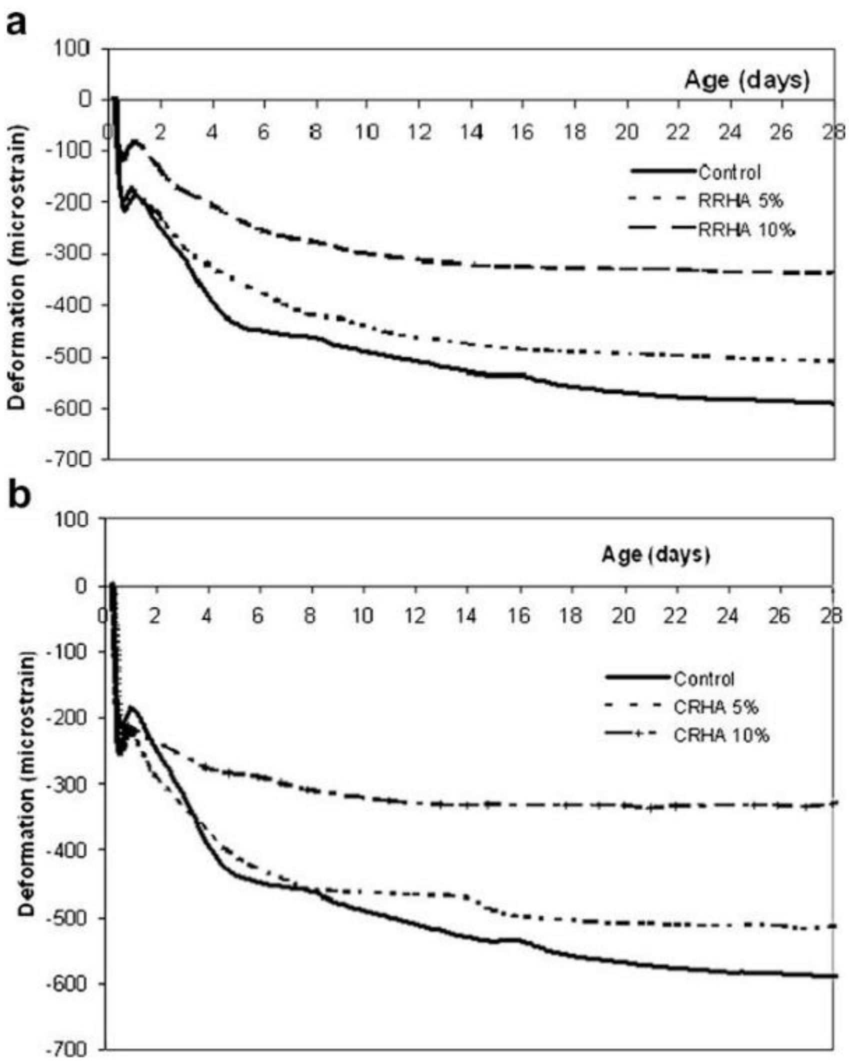

Figure 5. 8: Shrinkage strain of normal and RHA concrete [179]

5.4.2 Durability properties of RHA concrete

Freeze-thaw: Park et al. [180] reported the influence of incorporating RHA on freeze-thaw resistance of concrete at different w/b ratios. The w/b ratios of 0.38 and 0.45 were adopted to separately prepare concrete samples with 0, 5, and 10% RHA by weight of the binder. It should be noted that air-entraining admixture was purposefully not adopted in the investigation to examine the actual role of void spaces in RHA concrete during freezing and thawing. After 14 days of casting the specimens, they were placed in a controlled chamber where freezing and thawing action was simulated. The freezing and thawing cycles continued up to 600 cycles. Each sample's durability factor (DF) and compressive strength were determined after 300 and 600 cycles. Although all the concrete specimens were found durable at the end of 300 and 600 cycles since their DF factors were more than 80%, however, the DF factors for all the RHA concrete samples were more than those of the normal mixes. For instance, the DF factor for concrete with 5% RHA was 5.7 and 6.7% higher than that without RHA content and 10% RHA at the end of the 600 cycles. The results also revealed that the resistance of concrete to freeze-thaw reduces with an increase in the w/b ratio. This means that the DF factor reduces as w/b increases.

Furthermore, the influence of RHA in making foamed concrete samples containing 100% PC, 10% RHA & 90% PC, and 20% RHA & 80% PC in relation to the concrete's resistance to freeze-thaw action was evaluated [181]. Foam contents of 40 kg/m^3 and w/b ratio of 0.7 were adopted to prepare the concrete specimens. The fine aggregate was also partially replaced by 25 and 50% waste marble powder (WMP). After casting, the specimens were exposed to freezing and thawing cycles up till 200 times (i.e., 200 cycles). The loss in compressive and flexural strength was used as the index for freeze-thaw resistance. Based on the data reported, all the RHA concrete even without the inclusion of WMP showed higher resistance to freeze-thaw by exhibiting lower strength reductions. It was also observed that concrete containing 10% RHA exhibited lesser loss in strength compared to concrete with 20% RHA. For instance, the compressive strength loss at 200 cycles for concrete containing 0, 10, and 20% RHA were 40, 11.8, and 15.2%, respectively while those of the flexural strength were 28.9, 12.9, and 17.7%, respectively.

Sulphate attack: Ramezanianpo et al. [182] studied the effect of incorporating RHA in concrete in terms of the concrete's resistance to sulphate attack. 4 types of concrete mixtures were made with 0, 7, 10, and 15% RHA, respectively. 0.5 w/b ratio was used for the investigation. After casting the specimens with cube-moulds of size 100mm, they were cured in water for 28 days, after which they were separately immersed in a solution of 5% Na_2SO_4 and 5% $MgSO_4$ for 60 days. The change in strength was used as an index for sulphate attack resistance. It was observed that all the concrete samples showed an increase in compressive strength at 60 days of exposure except the normal concrete immersed in the magnesium sulphate solution. This increase in strength can be attributed to the continuous hydration of cementitious components to form more hydration products, thus reducing the porosity and increasing the strength of the concrete. Also, for the RHA concrete, the pozzolanic reaction of RHA to form additional calcium silicate hydrate (CSH) was dominant rather than the reaction of sulphate ions with the cement paste to form the expansive gel, ettringite at the early age of reaction. However, the latter reaction (sulphate attack) will be dominant at later ages which will reduce the strength of the concrete and probably lead to loss of weight. Therefore, the concrete with 10% RHA content was observed to perform best since it exhibited the highest compressive strength values.

Furthermore, a similar investigation was conducted by [183]. Concrete samples were made with 0, 10, 20, and 30% RHA to investigate their resistance to sulphate attack. Three cubic samples of size 100 mm were prepared for each mix. The specimens were immersed in a 5% Na_2SO_4 solution for up to 180 days. The pH of the solution was ensured to be constant throughout the investigation. Subsequently, the loss in compression strength was evaluated at 3, 7, 28, 56, 90, and 180 days. The loss in compressive strength only became significant at 90 days. 10% and 20% were observed to be the optimum percentage replacement level as their concrete samples exhibited the lowest strength loss over the testing period. For instance, the compressive strength loss at 180 days for concrete with 0, 10, 20, and 30% were about 32, 25.5, 25.5, and 27.5%, respectively.

Acid attack: Durability properties such as acid attack resistance of concrete containing 0, 5, 10, 15, and 20% were examined [172]. A constant w/b ratio of 0.35 and a superplasticizer to enhance the workability were used in all the mixes. Concrete cubes of size 100 mm were cast and cured for 28 days in water. The specimens were immersed in 5% HCl solution for 60 and 90 days. The initial concentration (pH of 2) of the solution was maintained throughout the investigation. The loss in weight and compressive strengths were used as indicators for acid attack resistance. The results indicated that there was a direct relationship between acid attack resistance and RHA content at both periods of testing; that is, the higher the replacement percentage, the higher the resistance.

For instance, the strength loss for concrete containing 0, 5, 10, 15, and 20% RHA were about 20.5, 17.4, 15.6, 13.0, and 10.0%, respectively at 90 days.

Hashem et al. [184] studied the effect of temperatures on the aggressiveness of acid on normal and RHA hardened cement paste (HCP) made with a w/b ratio of 0.3. The normal mix was prepared with 100% PC, while the RHA mix was prepared with 10% RHA and 90% PC. After 28 days of curing in water, the specimens were immersed in different acidic solutions (HCl) of 1, 2, and 3 pH values and at two different temperatures of 20 and 50^oC. It was ensured that the pH of each acidic solution was kept constant. The compressive strength of the specimens was measured after 15, 30, 60, 90, and 120 days of exposure. Firstly, the results indicated that HCP immersed in higher acidic concentration showed lesser strength than the ones in lower acidic solution. For instance, the compressive strength of the normal HCP immersed in a solution of H_2SO_4 with a pH value of 1 was about 22% higher than that of the normal HCP immersed in a solution of H_2SO_4 with a pH value of 3 after 60 days of exposure. Secondly, the inclusion of RHA in the mixtures increases the HCP's resistance to acid attack. It was observed that the compressive strength of concrete with 10% RHA was about 5.2 and 5.3% higher than that without RHA content after 60 and 120 days of exposure, respectively. Thirdly, samples cured at higher temperatures during acidic exposure exhibited higher compressive strength than those cured at lower temperatures. For instance, the RHA sample cured at 20^oC had a compressive strength value that was about 2.4% lesser than the sample cured at 50^oC.

Chloride-induced corrosion: The ability of RHA concrete to reduce chloride ingress in a chloride-laden simulated environment was investigated by Zareei et al. [185]. 5 various mixes were adopted to produce series of cubic samples containing 0, 5, 10, 15, 20, and 25% RHA by weight of the binder. W/b ratio of 0.38 was used for the investigation. After 28 days of curing the specimens, the specimens were immersed in a solution of 3% NaCl, and the total charge passed through each specimen in 6 hours, under 60V, was measured and recorded. As per the results, there was a decrease in the amount of total charge passed as the content of RHA in concrete increases. The amount of the total charge passed with concrete containing 25% RHA was 81.5, 74.8, 70.3, 62.5, and 55.14% lesser than those with 0, 5, 10, 15, and 20% RHA (see Figure 5.8).

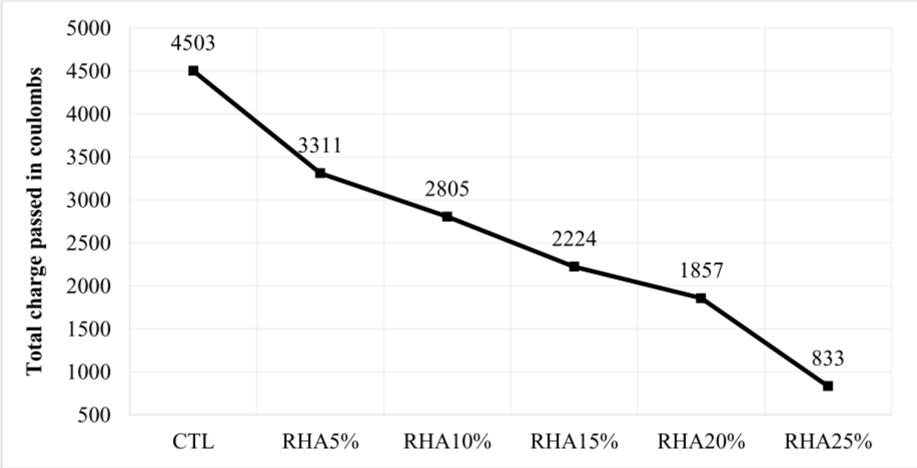

Figure 5. 9: Total charged passed of RHA concrete [185]

Furthermore, an experimental investigation on the suitability of using RHA and SF concrete in a chloride-driven environment was conducted [186]. For the RHA concrete samples, 5, 10, 15, and 20% replacement levels were used. Three cylindrical samples with a w/b ratio of 0.33 were prepared for each mix and cured for 28 days. Subsequently, the samples were immersed in sodium chloride solution, and 6V was passed through each of the samples for 6 hours. The results indicated that concrete with 20% RHA only allowed 261 coulombs to pass through it while the normal concrete allowed 2,782 coulombs to pass through it. The concrete with 5, 10, and 15% RHA exhibited total charged passed that was 67.6, 74.8, and 89.2% lesser than that of the normal concrete, respectively. It was also observed that concrete with 10% SF and concrete with 15% RHA had similar values of total charge passed.

Carbonation-induced corrosion: Cizer et al. [187] investigated the carbonation resistance of mortar containing RHA as a partial replacement for cement. The replacement levels with RHA were 30, 50, and 70% by weight of cement. Each mix was placed in a beam mould of size 40 × 40 × 160mm and cured at 60% RH and 20°C for 7, 28, 60, 90, and 120 days. After each curing period, each specimen was placed in accelerated carbonation conditions. The carbonation depth increases significantly as the RH content of the mixes increases. In fact, the sample with 70% RHA reached full carbonation depth at 60 days. This is due to the low content of cement in the mortar, 30% by weight of the binder, which thus reduces the pH of the mortar significantly. Also, the carbonation depth of concrete containing 30 and 50% RHA was 80.0 and 96.6% higher than that of the normal mix at 120 days.

More so, the influence of RHA concrete on carbonation depth at different relative humidity was investigated by [188]. The concrete samples were made with 0, 5, 10, 15, and 20% RHA. W/b ratio of 0.45 was adopted to cast the prisms samples of size 100 ×100 × 500 mm. 50 × 50 × 100 mm samples were cut from each of the 100 ×100 × 100 samples after curing them for 7, 14, and 28 days. The new prisms samples were preconditioned under 27ºC and different relative humidities of 30, 45, and 60%, so as to achieve a constant mass, for 7, 14, and 28 days. Subsequently, all the new prisms were subjected to accelerated carbonation (5% CO_2 and 27ºC) at the three different relative humidities: 30, 45, and 60%. The carbonation depth of each specimen was determined with the aid of a phenolphthalein indicator. Firstly, a direct relationship between the RHA content in concrete and carbonation depth was observed. This means that higher RHA content caused higher carbonation depth. For instance, after 7 days of curing, the carbonation depth of concrete with 20% RHA was 5.79mm, while those with 0, 5, and 15% RHA were 2.95, 3.65, and 5.06mm at the same RH of 45%. The increase in carbonation depth of RHA concrete can be explained by the low content of portlandite in the mix, which thus lowers the pH of the concrete. Secondly, the carbonation depths of all the specimens reduces as the curing days increase. For instance, the carbonation depth of concrete containing 0 and 5% RHA at RH of 60% were 1.49 & 1.77 mm, 1.12 & 1.43 mm, and 0.61 & 1.14 mm, for 7-, 14- and 28-days curing period, respectively. The reduction in carbonation depths with prolonged curing can be attributed to the continuous hydration reaction, thus producing more stronger hydration products. Therefore, the concrete samples will be less permeable to the ingress of CO_2 as the curing proceeds. Thirdly, an inverse relationship was observed between the RH and carbonation depth; that is, the higher the RH, the lower the carbonation depth.

