SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

UNIVERSITY OF THE WITWATERSRAND, JOHANNESBURG

An investigation into the use of shredded rubber as aggregate in concrete

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A research report submitted to the Faculty of Engineering and the Built Environment, University of the Witwatersrand, Johannesburg, in partial fulfilment of the requirements for the degree of Master of Science in Engineering.

Johannesburg, 2020

DECLARATION OF PLAGIARISM

I, JEEN VARGHESE KOCHUVADAKKEL, (Student number: 1762995), registered for CIVN-7019A-Research Program, for completion of Master of Science, in Engineering in School of Civil and Environmental Engineering. I hereby declare the following:

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March 24 2020

Re: Varghese Kochuvadakkel (1762995)

To whom it may concern,

Jeen Varghese Kochuvadakkel (1762995is currently registered as a Master Degree student at the School of Civil and Environmental Engineering, University of the Witwatersrand, Johannesburg. This letter is to confirm that, at the time of writing, Jeen does not need ethical clearance for his study entitled '*AnInvestigation Into The Use Of Shredded Rubber as Aggregate in Concrete*. This decision has been reached based upon a description of the project supplied by Jeen to the School of Civil and Environmental Engineering Ethics Committee, constituted as a subcommittee of the University Human Research Ethics Committee (Non-Medical), which has been evaluated by the subcommittee chair. This decision has then been ratified by the University Human Research Ethics Committee (Non-Medical). If, however, Jeen changes the methods of data collection and analysis for this project, this decision may no longer be valid. If such changes take place, this should be communicated to the School of Civil and Environmental Engineering Ethics Committee.

Please feel free to contact me should you require any further information.

Thank you.

Yours sincerely,

Prof John Ndiritu,
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ABSTRACT

Numerous studies are available on incorporating waste tyre rubber for aggregates in concrete, for use in the construction industry. However, limited information is available on the extent to which higher percentage of rubber, replacing aggregates can be introduced so as to use them in non-structural works. This study focuses on analyzing the compressive strength and durability properties of the concrete, when fine aggregates are replaced at 7.5, 15, 25, 35 and 50% by volume with shredded tyre rubbers. The characteristics of rubberized concrete to changes in water-to-binder ratios (0.40 and 0.55) and binder types 70/30 Portland Cement (PC)/fly ash (FA) and 50/50 PC/ground-granulated blastfurnace slag (GGBS) are studied by varying one parameter at a time.

Concrete cubes were cast and compressive strength tests conducted at 7, 28 and 56 days, and durability index tests at 28 and 56 days.

The results indicated significant reduction in compressive strength of mix concrete with increase in percentage of rubber content replacing fine aggregate by volume. The addition of fly ash binder in rubberized concrete have beneficial effects on compressive strength compared to GGBS binder. The compressive strength of rubberized concretes was greatly influenced by a change in water binder ratio. At lower water binder ratio of 0.40, irrespective of binder type, the concrete specimens containing up to 50% replacement of fine aggregates with rubber particles, attained the acceptance criteria of at least 2 MPa above minimum characteristic strength required for plain and reinforced non-structural concrete works. At higher water binder ratio of 0.55, the acceptance strength requirement for non-structural plain concrete works was attained with the maximum of 50% in FA concrete, and with concrete containing only up to 35% rubber particles in GGBS concrete. However, for reinforced non-structural works, acceptance requirement was attained with only up to 25% fine aggregate replaced with rubber content in GGBS concrete and with 35% rubber content in FA concrete with prolonged curing.

For the parameters tested, the OPI of FA binder rubberized concrete exhibited higher values compared to rubberized concrete with GGBS binder, indicating better permeability, thereby concrete quality. GGBS rubberized concrete exhibited better water sorptivity and chloride conductivity indices compared to FA concrete. The extent to which change in water binder ratio and binder type affects the durability properties of rubberized concrete, varies depending on the percentage of rubber particles incorporated in the mix design. Increasing percentage of rubber particles decreased the sorptivity indices indicating better quality of concrete. Furthermore, increase in rubber content in concrete proved to be detrimental to concrete durability with respect to chloride conductivity, which improved with prolonged curing. Irrespective of the variations with water binder ratio and binder type, rubberized concrete exhibited good durability properties with respect to oxygen permeability, water sorptivity and chloride conductivity.