For instance, the carbonation depth of concrete with 10% RHA at 45 and 60% RH were 2.64 and 1.37mm. The reduction of carbonation depth with increasing RH value can be attributed to the fact that the pores of the concrete will be saturated with water at higher RH, thereby inhibiting the ingress of CO_2 into the concrete.

Alkali-silica reaction: The utilization of RHA concrete in an environment prone to ASR was investigated [189]. The specimens were made with 0, 10, 20, 30, and 40% RHA by weight of the binder. Cement with a high amount of alkalis was adopted (0.83% by weight of cement). The aggregates adopted were also highly reactive. The specimens were subjected to a mortar bar expansion test for 14 and 28 days after 2 days of curing. Based on the provided data, the normal mix had expansion greater than the expansion limits for mortar at 14 and 28 days: 0.1 and 0.2%, respectively. All the RHA specimens had expansion limits lower than the various expansion limits except the specimen made with 10% RHA for the 14-day expansion test. 30 and 40% replacement levels were recommended for RHA replacement as they both exhibited expansion lower than 0.1% for both 14 and 28 days expansion tests.

Zerbino et al. [190] tested the effect of two different RHA on concrete's resistance to expansion due to ASR. The first type was RHA produced by uncontrolled burning conditions and without further processing by grinding it (NRHA). The other type was generated from a controlled burning environment and underwent a grinding process (GRHA). The concrete mix was made with 0, 15, and 25% RHA. Reactive aggregate and cement were used to accelerate the ASR of the samples. The concrete specimens of size 75 × 75 × 300 mm were prepared and tested for concrete prism expansion after curing. The test was done up to 52 weeks. The expansion tests revealed that all the samples made with NRHA showed expansion greater than the expansion limit of 0.04% for concrete samples, while the expansion for the GRHA samples was all below the limit of 0.04%. The low resistance of the NRHA samples can be traced to its low pozzolanic activity due to its large particles. Hence, well burnt and properly ground RHA should be adopted for the manufacturing of concrete so as to maximize its benefits.

5.5 Closure

This chapter presents the effect of different SCMs; FA, SF, MK, and RHA on mechanical and durability properties of concrete discussed in chapter 3 and 4, respectively. Table 5.1 - 5.4 give a summary of the effect of these four SCMs on the mechanical and durability properties of concrete. A further discussion on the benefits and limitations of the various SCMs is presented in chapter seven.

Table 5. 2: Summary of the effects of FA and SF on the mechanical properties of concrete

Mechanical properties	**References and their results** **Fly Ash Specimens**	**References and their results** **Fly Ash Specimens**	**References and their results** **Silica Fume Specimens**	**References and their results** **Silica Fume Specimen**
Compressive strength	[93] w/b = 0.4; 3 and 365 days concrete specimens, resp. 26.86 and 61.43 MPa – 0% 19.62 and 69.32 MPa – 20% 16.95 and 65.32 MPa – 30%	[94] w/b = 0.55; 3 and 91 days concrete specimens, resp. 24 and 29 MPa – 0% 18 and 31.5MPa – 20%	[125]; 7, 28, and 56 days concrete specimens, resp.; w/b = 0.30 & 0.35. 50, 70, and 77MPa &41, 50, and 51 MPa – 0% 57, 73, and 79MPa & 42, 58, and 60 MPa – 10% 61, 82, and 85MPa & 50, 63, and 68 MPa – 15%	[126]; 7 and 14 days mortar specimens, resp. 19.87 and 26.33 MPa – 0% 27.51 and 37.53 MPa – 10% 25.89 and 38.63 MPa – 15%
Tensile strength* and flexural strength**, respectively	[95] w/b= 0.45; 28 days concrete specimens 1.55 MPa – 0% 1.68 MPa – 10% 1.48 MPa – 20% 1.21 MPa – 30%	[96] w/b = 0.48; 7, 28 and 56 days concrete specimens, resp. 3.18, 5.70, and 6.00 MPa – 0% 3.12, 5.40, and 6.30 MPa – 10% 3.00, 5.25, and 6.60 MPa – 20% 2.25, 5.10, and 6.15 MPa – 30%	[127]; 28 days concrete specimens; w/b ratio 0.26, 0.30, and 0.42, respectively. 5.2, 5.0, and 3.5 MPa – 0% 6.3, 6.0, and 4.4 MPa – 10% 6.6, 6.3, and 4.45 MPa – 15% 6.5, 6.25, and 4.45MPa -20%	[128]; w/b = 0.36; 7 and 28 days concrete specimens, respectively. 3.92 and 4.95 MPa – 0% 4.82 and 5.35 MPa – 6% 5.17 and 5.79 MPa – 12% 4.62 and 5.58 MPa – 15%
Elastic modulus	[97] w/b = 0.59; 10 and 100 days concrete specimens, resp. 31 GPa and 35 GPa – 0% 22 GPa and 35 GPa – 20%	[98] w/b = 0.50; 28 days concrete specimens 33.42 GPa – 0% 30.12 GPa -15% 32.83 GPa – 30%	[129]; w/b = 0.35; 7 and 28 days concrete specimens, resp. 28.8 and 34.4 GPa – 0% 31.1 and 37.0 GPa – 10% 31.5 and 38.1 GPa – 15%	[130]; w/b = 0.25; 28 days concrete specimens. 29.69 GPa – 0%; 38.88 GPa – 5% 41.55 GPa – 10%; 43.03 GPa – 15% 40.35GPa – 20%; 35.15 GPa – 25%
Creep	[99] w/b = 0.50; contribution of creep to total deflection; 7, 28, and 180 days concrete specimens 46, 61, and 72% - 0% 42, 59, and 73% - 25% 49, 61, and 72% - 40%	[100] w/b = 0.55; 30% stress/strength ratio, creep strain measurements at 14 days 15.5×10^{-6} – 0% 16.2×10^{-6} – 20%	[129]; w/b = 0.35; creep strain measurements at 7 and 28 days of loading, resp. 595 and 413 µm – 0% 510 and 407 µm – 6% 459 and 381 µm – 10% 417 and 328 µm – 15%	[36]; w/b = 0.38; stress/ strength = 0.33, creep strain measurement at 14 and 33 days of loading, resp. 203 and 384 µm – 0% 197 and 313 µm – 10%
Shrinkage	[101] w/b ratio of 0.32 for 0% FA, 0.29 for 30% FA, and 0.30 for 50% FA; drying shrinkage measurements for 28 days and 6 months 347 and 554 microstrain – 0% 231 and 394 microstrain – 30% 256 and 413 microstrain – 50%	[102] w/b = 0.3; shrinkage strain measurements (10^4) at 7, 28, and 56 days 1.27, 2.80 and 5.12 – 0% 1.45, 3.70, and 4.90 – 10% 1.70, 4.03, and 7.27 – 40% 3.52, 5.09, and 9.45 – 70%	[129]; w/b = 0.35; drying shrinkage strains after 587 days in air. 532 µm – 0% 528 µm – 6% 523 µm – 10% 512 µm – 15%	[131] w/b = 0.4; drying shrinkage strains after 84 days of drying. 330 µm- 0%; 287 µm – 5% 291 µm – 10%; 316 µm – 15% 330 µm – 20%; 323 µm – 30%

* tensile strength is placed in the first column of each SCM

** flexural strength is placed in the second column of each SCM

Table 5. 3: Summary of the effects of FA and SF on the durability properties of concrete

Durability properties	**References and their results** **Fly Ash Specimens**	**References and their results** **Fly Ash Specimens**	**References and their results** **Silica Fume Specimens**	**References and their results** **Silica Fume Specimens**
Freeze-thaw resistance	[103] w/b = 0.44; specimens were subjected to 30 and 180 cycles of freezing and thawing and subsequently their comp. strengths were determined, respectively. 42.5 and 34.5 MPa – 0% 39 MPa and 37 MPa – 20% 38.5 and 42.5 MPa – 30%	[105] w/b = 0.46; specimens were subjected to 28 cycles of freezing and thawing and their surface scaling were determined for both air entrained and non-entrained specimens, respectively 315 and 375 gr/m^2 – 0% 270 and 330 gr/m^2 – 20%	[132]; w/b = 0.5; specimens were subjected to 100, 250 and 350 cycles, resp. and their relative dynamic modulus were presented: 97.8, 80.6, and 45.0% - 0% 93.0, 85.9, and 78.2% - 6% 92.2, 85.3, and 79.1% - 9% 95.5, 83.9, and 80.3% - 12%	[133]; w/b = 0.19; specimens were subjected to 300 cycles of freezing and thawing and their mass loss rate were presented: 0.6% - 0% 0.44 - 8% 0.47% - 10% 0.46% - 12% 0. 40% - 14%
Sulphate attack resistance	[106] w/b = 0.45; specimens immersed in 5% sodium sulphate solution and their % weight loss were evaluated after 28, 56, and 90 days, resp. 2.14, 2.8, and 3.35% - 0% 2.05, 2.75, and 2.98% - 10% 1.9, 2.48, and 2.56% - 30% 2.02, 2.71, and 2.98 -40%	[107]; specimens were subjected to 5% Na_2SO_4 and $MgSO_4$ for 15 to 150 days and their range of loss in weight are presented: 4.67 to 16.42% (Na_2SO_4) – 0% 3.19 to 12.03% (N_2SO_4) – 20% 5.58 to 18.2% ($MgSO_4$) – 0% 3.44 to 12.52 ($MgSO_4$) – 20%	[134]; w/b = 0.30 to 0.45; specimens exposed to 5% sodium sulphate solution for 15 hours and 9 hours drying daily for 90 days and their mass loss (%) were presented: w/b=0.30; 18.4% - 0% w/b= 0.30; 10.7% - 5% w/b = 0.45; 47.8% - 0% w/b= 0.45; 27.1% - 5%	[135]; w/b ratio = 0.45; specimens exposed to 5% Na_2SO_4 and $MgSO_4$ for 510 days, resp. and their strength, subsequently, were presented: 21.4 MPa (Na_2SO_4) & 31.1 ($MgSO_4$)- 0% 61.6 MPa & 32.8 MPa – 5% 58.1 MPa & 29.4 MPa – 10% 69.0 MPa & 29.1 MPa – 15%
Acid attack resistance	[106] w/b = 0.45; specimens exposed to 1% sulphuric acid and their % weight loss were evaluated after 28, 56, and 90 days, respectively 3.94, 4.54, 5.62% - 0% 2.16, 3.71, and 4.88% - 10% 1.61, 2.89, and 3.96% - 30% 2.02, 3.14, and 4.38% - 40%	[108]; specimens were exposed to 5% H_2SO_4 for 60 days and their % weight loss were evaluated: 5% - 0% 3.3% - 70%	[136]; w/b = 0.36; specimens were to 5% HCl for 30 days and their % loss in compressive strength were evaluated: 11.91% – 0%; 8.18% - 5% 7.69% - 10%; 8.02% - 15% 8.35% - 20%	[137]; w/b = 0.4; specimens were exposed to 1% HCl & H_2SO_4 for 28 days and their weight loss were evaluated: 2.31% (HCl) and 2.93% (H_2SO_4) – 0% 2.18% (HCl) and 2.78% - 10%
Chloride-induced corrosion	[109]; w/b = 0.31, 0.35, and 0.5; exposed to 3.5% NaCl for 140 and 250 days, respectively. 7943, 7262, and 4172 Ωcm^2 – 140 days – 0% 17927, 10162, and 6466 Ωcm^2 – 140 days -20% 9188, 11023, and 3935 Ωcm^2 – 250 days – 0% 17177, 13542, and 7211 Ωcm^2 - 250 days – 20%	[110]; w/b = 0.4, exposed to 3.5% NaCl for 28 and 90 days, respectively – RCPT. 3443 and 2916 coulombs – 0% 2775 and 887 coulombs – 40% 2459 and 1146 coulombs – 60%	[138]; w/b = 0.45; specimens cast with 0.2 – 3.0% NaCl by weight of binder and subjected to 20 cycles of wetting & drying and half-cell potential measurement were taken. -300 to – 583 mV – 0% -275 to – 450 mV – 10%	[139]; w/b = 0.6; resistivity measurements of embedded reinforced concrete were taken at 20 and 50ºC, respectively: 30 and 15 Ωm – 0% 210 and 70 Ωm – 10%
Carbonation-induced corrosion	[112]; carbonation depths were measured after 140 days of exposure to 3% CO_2, 65% RH & 23ºC 8.0mm – 0%; 13.5mm – 20% 18.0mm – 35%; 23.0mm – 50%	[113]; w/b = 0.5, carbonation depths were measured after 70 days of exposure to 4% CO_2, 55% RH & 20ºC 11mm – 0%; 12.3mm – 20% 15.5mm – 40%; 18mm – 50%	[140]; w/b = 0.52 and 0.65; carbonation depths were measured after 20 weeks exposure to 4% CO_2, 55% RH & 20ºC w/b=0.52; 10mm & w/b=0.65; 15mm -0% w/b=0.52; 12mm & w/b = 0.65; 17mm- 5%	[141]; w/b = 0.50; carbonation depths were measured after 100 days exposure to 3 % CO_2 at RH of 100% and 25ºC 9.0mm – 0%; 9.5mm – 5% 11.8mm – 10%; 12mm – 15%
Alkali-silica reaction	[114]; w/b =0.42 to 0.45; reactive siliceous limestone was used; concrete expansion test conducted for 2 years. >0.04% – 0%; > 0.04% – 10%; ≈0.04% - 20% =0.04% - 25%; <0.04% - 50%; <0.04% - 60%	[115]; 4 different reactive aggregates were used; mortar bar test conducted for 14 days =1.48%, 1.50%, 1.48%, and 0.93% - 0% =0.009%, 0.061%, 0.009%, and N/A- 25% =0.013%, 0.010%, 0.013%, and N/A – 40%	[142]; w/b= 0.5; reactive Spratt aggregate was employed; mortar bar test conducted for 14 days and their expansion values were: >0.10% – 0%; >0.10 – 4% = 0.10% - 8% <0.10 – 12 %	[143]; w/b = 0.45; reactive Spratt aggregate was used; concrete prism test was conducted for 1 year. > 0.04% - 0%; >0.04% - 4% <0.04% - 8%; <0.04% - 12%

Table 5. 4: Summary of the effects of MK and RHA on the mechanical properties of concrete

Mechanical properties	**References and their results** **Metakaolin Specimens**	**References and their results** **Metakaolin Specimens**	**References and their results** **Rice Hush Ash Concrete**	**References and their results** **Rice Hush Ash Concrete**
Compressive strength	[149]; w/b = 0.25; 7, 28 and 91 days concrete specimens resp. 45, 60, and 72 MPa – 0%; 48, 64, and 75MPa – 10% 49, 67, and 74 MPa – 5%; 45, 59, and 75 MPa – 15% 46, 59, and 72 MPa – 20%	[150]; w/b = 0.32; 3, 7, and 28 days concrete specimens resp. 28.84, 46.48, and 61.07 MPa – 0% 29.18, 50.73, and 70.74 MPa – 10% 29.69, 53.61, and 74.48 MPa – 15% 24.09, 42.41, and 59.89 MPa – 50%	[172]; w/b = 0.35; 28 and 90 days specimens resp. 70.0 and 73.0 MPa – 0% 71.43 and 73.78 MPa – 5% 73.57 and 77.17 MPa – 10% 58.58 and 63.28 MPa – 15%	[169]; w/b = 0.53; 28 and 90 days specimens, resp. 37.1 and 38.3 MPa – 0% 41.3 and 44.8 MPa – 10% 42.5 and 46.0 MPa – 20% 37.6 and 38.7 MPa -30%
Tensile strength* and flexural strength**, respectively	[150]; w/b = 0.32; 28 days concrete specimens 5.02 MPa – 0% 5.26 MPa – 10% 5.48 MPa– 15% 4.14 MPa – 50%	[149]; w/b = 0.25; 56 and 91 days concrete specimens, resp. 7.5 & 7.63 MPa – 0%; 7.62 & 7.64 MPa - 5% 7.65 & 7.7 MPa -10%; 7.65 & 7.7 MPa – 15% 7.55 & 7.62 MPa – 20%	[173]; 7 and 28 days specimens resp. 1.30, 2.15, and 3.75 MPa – 0% 1.35, 2.20, and 3.90 MPa - 5% 1.40, 2.25, and 4.05 MPa – 10% 1.30, 2.10, and 3.5 MPa – 15%	[174]; w/b = 0.53; 180 days specimens; RHA grinded for 180 (F1), 270 (F2) and 360 minutes (F3) were used resp. 5.1 MPa – 0% 5.5 MPa – 20% (F1) 5.7 MPa – 20% (F2) 6.1 MPa – 20% (F3)
Elastic modulus	[151]; 28 days specimens 39.5 GPa – 0%; 45.3 – 7.5% 50.0 GPa – 10%; 45.7GPa – 12.5% 40.6 GPa – 15%; 33.3 GPa – 17.5%	[152]; w/b = 0.48; specimens subjected to UPV test and their E_c were evaluated & presented at 28, 90, 180 and 300 days, resp. 42.0, 43.0, 42.5, and 43.0 GPa – 0% 42.0, 44.5, 45.0, and 46.0 GPa – 10%	[174]; w/b = 0.53; 28 and 180 days specimens grinded for 180 (F1), 270 (F2) and 360 minutes (F3) were used resp. 29.6 and 31.0 GPa – 0% 30.1 and 31.4 GPa – 20% (F1) 30.2 and 31.7 GPa – 20% (F2) 30.5 and 32.9 GPa – 20% (F3)	[175]; w/b = 0.7; block specimens of 60 days. 0.70 GPa – 0% 0.81 GPa – 10% 0.85 GPa – 15% 0.65 GPa – 20%
Creep	[153]; w/b = 0.28; stress/strength ratio of 0.2; creep strain measurements at 200 days of loading: 350 µm – 0% 280 µm – 5% 180 µm – 10% 166 µm – 15%	[154]; creep strain measurements at 200 days of loading: 280 µm – 0% 190 µm – 5% 105 µm – 10% 98 µm – 15%	[176]; w/b = 0.55; creep strain measurements at 500 days for ground RHA (GRHA) and non-ground RHA (NRHA) concrete, resp. 1750 µm – 0% 2500 µm - 15% (NRHA) 1700 µm – 15% (GRHA)	[177]; w/b = 0.4; creep strain measurements at 100 and 200 days resp. 700 and 735 µm – 0% 557 and 562 µm – 10% 482 and 497 µm – 15% 439 and 473 µm – 20%
Shrinkage	[155] w/b = 0.3 and 0.5; autogenous shrinkage strain measurements at 28 and 129 days, resp. w/b =0.3; 1350 µm/m and 1650 µm/m – 0% w/b = 0.5; 1150 µm/m and 1650 µm/m – 0% w/b = 0.3; 900 µm/m and 950 µm/ m – 20% w/b = 0.5; 590 µm/m and 630 µm/m – 20%	[43]; w/b = 0.35; drying shrinkage strain measurements at 7 and 42 days, resp. 400 and 682 µm – 0% 280 and 420 µm – 5% 245 and 395 µm – 15%	[178]; w/b = 0.18; autogenous shrinkage strain after 12 hours and 28 days, resp. 1.0 mm/m and 2.05mm/m – 0% 1.5 mm/m and 0.98mm/m – 10% 1.0 mm/m and 0.2mm/m – 20%	[179]; w/b = 0.3; autogenous shrinkage strain at 2 and 28 days, resp. 180 µm and 570 µm – 0% 220 µm and 490 µm – 5% 280 µm and 280 µm – 10%

Table 5.5: Summary of the effects of MK and RHA on the durability properties of concre

Durability property	**References and their results** **Metakaolin Specimens**	**References and their results** **Metakaolin Specimens**	**References and their results** **Rice Hush Ash Concrete**	**References and their results** **Rice Hush Ash Concrete**
Freeze-thaw resistance	[156]; w/b = 0.6; specimens subjected 100 cycles of freezing and thawing and their weight loss were evaluated and presented: 4.0% - 0% 2.5% - 5% 2.0% - 10% 3.0% - 20%	[157]; w/b = 0.37 and 0.47; specimens subjected to up to 300 cycles of freezing and thawing and subsequently, their modulus of elasticity were determined resp. 25.36 and 24.1 GPa – 0% 30.70 and 28.95 GPa – 12.5% 34.08 and 32.74 GPa – 15%	[180]; w/b = 0.38 and 0.45; specimens subjected to 300 cycles of freezing and thawing and their compressive strength were measured. 56 and 49 MPa – 0% 58 and 38 MPa – 5% 62 and 51 MPa – 10%	[181]; w/b = 0.7; specimens subjected to 200 cycles of freezing and thawing and their compressive strength were measured 9.78 MPa – 0% 16.67 MPa – 10% 15.83 MPa – 20%
Sulphate attack resistance	[158]; w/b = 0.38; specimens exposed to 5% $MgSO_4$ for 12 weeks and their % strength loss were evaluated and presented: 32% - 0%; 25% - 5% 20% - 10%; 23% - 15%	[159]; w/b = 0.4; specimens exposed to 10% $MgSO_4$ for 28, 56, and 90 days and their flexural strength loss were evaluated and presented: 43, 42, and 52% - 0% 14, 32, and 34% - 9% 13, 26, and 27% - 15%	[182]; w/b = 0.5; specimens exposed to 5% Na_2SO_4 and $MgSO_4$ for 60 days and their compressive strengths were measured and presented: 41.58 and 45.5MPa – 0% 49.78 and 50.28 MPa – 7% 51.40 and 55.49 MPa – 10% 47.74 and 52.16 MPa – 15%	[183]; specimens exposed to 5% Na_2SO_4 for 90 and 180 days, resp. and their % loss in compressive strength were presented: 13 and 32.0% - 0% 10 and 25.5% - 10% 9 and 25.5% - 20% 13 and 27.5% - 30% 15 and 27.0% - 40%
Acid attack resistance	[160]; specimens immersed in 5% HCl for 90 days with w/b = 0.30, 0.35, 0.4, and 0.45, resp. and their % weight loss were evaluated: 8.26, 8.62, 9.04, and 9.44% - 0% 9.44, 5.75, 6.02, and 6.60 – 10% 6.11, 6.32, 6.64, and 6.92 – 20% 8.32, 8.76, 9.12, and 9.52 -30%	[161]; w/b = 0.40; specimens subjected to 5% H_2SO_4 for 28 days and their % loss in weight were evaluated. 0.2% - 0% 0.8% - 15.0% 0.6% - 22.50% 0.25% - 30.0%	[172]; w/b = 0.35; specimens were subjected to 5% HCl for 60 and 90 days and their loss in weight were evaluated. 12.0 and 20.5% - 0% 8.5 and 17.4% - 5% 7.3 and 15.6% - 10% 5.0 and 10.0% - 20%	[184]; w/b = 0.3; specimens exposed to H_2SO_4 of pH value of 1 at 20ºC and 50ºC for 120 days and their compressive strength were measured 27.0 and 25.0 MPa – 0% 30.4 and 38.0 MPa – 10%
Chloride-induced corrosion	[162]; w/b = 0.5; embedded reinforced specimens exposed to chloride environments and their time to corrosion initiation were evaluated via half-cell potential measurements 136 days – 0%; 144 days – 5% 156 days – 10%; 158 days -15%	[163]; w/b = 0.6; specimens were exposed to 3.5% NaCl for 90 days and their bound chlorides at 10mm depth were evaluated. 0.06915% - 0%; 0.08218% - 3% 0.08894% - 6%; 0.09124% - 9%	[185]; w/b = 0.38; 28 days specimens exposed to 3% NaCl – RCPT 4503 coulombs – 0% 3311 coulombs – 5% 2805 coulombs – 10% 2224 coulombs – 15% 1857 coulombs – 20% 833 coulombs – 25%	[186]; w/b = 0.33; 28 days specimens exposed to NaCl – RCPT 2782 coulombs – 0% 800 coulombs – 5% 650 coulombs – 10% 300 coulombs - 15% 261 coulombs – 20%
Carbonation-induced corrosion	[164]; w/b = 0.5 & 0.6; specimens exposed to 5% CO_2, 20ºC, and 65% RH for 3 weeks and their carbonation depth was measured: w/b=0.5; 5.2mm & w/b = 0.6; 10.9mm – 0% w/b =0.5; 5.25mm & w/b = 0.6; 11.6mm - 5% w/b = 0.5; 5.26mm & w/b = 0.6; 10.2mm – 10%	[165]; w/b = 0.6; specimens exposed to 4% CO_2, 20ºC, and 55% RH for 70 days and their carbonation depth was measured: 9.5mm – 0%; 16.5 mm – 15% 17.6mm – 20%; 21.0mm – 25%	[187]; w/b = 0.48 to 0.7; specimens exposed to 60% RH and 20ºC and their carbonation depths were measured at 90 and 120 days, resp. 0.5 and 1.0mm – 0%; 3.5 & 4.0 mm -30% 8.8 and 9.7mm – 50%; 20 & 20mm – 70%	[188]; w/b = 0.46; specimens exposed to 5% CO_2 and 27ºC at different relative humidity: 45, and 60% for 28 days. 1.13 and 0.61 mm – 0% 1.18 and 1.14 mm – 5% 2.64 and 1.37 mm – 10% 3.44 and 1.69 mm – 15% 3.48 and 1.70 mm – 20%
Alkali-silica reaction	[166]; w/b = 0.42 to 0.45; highly reactive siliceous limestone was used; concrete expansion test conducted for 2 years. >0.04% – 0%; > 0.04% – 5% ≈0.04% - 10%; <0.04% - 15% <0.04% - 20%	[167]; w/b = 0.3 and 0.4; highly reactive siliceous limestone was used; concrete expansion test conducted for 2 years. >0.04% – 0%; > 0.04% – 5% <0.04% - 10%; <0.04% - 15% <0.04% - 20%	[189]; reactive aggregate was adopted; mortar expansion test was done for 14 and 28 days, resp. >0.1% & >0.2%– 0%; > 0.1% & <0.2% – 10% =0.1% &<0.2% - 20%; <0.1% & <0.2% - 20% <0.1% & <0.2% - 40%	[190]; w/b = 0.47; highly reactive siliceous limestone was used; concrete expansion test conducted for 2 years. >0.04% – 0% <0.04% - 15% <0.04% - 25%

6. EFFECTS OF TERNARY BLENDED CEMENT ON CONCRETE PROPERTIES

Ternary blended cement contains plain Portland cement and two SCMs. In this study, two ternary systems are reviewed: limestone calcined clay cement (LC^3) and fly ash, limestone, and plain Portland cement. The effects of these SCMs in relation to the mechanical and durability properties of concrete are presented in this chapter.

6.1 Limestone calcined clay cement (LC^3)

LC^3 is a blend of limestone powder, calcined clay, and plain Portland cement. The limestone powder is often produced by grinding limestone extracted from rock into fine particles. The calcined clay is usually produced by calcinating raw clay in a rotary or flash kiln at air temperature between 700^{o}C - 900^{o}C [191]. Subsequently, limestone powder, calcined clay, plain Portland cement are either inter-ground or separately ground and mixed to produce the LC^3 binder system. Furthermore, as reported by Yudiesky et al. [192], the cost production of the LC^3 binder system is 15-25% lesser than that of the Portland cement [192].

6.1.1 Mechanical properties of LC3 concrete

Strength properties: 46% of PC was replaced by LC^2 (31% of calcined clay and 15% of limestone) to produce LC^3 concrete and compare it with normal concrete and FA concrete containing 30% FA by weight of cement. 4% of gypsum was present in the LC^3 specimen [193]. The w/b ratios adopted were 0.35, 0.45, and 0.5. The concrete samples were cast and tested for compressive strength after 2, 7, 28, 90, and 365 days of curing. Despite the lower content of cement in the LC^3 mix, its specimens showed comparable compressive strength with PC concrete at an early age, unlike that of FA concrete. However, after 28 days, the compressive strength of FA and LC^3 concrete are higher than those of the normal concrete for all the w/b ratios [193].