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I would like to thank my family for their support, encouragement and prayers. A special mention to my husband and parents for never having had doubts about my ability to successfully see through my postgraduate studies.

I dedicate this thesis to my family, in particular to my husband Alexander Jimmy George and daughter Megan George.

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CHAPTER 1

INTRODUCTION

1.1 Background

South Africa like many other developing countries is experiencing a growth rate in the construction industry. GDP from construction in South Africa averaged 54070.96ZAR Million from 1960 to 2018 (GDP, 2018). This involves the usage of concrete in large scale, for the development of infrastructure. However, this increasing growth in the economy leads to depletion of natural resources. Natural mineral aggregates (e.g. granites, andesites, etc) have been used to make concrete for many years. However, with the increasing demand for concrete as a construction material, natural aggregates are fast becoming depleted, and carefully selected alternative materials have had to be sought to partially or fully replace the natural aggregates. Some of these materials include recycled concrete, mine tailings and shredded rubber tyres. This study focuses on the use of the latter, specifically on shredded rubber tyres.

South Africa generates 111 megatons of waste per annum according to Department of Environmental Affairs (DEA). 75 percent of this waste ends up in landfills (DEA, 2018). In South Africa the rate of recycling waste rubber tyres is less than 20 percent (Sebola, Mativenga and Pretorius, 2018). According to the Recycling and Economic Development Initiative of South Africa (REDISA) annual report, in 2014 the tyre waste arising in South Africa was 177 385 tons. Hence, understanding the performance of rubberized concrete will therefore encourage their use. Furthermore, less waste will be directed to landfill sites and natural resources required for conventional concrete are conserved.

1.2 Scrap rubber tyre as an alternative fine aggregate in concrete

The waste tyre rubber material added to concrete is generally classified by size rather than use or composition (Mushunje, Otieno and Ballim, 2018). Car and truck tyres are the two classes into which tyre maybe divided. They differ from each other regarding constituent materials, example natural and synthetic rubber. The three main categories of discarded tyre rubber considered are chipped, crumb and ground rubber. Generally, chipped or shredded rubber is used to replace the coarse aggregates, usually ranging from 13 – 76 mm in size. Crumb rubber is used to replace sand and this rubber is manufactured by special mills where big rubber particles are ground to particles of about 0.425 – 4.75 mm in dimension. Ground rubber is used to replace cement. In this category particles of 0.075 – 0.475 mm in dimension are used (Panda, Parhi and Jena, 2012) (Mushunje, Otieno and Ballim, 2018).

Scrap tyres are considered as a whole tyre, a slit tyre, a shredded or chipped tire, a ground rubber or as a crumb rubber product. Ground rubber maybe sized from particles as large as 19mm to as fine as 0.15mm. Crumb rubber usually consists of particles ranging in size from 4.75mm to less than 0.075mm. The production of shredded rubber tyres involves primary and secondary shredding (Federal HighwayAdministration, 2016).

According to South African Department of Environmental Affairs, 111megatonnes of waste is generated per annum, of which 75% ends up in landfills. 174,199tonnes of this comes from waste tyres in South Africa (DEA, 2018). According to the electronic National Administration Traffic Information System (eNatis) over 12million vehicles were registered in South Africa in 2017. According to (Zvimba *et al.*, 2013), South Africa has over 60 million stockpiled waste tyres and this figure increases by about 10 million every year.