Furthermore, a similar study was conducted by Sree et al. [194], where strength properties of LC^3 concrete, Pozzolana Portland Cement (PPC) Concrete, and normal concrete were compared. 0.46 w/b ratio was adopted for the three mixes. The tensile strength results indicated that LC^3 concrete performed slightly better in terms of splitting tensile strength than the other two mixes as its strength value was 0.63 and 1.3 % higher than that of the normal and PPC concrete, respectively. More so, [195] compared the flexural strength of LC^3 concrete with that of PPC concrete and normal concrete. The LC^3 mix was composed of 15% limestone, 30% calcined clay, 5% gypsum, and 50% clinker. From the results, the flexural strength of the LC^3 mix was about 7.6 and 24.3% higher than those of the normal and PPC concrete after 28 days of curing, respectively (see Figure 6.1; AO – Normal concrete, AL – LC^3 concrete, and AP – PPC concrete).

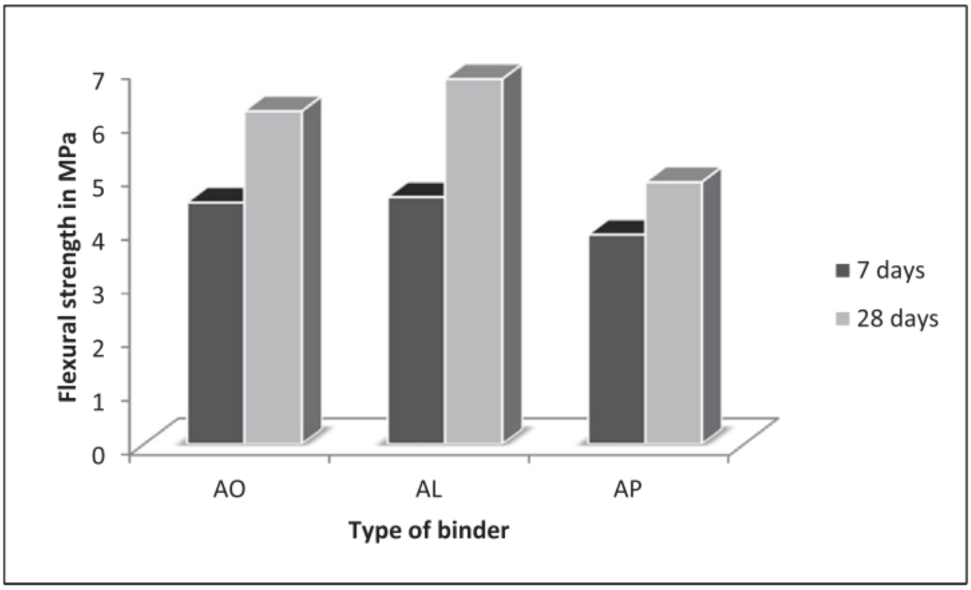

Figure 6. 1: Flexural strength of normal, LC^3 and PPC concrete [195]

Elastic modulus: 50% of cement clinker was replaced by 46% LC^2 and 4% gypsum to make LC^3 concrete specimens. Simultaneously, FA concrete samples with 30% FA and 70% PC were also manufactured, and both blended concrete samples were compared with the normal concrete specimens. Three w/b ratios of 0.35, 0.45, and 0.5 were used in the study. The results of the elastic modulus test indicated that all the mixes had comparable modulus of elasticity. For instance, for the 0.45 w/b ratio mix, the elastic modulus of the LC^3, the FA, and the normal concrete were 37.7, 33.7, and 35.8 GPa, respectively, at 28 days of testing [193].

Furthermore, Du & Pang [196] cast concrete specimens containing 100% PC, 70% PC + 30% LC^2, and 55% PC + 45% LC^2 to determine their mechanical properties. The elastic modulus of the specimens was determined after 1, 3, 7, 28, 91, and 182 days of casting. Based on the published data, the elastic modulus of concrete with 30% LC^2 showed the highest modulus of elasticity, followed by the samples with 45% LC^2 at all testing ages at 7 days and above. The better performance of LC^3 concrete can be traced to its denser microstructure due to its pozzolanic activity.

Creep: Ston et al. [197] carried out an experimental investigation on LC^3 and normal paste samples to determine their creep compliance. The LC^3 samples were manufactured by replacing PC with 50 and 35% of LC^2 content. The w/b ratio was kept constant at 0.4 during the investigation. After one month of casting and curing the specimens, the specimens were subjected to a sustained load of 7.3 MPa for up to 28 days. The results indicated that the paste with 35% LC^2 content exhibited the lowest creep, which was about 46.6 and 61.9% lesser than specimens with 50 and 0% of LC^2, respectively.

A similar investigation on creep behaviour of LC^3 and normal concrete was conducted by Scrivener et al. [198]. The normal mix contained 100% PC while the blended mix contained 30% calcined clay, 15% limestone, 5% gypsum, and 50% clinker. After curing the specimens in a sealed bag for 28 days, the samples were subjected to a loading corresponding to 15% of their characteristic compressive strength for 28 days. As per the reported data, the LC^3 specimen showed reduced creep values compared to the normal specimen. At 28 days of testing, the specific creep value of the LC^3 specimen was 63.5% lesser than that of the normal mix.

Shrinkage: The autogenous and drying shrinkage of high-performance mixes of normal and LC^3 pastes were investigated in the study of Du & Pang [196]. Two mixes of LC^3 specimens were made with 30 and 45% of LC^2 content by weight of the binder system, respectively. A w/b ratio of 0.3 was adopted. Although the autogenous shrinkage of the normal and blended concrete increases rapidly for the duration of testing (0-10 days), however, the strain value of the normal mix was lesser. For instance, the autogenous strain values of the specimens with 0, 30, and 45% of LC^2 were about 400, 700, and 800 micro-strains, respectively, at 10 days. The higher autogenous shrinkage noticed in the LC^3 samples can be attributed to the high pozzolanic reaction of the LC^3 system requiring consumption of more free water compared to the PC system. However, their drying shrinkage values showed that samples with 30 and 45% LC^2 exhibited 240 micro-strains at 182 days of testing while that of the normal mix was 365 micro-strains. LC^3 concrete showed a lower shrinkage value due to a refined pore structure that inhibits the outward movement of moisture.

In the same vein, the autogenous and total shrinkage values of concrete specimens made with a 0.45 w/b ratio were studied [193]. The specimens were made with 100% PC, 30% FA + 70% PC, and 46% LC^2 + 54% PC. After casting and curing the specimens for 28 days, they were wrapped with aluminium sheets before testing them for shrinkage up to 400 days. As per the results, the autogenous and total shrinkage of all the binders were very similar, although the concrete LC^3 specimen showed the highest shrinkage value.

6.1.2 Durability properties of LC^3 concrete

Freeze-thaw: The freeze-thaw resistance of LC^3 concrete was compared with that of normal, SF, and GGBS concrete in the study of Avet et al. [199]. The LC^3 mix was made with 55% PC, 30% calcined clay, and 15% limestone. For the SF and GGBS samples, PC was partially replaced with 10 and 50% replacement levels, respectively. A w/b ratio of 0.43 was adopted for the investigation. The specimens were exposed to 28 cycles of freezing and thawing (temperature variations) under the influence of a 3% NaCl solution. The mass loss after the investigation showed that the LC^3 showed the highest resistance to freeze-thaw by 62.5, 75.0, and 37.5% compared to the normal, SF, and GGBS concrete. It is recommended that more studies are conducted on the resistance of LC^3 concrete to freeze-thaw, as limited article was found on the subject matter.

Sulphate attack: The resistance to sulphate attack of mortar specimens consisting of 35% LC^2 content as a partial replacement for PC was compared with specimens made with 100% PC [200]. 0.5 w/b ratio was adopted for the two binder systems. After demoulding the specimens from the casting moulds, they were cured in sealed buckets containing water for 91 days to ensure the samples were well-hydrated before exposing them to a solution of 0.11 M Na_2SO_4. The change in length and mass of the specimens were observed for up to 719 days. The change in length of the LC^3 specimen at 719 days was 0.02%, while that of the normal mix was 0.60%, indicating lower resistance to sulphate attack of the PC-based mixture.

Furthermore, in a similar study, the clinker content of cement was replaced with LC^2 at replacement levels of 55, 45, and 35% [201]. A w/b ratio of 0.55 was adopted for the investigation. After casting and curing the specimens for 28 days, they were immersed in a 2% solution of $MgSO_4$ for 28 days. The specimen with 45% LC^2 content was found to be the most durable mix as its specimen exhibited the lowest weight loss.

Acid attack: The behaviour of LC^3 and PPC mortar was compared to that of normal mortar in an acidic environment [202]. The LC^3 blend consisted of 50% clinker, 30% calcined clay, 15% limestone, and 5% gypsum. Two w/b ratios of 0.5 and 0.6 were used. The specimens were cured for 90 days before immersing them in a 3% solution of H_2SO_4 for up to 120 days. The residual compressive strength and weight were used as indices for acid attack resistance. The LC^3 samples showed the highest resistance to acid attack, followed by the PPC samples and then the normal mixes. For instance, the loss in compressive strength of the LC^3 sample with a w/b ratio of 0.5 was 20 and 12% lesser than those with 100% PC and PCC, respectively. A similar trend was observed in the mass loss (see Figure 6.2).

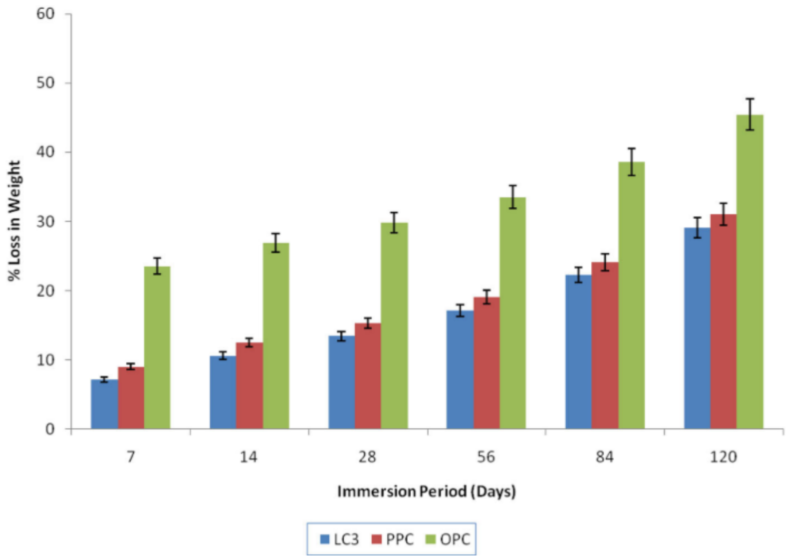

Figure 6. 2: Loss in weight due to acid attack for PC, PPC and LC^3 concrete [202]

Moreover, an investigation on acid attack resistance of LC^3 concrete was conducted by Bansal & Talakokula [203]. 50% of cement clinker was replaced by LC^2 content to make the LC^3 concrete specimens. A w/b ratio of 0.45 was adopted. The specimens were immersed separately in solutions with a high concentration of HCl and H_2SO_4 for 30 days. The weight loss of the LC^3 sample and the normal sample were similar for samples immersed in HCl, while the weight loss of the LC^3 sample (0.119%) was slightly higher than that of the normal concrete (0.064%) for samples immersed in H_2SO_4.

Chloride-induced corrosion: The corrosion resistance of reinforced concrete containing 100% PC, 30% FA + 70% PC, and 50% LC^2 + 50% PC was investigated [204]. A w/b ratio of 0.5 was adopted in the study. Three specimens were cast for each mix, and an impressed current corrosion test was conducted on them by subjecting the specimens to 15V for 10 hours daily until visible cracks were noticed on the specimens. After the test, the instantaneous corrosion current (I) of each specimen was determined. The LC^3 samples showed the lowest current (0-30mA), thereby indicating the highest resistivity to chloride ingress, followed by the FA samples (1—30mA), while those of the normal concrete samples ranged between 40-80mA.

Furthermore, the performance of concrete in chloride laden environment was studied [205]. Three different binder systems were used in the study: 100% PC, 80% PC + 13% flash calcined clay + 7% limestone (LC3-F), and 56% PC + 30% rotary calcined clay + 14% limestone (LC3-R). The w/b ratio was kept constant at 0.45 throughout the investigation. The specimens were subjected to a bulk diffusion test. From the results, the LC3-R specimen showed the highest resistance to chloride ingress. The chloride diffusion coefficients for the normal, LC3-F and LC3-R were 34.42×10^{-12} m^2/s, 8.18×10^{-12} m^2/s, and 7.97×10^{-12} m^2/s, respectively. The better performance of LC^3 samples can be attributed to their refined pore structure and chloride binding capacity.

Carbonation-induced corrosion: Khan et al. [206] performed an investigation on carbonation resistance of normal and LC^3 concrete. The LC^3 samples were cast with 15, 30, and 45% of LC^2 content by weight of the respective binder systems. 0.45 was used as a w/b ratio. The specimens were subjected to accelerated carbonation conditions (1% CO_2, 23^oC, and 55% RH) for up to 8 weeks. Simultaneously, another set of specimens were subjected to natural carbonation for 30 weeks. For the accelerated test, the carbonation depth for concrete containing 0, 15, 30, and 45% of LC^2 were 0, 7, 12, and 15 mm, respectively, while there was no carbonation observed for all the concrete specimens subjected to natural carbonation except for the specimen containing 45% LC^2 that showed carbonation depth of 7mm. It can be deduced that LC^3 concrete will be durable in a carbonation-driven environment so far; the cement replacement level does not exceed 30% and with an adequate concrete cover.

Díaz et al. [207] subjected LC^3 and normal concrete samples to natural carbonation in a marine area of high aggressiveness for up to 3 years. The RH in the area was above 80%. The results showed that the LC^3 concrete sample showed higher resistance to carbonation than the normal concrete. For instance, the carbonation depth of the LC^3 sample was 3mm after 3 years, while that of the normal concrete was 5mm. The higher resistance of the LC^3 sample may be due to the high RH (pores were filled with water) and refined pore structure that inhibits the movement of CO_2 into the concrete.

Alkali-silica reaction: Favier & Scrivener [208] studied ASR and sulphate attack resistance of LC^3 and normal mortar for several years. The replacement levels of PC with LC^2 were 15, 30, and 45%. The specimens were cast with a w/b ratio of 0.46 and cured for 28 days in water. Subsequently, they were exposed to a solution of 0.32 mol/litre NaOH for up to 1000 days. As per the results, all the LC^3 specimens do not exceed the expansion limit of 0.04%, while the normal concrete gave an expansion value greater than 0.2% within 100 days of exposure to the alkaline solution.

A similar investigation was conducted by Favier et al. [209]. In their study, the binder systems employed were 100% PC, 50% PC + 30% calcined clay + 15% limestone + 5% gypsum (LC^3-C), and 50% PC + 30% metakaolin + 15% limestone + 5% gypsum (LC^3 – M). High reactive aggregates were used in making the specimens. The specimens were separately exposed to solutions of NaOH with 0.32 M and 1.6 M for up to 180 days. The samples made with the two types of LC^3 showed similar expansion of about 0.02%, lesser than the expansion limit of 0.04%, while the normal specimens exceeded the limit for both alkaline concentrations, showing 0.25 and 0.34% for specimens immersed in solutions of 0.32 M and 1.6 M NaOH, respectively.

6.2 Limestone-fly ash concrete (LFA)

LFA blended cement consists of limestone powder, fly ash, and plain cement. The production of fly ash and limestone has been discussed in sections 5.1 and 6. 1, respectively. The LFA binder system is produced by either separately grinding or inter-grinding limestone powder, fly ash, and plain Portland cement together. Like FA, limestone is also relatively cheap compared to PC [210].

6.2.1 Mechanical properties of LFA concrete

Strength properties: The compressive and tensile strengths of normal and blended concrete samples were determined in the study of Sowjanya & Adiseshu [211]. The blended concrete specimens were manufactured using 80% PC + 20% FA, 75% PC + 20% FA + 5% limestone powder (LP), 70% PC + 20% FA + 10% LP, 65% PC + 20% FA + 15% LP, and 60% PC + 20% FA + 20% LP.

The specimens were cast and tested for compressive and tensile strength after curing them for 7, 28, 56, and 90 days. A w/b ratio of 0.45 was used. As per the compressive and tensile strength results, the normal concrete had higher strength values than the other blended mixes at 7 and 28 days of testing. However, from 56 to 90 days, concrete samples with 70% PC + 20% FA + 10% LP (optimized mix) showed the highest strength values. For instance, the compressive strength of the optimized mix was 1.5 and 8.38% greater than that of the normal concrete at 56 and 96 days of testing, respectively. The better performance of the ternary system over the binary system (FA concrete) can be attributed to the synergic reaction of FA and LP that causes more hydration products to be produced.

More so, the flexural strengths of normal and LFA concrete were determined [212]. The PC replacement levels with LFA were 29, 37, and 45%. All the samples were cast with a w/b ratio of 0.42 and tested for flexural strength after 28, 90, 180, and 360 days. According to the results, all the LFA concrete had flexural strength values greater than the control mix. The sample with 37% LFA (20% FA and 17% LP) exhibited the highest strength.

Elastic modulus: The elastic modulus of self-compacting concrete incorporating FA and LP was compared with that of the normal concrete [213]. 4 types of LFA samples were manufactured: 70% PC + 10% FA + 20% LP, 70% PC + 20% FA + 10% LP, 40% PC + 20% FA + 40% LP, and 40% PC + 40% FA + 20% LP. All the samples were tested for elastic modulus at 28 and 91 days. Only the blended sample with 20% FA + 10% LP showed similar values of elastic modulus with the normal concrete at all ages of testing, indicating 30% as a suitable replacement level of PC with LFA content (see Figure 6.3).

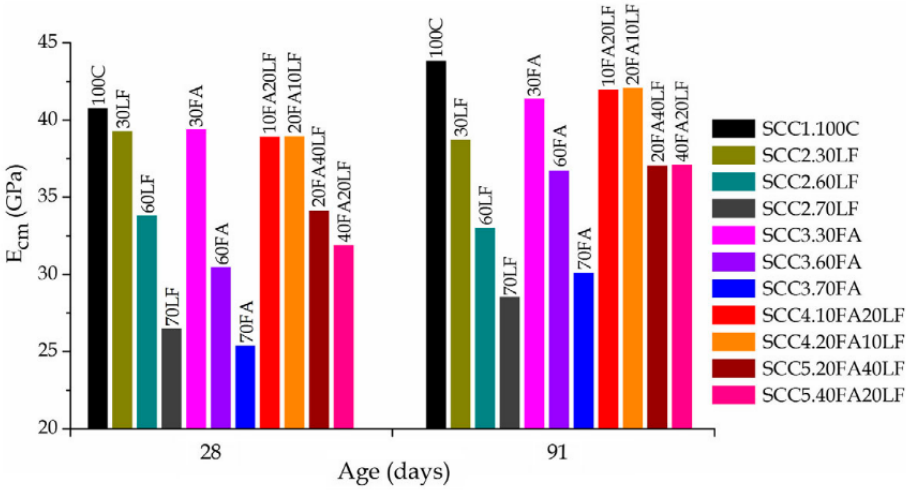

Figure 6. 3: Elastic modulus of different types of concrete [213]

Robalo et al. [214] determined the modulus of elasticity of concrete specimens consisting of 100% PC, 70% PC + 30% LP, and 50% PC + 28.5% FA + 21.5% LP (LFA concrete). Two prismatic specimens were cast for each mix and were tested for elastic modulus test after 28 days of curing. The elastic modulus of the PC, LP, and LFA concrete were 45.6, 42.4, and 47.9 GPa, respectively. The excellent performance of the LFA concrete can be traced to its refined pore structure due to the synergy action of LP and FA.

Creep: The creep behaviour alongside other properties of concrete were evaluated by Robalo et al. [215]. The samples were cast with 100% PC, 71.5% PC + 28.5% LP (LP concrete), and 50% PC + 28.5% FA + 21.5% LP (LFA concrete). Two prismatic samples were made for each mix and tested for compressive creep after 28 days of curing up to 210 days. Based on the published data, the normal concrete had the highest creep coefficient, which was about 25.6, and 39.1% higher than those of the LFA and LP concrete, respectively. The higher strengths of the blended samples compared to the normal concrete were responsible for their lower creep coefficients, as noted by the authors. It should be noted that limited literature was found on the creep behaviour of LFA concrete.

Shrinkage: In the study of Silva & de Brito [213], total shrinkage strains of normal and LFA concrete were determined. The shrinkage values of the specimens were measured daily for the first 14 days after demoulding them at 24 hours of casting and subsequently measured weekly for up to 182 days. The LFA mixes were manufactured with 70% PC + 10% FA + 20% LP, 70% PC + 20% FA + 10% LP, 40% PC + 20% FA + 40% LP, and 40% PC + 40% FA + 20% LP. The mix with 20% FA + 40% LP (60% LFA) showed the lowest shrinkage strain for all the testing duration. At 182 days, its shrinkage value was about 9.3% lesser than that of the normal mix.

Furthermore, the drying shrinkage of LFA concrete specimens over a period of 1 year was compared with that of the normal and FA concrete [216]. The LFA mixes were made with 38% PC + 38% FA + 24% LP (LFA 1) and 23% PC + 53% FA + 24% LP (LFA 2) while the FA concrete was made with 50% PC + 50% FA. The w/b ratios were 0.288, 0.32, 0.266, and 0.29 for the normal, FA, LFA1, and LFA 2 specimens. The high-water demand of FA powder caused the higher w/b ratio of the blended concrete. The use of superplasticizers could not produce workable concrete. The results showed that the drying shrinkage values were similar for all the mixes except the LFA 2 mix. The higher shrinkage exhibited by the LF2 sample can be traced to its higher w/b ratio compared to the normal mix.

6.2.2 Durability properties of LFA concrete

Freeze-thaw: The freeze-thaw resistance of roller-compacted concrete was evaluated by Huang et al. [217]. The percentage of replacement of PC with LFA ranged within 40-55%. All the concrete samples were prepared with a w/b ratio of 0.45. After curing the specimens, they were placed in the air at -25^{o}C (freezing) for three hours and subsequently exposed to thawing (immersed in water at 20^{o}C) for one hour. The specimens were exposed to 100 cycles of freezing and thawing, and their relative dynamic modulus of elasticity (RDME) was evaluated from the UPV test after each cycle. At the end of the investigation, the specimen made with 40% LFA (30% FA and 10% LP) showed the highest value of RDME (79%), indicating the high resistance to freeze-thaw of the sample, while the normal sample showed a lower value of RDME (43%). The better performance of the LFA sample could be attributed to its lesser void network compared to the normal mix. It should be noted that limited literature was found on the freeze-thaw resistance of LFA concrete.

Sulphate attack: The sulphate attack resistance of blended concrete was compared with the PC-based mortar [218]. PC was partially replaced with 30-50% LP to make the LP mortar, while 50% of PC was replaced by FA to make the FA mortar. The LFA mortar was manufactured with 50% PC + 30% LP + 20% FA. A constant w/b ratio of 0.5 was adopted. The specimens were immersed in a tank containing 2% of $MgSO_4$ after 28 days of curing for 90 days. Subsequently, the expansion of the specimens was evaluated and used as an index of sulphate attack resistance. Although the LP mortar with 50% LP content showed the lowest expansion at all testing ages, however, the expansion values of the LFA mortar were lower than all other samples. For instance, the LFA mortar had an expansion value that was about 15.4, 15.4, and 21.5% lesser than the LP mortar with 30% LP content, the FA mortar, and the normal mortar, respectively, at 90 days. It should be noted that limited literature was found on the sulphate attack resistance of LFA concrete.

Acid attack: Acid resistance of cement pastes consisting of LP and FA was investigated in binary and ternary binder systems [219]. The binary systems were made with 70% PC + 30% FA and 90% PC + 10% LP, while the ternary system was made with 70% PC, 20% FA, and 10% LP. All the mixes were designed with two w/b ratios of 0.25 and 0.40. Before exposing the samples to a solution of H_2SO_4 for 420 days (with a pH of 1), they were cured for 28 days in lime water. The weight loss of the specimens was determined weekly until 420 days. The results indicated that the ternary mix (LFA pastes) showed the lowest weight loss, followed by the FA pastes and then the LP pastes. The normal pastes showed the highest weight loss. For instance, the weight loss of LFA specimen made with a 0.4 w/b ratio was about 16% lesser than that of the normal paste at 420 days. It was also observed that pastes with a higher w/b ratio (0.4) showed lesser weight loss than the paste with a lower w/b ratio (0.25). This was due to the larger pore size of the paste with a w/b ratio of 0.4, which made it accommodate the expansive gel more than the other paste.

Furthermore, Liu et al. [220] separately immersed cement pastes made with 25% LP + 25% FA + 50% PC and 100% PC in acetic solution and sulphuric solution for up to 28 days. The compressive strengths of the specimens after the immersion showed that the LFA samples had higher resistance to acetic acid attack than the normal paste under the same condition.

Chloride-induced corrosion: Chloride penetrability of normal and blended concrete consisting of FA and LFA was investigated [221]. The FA samples were made by partially replacing PC with 40 and 60% PC and the LFA samples consisted of 60% PC + 30% FA + 10% LP and 40% PC + 45% FA + 15% LP. The w/b ratio adopted ranged between 0.4 – 0.5. The specimens were cast and cured in lime water for 56 days before conducting the rapid chloride penetrability test (RCPT). According to the results, the LFA sample with 40% LFA content showed the highest resistance to chloride penetrability as its coulomb values was about 7.9, 75.9, 60.5, and 51.3% lesser than those containing 60% LFA, 40% FA, 60% FA, and 100% PC, respectively.

In the same vein, the chloride penetration migration coefficients of concrete samples with different blends of LP and FA in binary and ternary forms were presented [222]. As per the results, all the binary and ternary mixes prepared for the study exhibited significantly higher chloride ingress resistance than the normal mix. The specimen made with 35% PC + 40% FA + 25% LP had the highest resistance, showing a chloride penetration coefficient that was about 81% lesser than that of the control mix (see Figure 6.4).

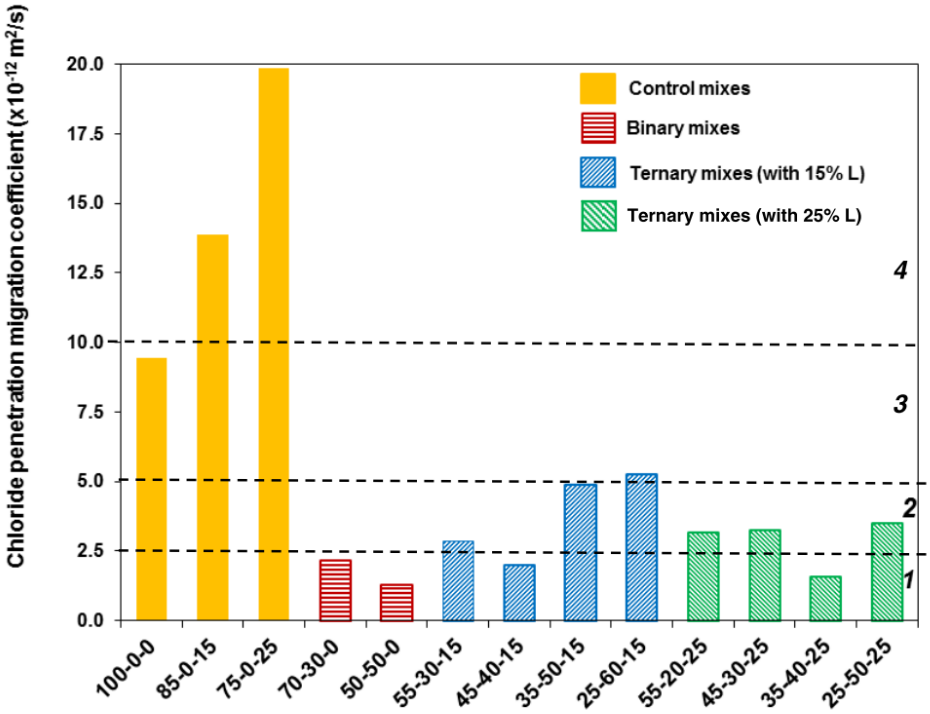

Figure 6. 4: Chloride migration coefficient of various types of concrete [222]

Carbonation-induced corrosion: In the study of Robalo et al. [215], carbonation resistances of normal and blended concrete were evaluated. The specimens were cast with 100% PC, 70% PC + 30% LP (LP concrete), and 50% PC + 28.5% FA + 21.5% LP (LFA concrete). Two specimens were prepared for each mix and exposed to accelerated conditions after curing for 7, 14, 28, and 42 days. The carbonation depths at 42 days of exposure to accelerated carbonation were 18.1, 11.0, and 12.9mm for the normal, LP and LFA concrete sample, respectively. The better performance of the binary and ternary mix over the normal mix could be attributed to the fact that the pore refinement ability of the binary and ternary mixes were dominant rather than the loss of alkalinity in the mixes.

Furthermore, concrete samples were exposed to acceleration carbonation (1% CO_2 and 75% RH) for 56 days and natural carbonation up to 365 days in the study of [212]. The blended samples were made with 29, 37, and 45% LFA content, and their results were compared with the normal mix. As per the results, the normal and the specimen with 29% LFA showed similar carbonation depths for both exposure conditions. For instance, the carbonation depths of the two types of specimens were 2.0 and 2.5mm at 180 days when exposed to natural carbonation, while the carbonation depths of the normal specimen and specimen with 29% LFA were 3.5 and 4.0 mm, respectively.

Alkali-silica reaction: Various combinations of LP and FA as binary and ternary systems were made to manufacture mortar samples [223]. The replacement levels of PC with FA and LP ranged between 0-30% (binary systems) while the ternary systems were made with 10% LP + 10% FA + 80% PC, 15% LP + 15% FA + 70% PC, and 20% LP + 20% FA + 60% PC. All the specimens were cast with a w/b ratio of 0.47. After curing them for two days, they were immersed in a solution of 1 N NaOH for up to 14 days. The results of the expansion tests indicated that the normal specimens showed the highest expansion values, indicating their lowest resistance to ASR. On the other hand, the average expansion value of the specimen made with 20% LP + 20% FA was 29.1, 26.2, 7.7% lesser than samples containing 100% PC, 20% FA, and 20% LP, respectively. The better performance of the ternary systems (LFA) over the binary systems and PC can be attributed to the synergic effect resulting from FA and LP. FA plays an essential role by reducing the alkalinity in the system due to pozzolanic activity, while LP acts as a filler to obtain a dense microstructure in addition to the fact that LP does not contain alkali content, unlike PC and FA. It should be noted that limited literature was found on the ASR resistance of LFA concrete.

6.3 Closure

This chapter presents the effect of two ternary SCMs; LC^3 and LFA on mechanical and durability properties of concrete discussed in chapter 3 and 4, respectively. Table 6.1 and 6.2 give a summary of the effect of these two ternary SCMs on mechanical and durability properties of concrete, respectively. A further discussion on the benefits and limitations of the various SCMs is presented in chapter seven.

Table 6. 1: Summary of the effects of LC^3 and LFA on the mechanical properties of concrete

Mechanical properties	**References and their results** **LC^3 Specimens**	**References and their results** **LC^3 Specimens**	**References and their results** **LFA Concrete**	**References and their results** **LFA Concrete**
Compressive strength	[193]; w/b = 0.45; 2, 7, 28 and 365 days concrete specimens, resp. 22.0, 32.0, 45.0 and 53.0 MPa– 0% 23.0, 33.5, 47.0 and 57.0 MPa – 46%		[211]; w/b = 0.45; 7, 28 and 90 days concrete specimens, resp. 29, 43, and 44 MPa – 0% 22, 41, and 46 MPa – 25% 24, 42, and 48 MPa – 30% 21, 36, and 39 MPa – 40%	
Tensile strength* and flexural strength**, respectively	[194]; w/b = 0.46; 7 days concrete specimens 3.15 MPa – 0% 3.17 MPa – 45%	[195]; w/b = 0.4; 7 and 28 days concrete specimens, resp. 4.75 and 6.5 MPa – 0% 4.90 and 7.0 MPa – 45%	[211]; w/b = 0.45; 7, 28 and 90 days concrete specimens, resp. 3.1, 3.5, and 3.5 MPa – 0% 2.8, 3.4 and 3.6 MPa – 25% 2.9, 3.5, and 3.8 MPa – 30% 2.7, 3.3, and 3.4 MPa – 40%	[212]; w/b = 0.42; 28, 90 and 360 days concrete specimens, resp. 5.0, 5.2, and 5.2 MPa - 0% 5.1, 5.3, and 5.4 MPa – 29% 5.1, 5.5, and 5.9 MPa – 37% 5.1, 5.4 and 5.6 MPa - 45%
Elastic modulus	[193]; w/b = 0.45; 28 days concrete specimens 35.8 GPa – 0% 37.7 GPa – 46%	[196]; w/b = 0.3; 3, 28 and 182 days specimens, resp. 35.0, 36.5, and 42.0 GPa – 0% 35.0, 37.5, and 43.0 GPa – 30 % 32.0, 37.0, and 42.5 GPa – 45%	[213]; w/b = 0.27 – 0.56; 28 and 91 days specimens, resp. 41.0 and 43.0 GPa – 0% 48.0 and 41.5 GPa – 30% 33.5 and 35.0 GPa – 60%	[214]; w/b = 0.47; 28 days concrete specimens 45.6 GPa – 0% 47.9 GPa – 50%
Creep	[197]; w/b = 0.4; creep compliance measurements at 14 and 28 days, resp. 80 and 103 MPa^{-1} – 0% 42 and 56 MPa^{-1} – 35% 61 and 75 MPa^{-1} – 50%	[198]; creep compliance measurements at 28 days 102 MPa^{-1} – 0% 51 MPa^{-1} – 45%	[215]; w/b = 0.67 (normal) and 0.72 (LFA); creep coefficient of 210 days concrete specimens. 2.12 – 0% 1.30 – 50%	
Shrinkage	[196]; w/b = 0.3; autogenous shrinkage at 10 days and drying shrinkage strains at 180 days, resp. 400 and 365 µm – 0% 680 and 240 µm – 30% 800 and 240 µm – 45%	[193]; w/b = 0.45; total shrinkage strain at 28 days 390 µm – 0% 430 µm – 46%	[213]; w/b = 0.27, total shrinkage strain measurements at 182 days 450 µm – 0% 440 µm – 30% 390 µm – 60%	[216]; w/b = 0.266 to 0.290; drying shrinkage measurements at 1 year. 445 µm – 0% 500 µm – 62% 710 µm – 77%

Table 6. 2: Summary of the effects of LC3 and LFA on the durability properties of concrete

Durability properties	**References and their results** **LC3 Specimens**	**References and their results** **LC3 Specimens**	**References and their results** **LFA Concrete**	**References and their results** **LFA Concrete**
Freeze-thaw resistance	[199]; w/b = 0.43, specimens subjected to 28 cycles of freezing and thawing and their mass loss were evaluated 4% - 0% 1% - 50%		[217]; w/b = 0.45; specimens subjected to 100 cycles of freezing and thawing and their relative dynamic elastic modulus were evaluated 43% - 0% 79% - 40% 33% - 55%	
Sulphate attack resistance	[200]; w/b = 0.5; specimens exposed to 0.11 M Na_2SO_4 for up to 719 days and their % change in length were recorded: 0.6% - 0% 0.02% - 35%	[201]; w/b = 0.55; specimens exposed to 2% solution of $MgSO_4$ for 28 days and their % loss in weight were evaluated. 12.4% - 0%; 9.3% - 35% 5.8% - 45% 11.6% - 55%	[218]; w/b = 0.5; specimens exposed to 2% $MgSO_4$ for 90 days and their expansion were measured. 350×10^{-6} – 0% 290×10^{-6} - 50%	
Acid attack resistance	[202]; w/b = 0.5 and 0.6; specimens subjected to 3% solution of H_2SO_4 for up to 120 days 43 and 46 % - 0% 27 and 32% - 45%	[203]; w/b = 0.45; specimens exposed to H_2SO_4 and HCl for 30 days, and their % loss in mass were evaluated, resp. 0.064 and 0.100 – 0% 0.119 and 0.114 – 50%	[219]; w/b = 0.25 and 0.4; specimens exposed to H_2SO_4 (pH of 1) for 90 days and their % loss in weight were evaluated, resp. 33 and 20% - 0% 25 and 5% - 30%	[220]; w/b = 0.3; specimens exposed to sulphuric (pH=2) and acetic acid (pH=4), for 28 days and their % loss in strength were evaluated, resp. 53 and 55 % - 0% 60 and 34% - 50%
Chloride-induced corrosion	[204]; w/b = 0.5; specimens subjected to impressed current corrosion test 40 - 80mA – 0% 0 – 30mA – 50%	[205]; w/b = 0.45; specimens were subjected to bulk diffusion test and their diffusion coefficient were determined 34.42×10^{-12} m^2/s – 0% 8.18×10^{-12} m^2/s – 20% 7.97×10^{-12} m^2/s – 44%	[221]; w/b = 0.4 – 0.5; specimens subjected to chloride penetrability test at 56 days (RCPT) 2465 coulombs – 0% 1199 coulombs – 40% 1294 coulombs – 60%	[222]; w/b = 0.35; one-year concrete specimens subjected to chloride migration test and their coefficient of migration were determined. 9.2×10^{-12} m^2/s – 0% 2.3×10^{-12} m^2/s – 45% 1.7×10^{-12} m^2/s – 65%
Carbonation-induced corrosion	[206]; w/b = 0.45; specimens subjected to 1% CO_2, 23ºC, and 55% RH for up to 8 weeks and their carbonation depths were recorded: 0 mm - 0 %; 7 mm – 15% 12 mm – 30 %; 15 mm – 45%	[207]; w/b = 0.42; specimens subjected to natural carbonation for 3 years at RH of 80% and their carbonation depth were recorded 5 mm – 0% 3 mm – 45%	[215]; w/b = 0.67 (normal) and 0.72 (LFA); specimen exposed to 5% CO_2, 23ºC, and 65% RH for up to 42 days and their carbonation depth were recorded 18.1 mm – 0% 12.9 mm – 50%	[212]; w/b = 0.42; specimens subjected to natural carbonation for one year and their carbonation depth were recorded 3.5 mm – 0%; 4.0 mm – 29% 8.0 mm – 37%; 8.0mm – 45%
Alkali-silica reaction	[208]; w/b = 0.46; specimens exposed to 0.32 mol/litre NaOH for up to 1000 days. concrete expansion test conducted were subsequently conducted >0.04% – 0%; < 0.04% – 15% <0.04% - 30%; <0.04% - 45%	[209]; w/b = 0.46; specimens exposed to solutions of NaOH with 0.32 M and 1.6 M for up to 180 days. 0.04% – 0% <0.04% - 50%	[223]; w/b = 0.47; specimens (prepared with 1.06% Na_2O) exposed to I N of NaOH for 14 days (mortar expansion test) 0.4538% - 0% 0.4386 – 20% 0.3602 – 30% 0.3216 – 40%	

7. DISCUSSIONS

This section presents an integrated discussion on the information provided in chapters 5 & 6 and gives a general overview of the effects of each SCM on the various properties of concrete. Table 7.1 and Table 7.2 presented in this chapter contain a summary of the optimum replacement levels, as per the reviwed texts, of the binary and ternary systems, respectively.

Strength

Based on the reviewed literature [94]–[96], it was evident that the inclusion of FA in the manufacture of concrete gives relatively low strength at an early age (i.e., before 56 days) due to the slow hydration of FA concrete, however, the inclusion of up to 30% FA in concrete increases the strength of the concrete at later ages (i.e., after 56 days). The better performance of FA concrete at a later age can be attributed to the reaction of FA with free lime in the cement paste, which resulted in the formation of additional cementitious materials (CSH), hence, increasing its strength. As per SF blended concrete, concrete containing 15% of SF was found to give the optimum strength value in terms of compressive and tensile strengths [125]–[127], while the optimum replacement level was found to be 12% for flexural strength [128]. It is important to note that SF, unlike FA, is a very reactive SCM that has a high rate of hydration, thereby producing concrete with higher strength compared to normal concrete at an early age (i.e., 3 days). A similar effect was noticed in MK concrete [149], [150], with optimum replacement level ranging between 10-15% as they both produce concrete with comparative strength values, even though 15% MK concrete produced a bit higher strength value. Hence, 10% could be suggested as the optimum replacement level considering the relatively high cost of MK in the market. The strength enhancement of MK concrete can be traced to the high pozzolanic reactivity of the concrete and pore refinement, making the concrete matrix denser than the normal concrete. Moreover, RHA concrete also showed higher strength compared to the normal concrete at all ages [172], [173], with 20% being postulated as the optimum replacement level for compressive strength. It was also noted that the strength properties of RHA concrete increase as its particle size reduces [174]. More so, with the same mix proportion, concrete made with LC^3 showed [193], [194] higher strength than normal concrete at all ages, with 46% as the optimum replacement level. Furthermore, due to the inclusion of FA in LFA concrete, LFA concrete showed lesser strength compared to normal concrete at an early age; however, the strength of normal concrete was less than that of LFA concrete at a later age (i.e., after 56 days) [211], [212]. Also, it is important to note that in the ternary mix, LFA concrete developed higher strength at an early age compared to binary mix, FA concrete. This can be traced to the nucleation effect of limestone that leads to the formation of more hydration products at an early age. Conclusively, SCMs such as SF, MK, RHA, and LC^3 improve the strength of concrete at all ages while FA and LFA improves the strength of concrete at a later age (i.e., after 56 days).

Elastic modulus

Concrete containing any amount of FA [97], [98] has shown a lesser elastic modulus compared to that of the normal concrete at an early age (i.e., before 56 days). This is due to the fact that compressive strength is directly related to elastic modulus from the reviewed literature. However, at a later age, when strength gain is high, the elastic modulus of FA concrete tends to be comparable (i.e., the same value or almost the same value) with that of the normal concrete. Furthermore, as expected, due to the higher strength of SF concrete, the results presented in the reviewed literature indicated that SF concrete exhibits a higher elastic modulus compared to that of normal concrete. The same trend was observed in MK concrete as their modulus of elasticity was higher than that of the normal concrete [151], [152], which is majorly due to the refinement of the concrete matrix and refined pores in the interfacial transition zone. More so, RHA is not only useful in concrete [174] but also in the making of building blocks [175] as the inclusion of RHA up to 15% improves the elastic modulus of such block. The presence of amorphous silica and fine particle of RHA can be responsible for the positive effect of RHA concrete on the modulus of elasticity. In the same vein, LC^3 concrete exhibited higher values of elastic modulus than normal concrete due to the relatively dense microstructure of LC^3 concrete [193], [196]. In addition, 50% was found as the optimum replacement level for LFA concrete in relation to elastic modulus as concrete containing such dosage exhibited slightly higher elastic modulus than normal concrete [214]. To conclude, due to strength development, SCMs such as SF, MK, RHA, and

LC^3 give comparable elastic modulus compared to that of normal concrete at all ages while FA and LFA concrete give comparable elastic modulus at a later age (i.e., 56 days).

Creep

According to [99], [100], the creep behaviour of FA and normal concrete are comparable at a later age (after 28 days). More so, the inclusion of SF in the manufacture of concrete was found to markedly reduce the creep strain of such concrete compared to that of the normal concrete [36], [129]. This indicates that SF concrete can be adopted in the manufacture of prestressed concrete as prestressed losses are majorly influenced by creep and shrinkage. Furthermore, at every age of testing, the creep measurement of MK concrete showed a comparative advantage over normal concrete as there was a significant reduction in creep of concrete with the inclusion of MK [153], [154], with 15% as the optimum replacement level. This positive effect can be traced to the improved aggregate-paste interface and denser paste matrix of MK concrete. Moreover, as expected, the creep behaviour of concrete was improved by adding up to 20% RHA in concrete at every age of testing [176], [177]. However, it is important to grind the RHA to fine particles as un-processed/ non-grounded RHA was found to give a higher creep strain than normal concrete [177]. In addition, creep compliance was another index that was used to assess the creep behaviour of normal and LC^3 concrete [197], [198], which concrete containing 35% LC^3 produced the optimum performance in relation to creep behaviour. A similar positive influence of including LFA in the manufacture of concrete was noticed [215], with 50% as the optimum replacement level. In general, the inclusion of appropriate dosage of SF, MK, RHA, LC^3 and LFA improves the creep behaviour of concrete at all ages while FA concrete gives comparable creep behaviour to that of normal concrete at a later age (i.e., after 28 days).

Table 7. 1: Optimum replacement level for the binary systems

Properties	References and their results Fly Ash Specimens	References and their results Silica Fume Specimens	References and their results Metakaolin Specimens	References and their results RHA Specimens
Strength	[94] w/b = 0.55; 3 and 91 days concrete specimens, resp. 24 and 29 MPa – 0% 18 and 31.5MPa – 20%	[125]; 7, 28, and 56 days concrete specimens, resp.; w/b = 0.30 & 0.35. 50, 70, and 77MPa &41, 50, and 51 MPa – 0% 61, 82, and 85MPa & 50, 63, and 68 MPa – 15%	[149]; w/b = 0.25; 7, 28 and 91 days concrete specimens resp. 45, 60, and 72 MPa – 0%; 48, 64, and 75 MPa – 10%	[169]; w/b = 0.53; 28 and 90 days specimens, resp. 37.1 and 38.3 MPa – 0% 42.5 and 46.0 MPa – 20%
Elastic modulus	[98] w/b = 0.50; 28 days concrete specimens 33.42 GPa – 0% 32.83 GPa – 30%	[129]; w/b = 0.35; 7 and 28 days concrete specimens, resp. 28.8 and 34.4 GPa – 0% 31.5 and 38.1 GPa – 15%	[151]; 28 days specimens 39.5 GPa – 0%; 50.0 GPa – 10%;	[175]; w/b = 0.7; block specimens of 60 days. 0.70 GPa – 0% 0.85 GPa – 15%
Creep	[100] w/b = 0.55; 30% stress/strength ratio, creep strain measurements at 14 days 15.5×10^{-6} – 0% 16.2×10^{-6} – 20%	[129]; w/b = 0.35; creep strain measurements at 7 and 28 days of loading, resp. 595 and 413 µm – 0% 417 and 328 µm – 15%	[154]; creep strain measurements at 200 days of loading: 280 µm – 0% 98 µm – 15%	[177]; w/b = 0.4; creep strain measurements at 100 and 200 days resp. 700 and 735 µm – 0% 439 and 473 µm – 20%
Shrinkage	[101] w/b ratio of 0.32 for 0% FA, 0.29 for 30% FA, and 0.30 for 50% FA; drying shrinkage measurements for 28 days and 6 months 347 and 554 microstrain – 0% 231 and 394 microstrain – 30%	[129]; w/b = 0.35; drying shrinkage strains after 587 days in air. 532 µm – 0% 512 µm – 15%	[43]; w/b = 0.35; drying shrinkage strain measurements at 7 and 42 days, resp. 400 and 682 µm – 0% 245 and 395 µm – 15%	[178]; w/b = 0.18; autogenous shrinkage strain after 12 hours and 28 days, resp. 1.0 mm/m and 2.05mm/m – 0% 1.0 mm/m and 0.2mm/m – 20%
Freeze-thaw resistance	[103] w/b = 0.44; specimens were subjected to 30 and 180 cycles of freezing and thawing and subsequently their comp. strengths were determined, respectively. 42.5 and 34.5 MPa – 0% 38.5 and 42.5 MPa – 30%	[133]; w/b = 0.19; specimens were subjected to 300 cycles of freezing and thawing and their mass loss rate were presented: 0.6% - 0% 0. 40% - 14%	[157]; w/b = 0.37 and 0.47; specimens subjected to up to 300 cycles of freezing and thawing and subsequently, their modulus of elasticity were determined resp. 25.36 and 24.1 GPa – 0% 34.08 and 32.74 GPa – 15%	[180]; w/b = 0.38 and 0.45; specimens subjected to 300 cycles of freezing and thawing and their compressive strength were measured. 56 and 49 MPa – 0% 62 and 51 MPa – 10%
Sulphate attack	[106] w/b = 0.45; specimens immersed in 5% sodium sulphate solution and their % weight loss were evaluated after 28, 56, and 90 days, resp. 2.14, 2.8, and 3.35% - 0% 1.9, 2.48, and 2.56% - 30%	[135]; w/b ratio = 0.45; specimens exposed to 5% Na_2SO_4 and $MgSO_4$ for 510 days, resp. and their strength, subsequently, were presented: 21.4 MPa (Na_2SO_4) & 31.1 ($MgSO_4$)- 0% 61.6 MPa & 32.8 MPa – 5%	[159]; w/b = 0.4; specimens exposed to 10% $MgSO_4$ for 28, 56, and 90 days and their flexural strength loss were evaluated and presented: 43, 42, and 52% - 0% 13, 26, and 27% - 15%	[182]; w/b = 0.5; specimens exposed to 5% Na_2SO_4 and $MgSO_4$ for 60 days and their compressive strengths were measured: 41.58 and 45.5MPa – 0% 51.40 and 55.49 MPa – 10%
Acid attack resistance	[106] w/b = 0.45; specimens exposed to 1% sulphuric acid and their % weight loss were evaluated after 28, 56, and 90 days, respectively 3.94, 4.54, 5.62% - 0% 1.61, 2.89, and 3.96% - 30%	[137]; w/b = 0.4; specimens were exposed to 1% HCl & H_2SO_4 for 28 days and their weight loss were evaluated: 2.31% (HCl) and 2.93% (H_2SO_4) – 0% 2.18% (HCl) and 2.78% - 10%	[160]; specimens immersed in 5% HCl for 90 days with w/b = 0.30, 0.35, 0.4, and 0.45, resp. and their % weight loss were evaluated: 8.26, 8.62, 9.04, and 9.44% - 0% 6.11, 6.32, 6.64, and 6.92 – 20%	[172]; w/b = 0.35; specimens were subjected to 5% HCl for 60 and 90 days and their loss in weight were evaluated. 12.0 and 20.5% - 0% 5.0 and 10.0% - 20%
Chloride-induced corrosion	[110]; w/b = 0.4, exposed to 3.5% NaCl for 28 and 90 days, respectively – RCPT. 3443 and 2916 coulombs – 0% 2775 and 887 coulombs – 40%	[139]; w/b = 0.6; resistivity measurements of embedded reinforced concrete at 20 and 50ºC: 30 and 15 Ωm – 0% 210 and 70 Ωm – 10%	[162]; w/b = 0.5; time to corrosion initiation were evaluated via half-cell potential measurements 136 days – 0%; 158 days -15%	[185]; w/b = 0.38; 28 days specimens exposed to 3% NaCl – RCPT 4503 coulombs – 0% 833 coulombs – 25%
Carbonation-induced corrosion	[112]; carbonation depths were measured after 140 days of exposure to 3% CO_2, 65% RH & 23ºC 8.0mm – 0%; 13.5mm – 20%	[141]; w/b = 0.50; carbonation depths were measured after 100 days exposure to 3 % CO_2 at RH of 100% and 25ºC 9.0mm – 0%; 9.5mm – 5%	[164]; w/b = 0.5 & 0.6; specimens exposed to 5% CO_2, 20ºC, and 65% RH for 3 weeks. w/b=0.5; 5.2mm & w/b = 0.6; 10.9mm – 0% w/b =0.5; 5.25mm & w/b = 0.6; 11.6mm - 5%	[188]; w/b = 0.46; specimens exposed to 5% CO_2 and 27ºC at RH of 45, and 60% for 28 days. 1.13 and 0.61 mm – 0% 1.18 and 1.14 mm – 5%
Alkali-silica reaction	[114]; w/b =0.42 to 0.45; reactive siliceous limestone was used; concrete expansion test conducted for 2 years. >0.04% – 0%; <0.04% - 60%	[143]; w/b = 0.45; reactive Spratt aggregate was used; concrete prism test was conducted for 1 year. > 0.04% - 0%; <0.04% - 12%	[167]; w/b = 0.3 and 0.4; highly reactive siliceous limestone was used; concrete expansion test conducted for 2 years. >0.04% – 0%; <0.04% - 20%	[189]; reactive aggregate was adopted; mortar expansion test for 28 days >0.2% -0%; <0.2% - 40%

Table 7. 2: Optimum replacement level for the ternary systems

Properties	**References and their results** **LC^3 Specimens**	**References and their results** **LFA Specimens**
Strength	[193]; w/b = 0.45; 2, 7, 28 and 365 days concrete specimens, resp. 22.0, 32.0, 45.0 and 53.0 MPa– 0% 23.0, 33.5, 47.0 and 57.0 MPa – 46%	[211]; w/b = 0.45; 7, 28 and 90 days concrete specimens, resp. 29, 43, and 44 MPa – 0% 24, 42, and 48 MPa – 30%
Elastic modulus	[193]; w/b = 0.45; 28 days concrete specimens 35.8 GPa – 0% 37.7 GPa – 46%	[214]; w/b = 0.47; 28 days concrete specimens 45.6 GPa – 0% 47.9 GPa – 50%
Creep	[197]; w/b = 0.4; creep compliance measurements at 14 and 28 days, resp. 80 and 103 MPa^{-1} – 0% 42 and 56 MPa^{-1} – 35%	[215]; w/b = 0.67 (normal) and 0.72 (LFA); creep coefficient of 210 days concrete specimens. 2.12 – 0% 1.30 – 50%
Shrinkage	[196]; w/b = 0.3; autogenous shrinkage at 10 days and drying shrinkage strains at 180 days, resp. 400 and 365 µm – 0% 800 and 240 µm – 45%	[213]; w/b = 0.27, total shrinkage strain measurements at 182 days 450 µm – 0% 390 µm – 60%
Freeze-thaw resistance	[199]; w/b = 0.43, specimens subjected to 28 cycles of freezing and thawing and their mass loss were evaluated 4% - 0% 1% - 50%	[217]; w/b = 0.45; specimens subjected to 100 cycles of freezing and thawing and their relative dynamic elastic modulus were evaluated 43% - 0% 79% - 40%
Sulphate attack	[201]; w/b = 0.55; specimens exposed to 2% solution of $MgSO_4$ for 28 days and their % loss in weight were evaluated. 12.4% - 0% 5.8% - 45%	[218]; w/b = 0.5; specimens exposed to 2% $MgSO_4$ for 90 days and their expansion were measured. 350×10^{-6} – 0% 290×10^{-6} - 50%
Acid attack resistance	[202]; w/b = 0.5 and 0.6; specimens subjected to 3% solution of H_2SO_4 for up to 120 days 43 and 46 % - 0% 27 and 32% - 45%	[219]; w/b = 0.25 and 0.4; specimens exposed to H_2SO_4 (pH of 1) for 90 days and their % loss in weight were evaluated, resp. 33 and 20% - 0% 25 and 5% - 30%
Chloride-induced corrosion	[204]; w/b = 0.5; specimens subjected to impressed current corrosion test 40 - 80mA – 0% 0 – 30mA – 50%	[222]; w/b = 0.35; one-year concrete specimens subjected to chloride migration test and their coefficient of migration were determined. 9.2×10^{-12} m^2/s – 0% 1.7×10^{-12} m^2/s – 65%
Carbonation-induced corrosion	[207]; w/b = 0.42; specimens subjected to natural carbonation for 3 years at RH of 80% and their carbonation depth were recorded 5 mm – 0% 3 mm – 45%	[215]; w/b = 0.67 (normal) and 0.72 (LFA); specimen exposed to 5% CO_2, 23ºC, and 65% RH for up to 42 days and their carbonation depth were recorded 18.1 mm – 0% 12.9 mm – 50%
Alkali-silica reaction	[209]; w/b = 0.46; specimens exposed to solutions of NaOH with 0.32 M and 1.6 M for up to 180 days. 0.04% – 0% <0.04% - 50%	[223]; w/b = 0.47; specimens (prepared with 1.06% Na_2O) exposed to I N of NaOH for 14 days (mortar expansion test) 0.4538% - 0% 0.3216 – 40%

Shrinkage

There is a contrasting behaviour in relation to the shrinkage of FA concrete among the reviewed literature. [101] results showed that the inclusion of FA reduces the shrinkage of concrete while that of [102] showed otherwise. This could be due to the fact that a higher amount of cement was used in the latter study [102], which contributed to higher shrinkage values. However, in general, when a moderate dosage of cement is used in making FA and normal concrete, the shrinkage of FA concrete is expected to be lesser or comparable to that of the normal concrete at a later age due to its slow rate of hydration. More so, the cited texts [129], [131] showed that the inclusion of SF reduces the total and drying shrinkage of concrete. This is due to the low diffusivity of SF concrete due to the fine pore structure of SF, and thus, the rate of water loss is reduced compared to that of normal concrete. However, SF concrete gives higher autogenous shrinkage due to its relatively high rate of hydration and increase in capillary tension that occurs due to the refinement of pore size distribution in SF concrete. Furthermore, the autogenous shrinkage of MK concrete was found to be higher than that of the normal concrete at an early age (i.e. 3 days) [155], which can be attributed to heterogeneous nucleation. However, the addition of MK in concrete reduces the autogenous shrinkage in the long run, as the pozzolanic activity of MK concrete becomes dominant. Moreover, as expected, the drying shrinkage of MK concrete was lesser than that of the normal concrete [43], with 15% as the optimum replacement level due to the dominant effect of the pozzolanic activity of MK concrete. More so, RHA concrete has followed the same trend as that of the MK concrete as the inclusion of RHA increases the autogenous shrinkage of concrete at an early age (i.e. 3 days) and reduces it at a later age (after 3 days) [178], [179]. Autogenous shrinkage was found to be dominant in the measurement of the total shrinkage of LC^3 concrete as its shrinkage strain was more than that of the normal concrete [193]. This is probably due to the higher rate of hydration in the LC^3 system. However, the drying shrinkage of such LC^3 concrete has also been found lesser than that of the normal concrete[193]. The filler effect of limestone and pozzolanic effect of fly ash was dominant in the LFA mix to produce LFA concrete with refined pore structures as its drying shrinkage was lesser than that of the normal concrete [213]. A general conclusion could be that drying shrinkage of concrete incorporating SF, MK, RHA, and LC^3 are lesser than that of the normal concrete at all ages while their autogenous shrinkage strains are lesser compared to the normal concrete at a later age (i.e., 10 days). Additionally, drying and autogenous shrinkage of FA and LFA concrete are improved at a later age (i.e., 28 days).

Freeze-thaw

For FA specimens, the reviewed texts [103], [105] showed that FA concrete showed better performance in an environment subjected to various cycles of freezing and thawing. This could be traced to the fact that FA concrete is relatively denser and less absorptive. For SF concrete, relative dynamic modulus [132] and mass-loss rate [133], to which 14% replacement level was found to be optimum, were used as indices for assessing the resistance of concrete to freeze-thaw action. SF concrete showed better performance than the normal concrete. This performance can be due to the reduction in the binder porosity due to the filling of micro voids by SF. A similar trend was noticed in MK concrete as they exhibited better performance to freeze-thaw actions than normal concrete [156], [157]. The same trend was also noticed in RHA concrete [180], [181], with 10% as the optimum replacement level. This better performance of RHA in freezing and thawing environment can be attributed to the unique microstructural property of RHA, which make it possible to reduce the air void spacing factor. Moreover, LC^3 and LFA concrete also showed better performance to the action of freezing and thawing [199], [217] compared to normal concrete. Conclusively, all the reviewed SCMs could improve the resistance of blended concrete to the actions of freezing and thawing due to a relatively denser microstructure of blended concrete compared to the normal concrete.

Sulphate attack

Although specimens exposed to $MgSO_4$ deteriorates more rapidly than the ones exposed to Na_2SO_4 of equal concentrations, however, in relation to normal concrete; FA concrete, with 30% as optimum replacement level, gives a better resistance to this form of attack compared to normal concrete [106], [107]. However, a worse case was noticed with SF [135] immersed in a solution of $MgSO_4$ as any addition of more than 5% SF to concrete produced concrete of less durability compared to that of the normal concrete. This occurrence can be justified by decalcification (which is most common in SF concrete), M-S-H formation, which occurs in such concrete, eventually leading to cement bond destruction. Unlike SF, as per the reviewed texts, MK concrete was found to be more durable than normal concrete under $MgSO_4$ attack [158], [159], with 15% as the optimum replacement level. This can be attributed to the reduction in the amount of portlandite in MK concrete; hence, the quantity of gypsum formed in MK concrete will be lesser than that of the normal concrete. Surprisingly, RHA concrete exposed to sulphate attack for 60 days was found to lose no strength while there is strength reduction for the normal concrete [182]. This indicates that the pozzolanic reaction of RHA to form additional calcium silicate hydrate (CSH) was dominant rather than the reaction of sulphate ions with the cement paste to form the expansive gel, ettringite at the early age of reaction. A positive influence of including LC^3 and LFA in the manufacture of concrete exposed to sulphate attack was also noted [201], [218], [224]. Generally, all the SCMs have the potential to produce a more durable concrete in sulphate-rich environment compared to the normal concrete as pozzolanic activity of the SCMs would cause less availability of portlandite in the blended concrete, thereby reducing the formation of gypsum.

Acid attack

After exposure to an acidic environment, weight loss of the FA specimens [106], [108] was used as an index of evaluating their resistance to acid attack, which revealed that concrete containing up to 70% FA (by weight of cement) is durable in an acidic environment. The same trend was noticed for SF concrete exposed to hydrochloric and sulphuric acid environments [136], [137]. The relatively high performance of SF concrete in an acidic environment is traced to the less availability of portlandite in the concrete which thus makes it to be less susceptible to acid attack. However, MK concrete under a highly aggressive acidic environment (5% H_2SO_4) was found less durable than the normal concrete as its loss in weight was slightly higher than that of the normal concrete [161]; meanwhile, MK concrete was more durable in a less aggressive environment (5% HCl) [160]. Furthermore, RHA concrete was found to be more durable in an acidic environment than normal concrete [172], [184]. It was also noticed that an increase in the curing temperature of RHA concrete increases its resistance to acid attack. This is a phenomenon that can be traced to the increase in crystallinity of supplementary hydrates as crystalline CSH is attacked less by acid compared to amorphous CSH. In the same vein, LC^3 and LFA concrete was found to be more durable in an acidic environment compared to normal concrete [202], [203], [219]. Conclusively, the incorporation of appropriate dosage of SCMs in concrete would increase the resistance of such concrete to acid attack (but maybe less durable in environment of high concentration of sulphuric acid) due to the less availability of portlandite in blended concrete, to which the acid reacts with.

Chloride-induced corrosion

Although FA concrete containing up to 60% was found to be more durable than normal concrete when exposed to a chloride-rich environment, however, binder system containing 40% of FA was found to give the optimum performance in terms of chloride permeability [110]. Furthermore, as per the reviewed texts, half-cell potential measurements [138] and resistivity measurements [139] were used to determine the resistance of normal and SF concrete to chloride-induced corrosion, to which the SF concrete showed better performance. Pore structure refinement of SF concrete can be said to be the major factor contributing to its effectiveness in mitigating chloride induced corrosion. The initial time to corrosion [162] and amount of bonded chloride ions in concrete [163] were used to access the resistance of normal and MK concrete to chloride-induced corrosion, to which MK concrete gave better performance. The ability of MK concrete to bind more chlorides can be attributed to the formation of Friedel's salt produced from the reaction of C_3A with chlorides. More so, the chloride penetrability of RHA concrete was found to be lesser than that of normal concrete due to its (RHA concrete) refined pore structure [185], [186]. For LC^3 concrete, bulk diffusion test and impressed corrosion test [204] [205] were used to assess their chloride-induced corrosion property, to which 50% was found as the optimum replacement level. More so, 65% was found as the optimum replacement level for LFA concrete in a chloride-rich environment [222]. Generally, the inclusion of SCMs improves the resistance of concrete in chloride-rich environment due to chloride binding and pore refinement abilities of blended concrete.

Carbonation-induced corrosion

The reviewed texts [112], [113] indicated that the increase in the quantity of FA in the manufacture of concrete increases the carbonation depth of such concrete. This is due to the reduction of calcium hydroxide in concrete as the FA content increases, which subsequently lowers the alkalinity of concrete. Like FA concrete, the inclusion of SF in concrete increases the carbonation depth [141], [142], but concrete containing 5% SF has shown comparable carbonation depths with that of the normal concrete. Meanwhile, irrespective of SF content, an increase in the w/b ratio of concrete significantly increased the carbonation depth. Hence, the w/b ratio can have more influence on carbonation depth than the amount of SF. The same trend was observed in MK concrete as an increase in w/b ratio increases the carbonation depth [164], [165]. However, the inclusion of 10% MK in concrete reduces the depth of carbonation. This means that the detrimental effect (for carbonation) of portlandite consumption in the MK concrete was lesser than the beneficial effect of pore refinement. Furthermore, as expected, carbonation depth increases with an increase in RHA content [187], [188]. However, it decreases with an increase in RH, which can be a result of the fact that the pores of the concrete will be saturated with water at higher RH, thereby inhibiting the ingress of CO_2 into the concrete. Additionally, the addition of LC^3 increases the carbonation depth of concrete exposed to accelerated carbonation [207]. However, LC^3 concrete was more durable than normal concrete when exposed to natural carbonation. This means that LC^3 concrete would be more durable than normal concrete in a real-life situation, provided that an adequate cover is used. Interestingly, the pore refinement ability of LFA concrete rather than the effect of portlandite consumption when exposed to carbonation was dominant as per results reported in the reviewed texts [212], [215]. A general conclusion could be that the inclusion of SCM increases the depth of carbonation of concrete due to the consumption of portlandite in blended concrete, however, in real-life situations; blended concrete can provide adequate resistance to carbonation as the ingress of the carbonation front to the surface of the embedded steel in concrete is naturally slow especially in a dense blended concrete.

Alkali silica reaction

FA concrete containing up to 60% was found to give adequate and higher resistance to ASR compared to normal concrete [114], [115]. This trend was also noticed in the mortar bar, and concrete prism expansion tests conducted on SF and normal concrete [142], as concrete containing 12% SF gave the optimum resistance to ASR while concrete containing less than 8% SF gave undesirable expansion, though better than that of the normal concrete. The inhibition of alkali and water movement in SF concrete can justify its better performance in an ASR-rich environment. More so, as expected, MK concrete produced lesser expansion due to ASR compared to the normal concrete, with 15% optimum replacement level [166], [167]. The entrapment of alkalis by additional hydrates produced by MK concrete and lower pH of MK pore solutions can be attributed to the effectiveness of MK concrete in mitigating expansion due to ASR. In addition, RHA concrete was found to be more durable than normal concrete in an ASR-rich environment as RHA concrete exhibited a lesser amount of expansion than the allowable expansion limits for both concrete and mortar bar test [189], [190]. A similar trend was also noticed in LC^3 concrete as the inclusion of 50% LC^2 in concrete gives optimum performance in relation to ASR [209], [211]. Although the expansion limit of 0.1% was exceeded for normal and LFA mortar [223], which is due to the high alkali content present in the plain cement adopted; however, the expansion exhibited by the LFA concrete was significantly lesser than that of the normal concrete. In conclusion, the inclusion of appropriate dosage of the reviewed SCMs have the tendency to increase the resistance of such blended concrete to expansion caused by ASR due to their (blended concrete) alkali binding ability, refined pore matrix and impermeable pore structure that inhibit the movement of alkalis and water.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1. Conclusions

This study presents a review of the mechanical and durability properties of blended concrete and mortar, using different SCMs. Four binary systems: FA, SF, MK, and RHA, and two ternary systems: LC^3 & LFA, are reviewed. It is clear from the previous chapters that the inclusion of appropriate dosages of the different SCMs in the manufacture of concrete and mortar can affect their properties.

Prior to concluding this study, it should be noted that this study is purely based on reported data in the literature or previous studies, and the results may vary in different conditions. For instance, blended concrete and mortar properties may be altered by different sources and processing methods of SCMs, curing method & duration, construction method, workmanship, etc. Hence, to achieve a favourable result on a particular concrete property, it would be better to conduct trial mixes or get information from previous relating projects. The following general conclusions can be made:

- The strength properties (compressive, tensile, and flexural strength) and elastic modulus of concrete incorporating FA and LFA are lower than reference concrete at an early age (i.e., before 56 days) due to their slow pozzolanic reaction. However, the inclusion of the SCM can show higher strength at a later age (i.e., after 56 days).

- Due to their high pozzolanic activity, SCMs such as SF, MK, RHA, and LC^3 can improve the strength of concrete at all ages.

- Due to strength development, SCMs such as SF, MK, RHA, and LC^3 give higher or comparable elastic modulus compared to that of normal concrete at all ages while FA and LFA concrete give higher or comparable elastic modulus at a later age (i.e., 56 days).

- Concrete containing appropriate dosage of SF, MK, RHA, LC^3 and LFA improves the creep behaviour of concrete at all ages while FA concrete gives comparable creep behaviour to that of normal concrete at a later age (i.e., after 28 days).

- Drying shrinkage of concrete incorporating SF, MK, RHA, and LC^3 are lesser than that of the normal concrete at all ages while their autogenous shrinkage strains are lesser compared to the normal concrete at a later age (i.e., 10 days). Additionally, drying and autogenous shrinkage of FA and LFA concrete are improved at a later age (i.e., 28 days).

- The SCMs reviewed in this study could improve the resistance of blended concrete to the actions of freezing and thawing due to their relatively denser microstructure compared to the normal concrete.

- The SCMs reviewed in this study have the potential to produce a more durable concrete in sulphate-rich environment compared to the normal concrete due to their pozzolanic activity that facilitates less availability of portlandite in the blended concrete, which react to form the expansive gels.

- The incorporation of appropriate dosage of the reviewed SCMs in concrete would increase the resistance of such concrete to acid attack (but maybe less durable in environment of high concentration of sulphuric acid) due to the less availability of portlandite in blended concrete, to which the acid reacts with.

- Due to chloride binding and pore refinement abilities of blended concrete, the inclusion of the reviewed SCMs in the manufacture of concrete can improve the resistance of such blended concrete in chloride-rich environment.

- Generally, the inclusion of SCMs reduces the carbonation resistance of concrete due to the consumption of portlandite. However, lower dosage of SCMs (i.e., 5-10%) with a long curing period can show comparable carbonation depths with those of normal concrete.

- The inclusion of appropriate dosage of the reviewed SCMs have the tendency to increase the resistance of such blended concrete to expansion caused by ASR due to their (blended concrete) alkali binding ability, refined pore matrix and impermeable pore structure.

- The utilization of SCMs in the cement and concrete industry helps in reducing carbon footprint.

- The synergistic effect of two SCMs (i.e. ternary system) seems to give more benefits than the corresponding individual SCM (i.e. binary system).

- Since many of the SCMs are by-products and agro-waste materials, these SCMs can mitigate environmental and economic issues caused by cement production.

8.2. Recommendations

The following recommendations are made based on the literature review for future research and direction in the study domain:

- It is recommended that more experiments should be conducted on freeze-thaw resistance of LC^3 concrete and creep behaviour, freeze-thaw resistance, sulphate attack resistance, and ASR resistance of LFA concrete as limited articles are available on the respective subject matter.

- It is recommended that the reviewed SCMs in this study should be standardized where applicable.

- Most of the previous studies on the reviewed properties of the different SCMs focused on either laboratory or field experiments. In order to have more robust results, it is recommended that laboratory and field investigations are carried out together so that the results could be correlated with one another.

- It is recommended that cost benefit analysis of various SCMs should be conducted, as it can increase the awareness of SCMs in the construction industry.

- Favourable policy on the adoption of various SCMs should be developed by government or relevant agencies where applicable.

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