Much effort has been initiated to investigate use of recycled tyre rubber in construction industry. The most common application is rubberized asphalt concrete that has proved effective for decades. In recent years, application of recycled tyre rubber in concrete manufacturing has been investigated by considering replacement of aggregates with tyre crumbs. Rubber has lower stiffness compared to aggregates and leads to earlier onset of micro-cracking in the concrete matrix, with a significant decrease in compressive strength. The decrease in strength, limits the use of tyres for replacement on aggregates in structural concrete (Waris *et al.*, 2018). Thus, cheaper or more environmentally friendly options as alternatives for aggregates yet yielding adequate performance in concrete applications are considered. Therefore, to manage over 60 million stockpiled tyre waste in South Africa, and conserve natural aggregate resource, this study investigates the use of waste rubber tyre as partial replacement of fine aggregates in non-structural concrete; defined as concrete with low strength (<25N/mm^2) used only when small compression or temporary loading is involved (Concrete Society, 1966). SANS 10100- Part 2 and 3 specifies a minimum characteristic cube strength of 15 MPa is specified for plain concrete and 20 MPa if the concrete is to include embedded metal (SANS10100-2, 2014; Viljoen, 2015). This type of concrete could be applied in the construction of ditches, sidewalks and other similar civil works (López-Uceda *et al.*, 2016). The study focuses on the influence of binder type, water-to-binder ratio and rubber tyre replacement percentage on durability properties and compressive strength of concrete. Effect of the strength parameter and durability properties are interpreted against the mix prepared of OPC/FA and OPC/GGBS that had 0% tyre rubber replacement. This helps in better understanding the effects of utilizing rubber tyres with different binder type and water-to-cement ratio.

1.3 Problem statement

The steady growth in concrete demand leads to several environmental and economic impacts. The feasibility of utilizing shredded rubber wastes in concrete for non-structural concrete works, provides significant environmental and economic gains. Therefore, furthering the understanding of the waste rubber, as alternative aggregate promotes increased use of rubber waste in concrete.

In order to improve the understanding of using shredded rubber waste in non-structural concrete, the following needs to be understood:

a) The effect of water-to-binder ratio and binder type, on the compressive strength of concrete, at different percentage of shredded tyre rubber replacing fine aggregate by volume.
b) The effect of water-to-binder ratio and binder type, on the durability properties of concrete, namely, oxygen permeability, water sorptivity and

chloride conductivity, at different percentage of shredded tyre rubber replacing fine aggregate by volume.

1.4 Justification for focusing on non-structural concrete

Addition of rubber particles to replace mineral aggregates in concrete, affects the compressive strength of concrete, as aggregates are replaced with relatively weaker rubber. Hence, the replacement level of aggregates by scrap tyre rubber above 20% is not recommended to accomplish the desired qualities to be used in structural concrete works. This is supported by many research works (Topcu and Avcular, 1997; Oikonomou and Mavridou, 2009; Topçu and Unverdi, 2018) (Waris *et al.*, 2018). Although these small levels of replacement make a difference, these applications do not generate enough volume to make a noticeable dent in the existing stockpiles of scrap tires.

This study focuses on maximizing the use of rubber content to manage the piles of rubber waste. Hence a replacement level up to 50% is investigated. As a drop in the strength of concrete is anticipated, the potential of using this concrete in non-structural works is studied. By maximizing rubber content, a minimum characteristic strength of 15 MPa, required for use in plain non-structural work is anticipated.

1.5 Aim and objectives

The aim of this study is to investigate the feasibility of replacement of natural fine aggregates in concrete with waste shredded rubber tyres from industry and the effect of binder content and type on the workability, compressive strength, water absorption, oxygen permeability and chloride conductivity of a non-structural concrete.

This aim will be achieved by fulfilling the following objectives:

1. Analyzing the effect of water-to-binder ratio and binder type on rubberized concrete; and the sensitivity of compressive strength of concrete to changes

in these parameters, prepared by varying percentage of shredded rubber replacing fine aggregate.

2. Analyzing the effect of water-to-binder ratio and binder type on rubberized concrete; and the sensitivity of durability properties of concrete to changes in these parameters, prepared by varying percentage of shredded rubber replacing fine aggregate.
3. The relationship between the test results are compared and analyzed to determine the feasibility of using shredded rubber tyre wastes as fine aggregates in non-structural concrete. Evaluate the performance of concrete on replacing aggregates percentage by rubber, to solve the issues of depleting natural resource sand and on other hand to maximize the use of industrial waste.
4. Find the maximum percentage of rubber that can replace the natural resource of fine aggregate (FA) in concrete proportion, which makes its application in non-structural purposes practical.

1.6 Scope and limitations

The scope of the study is limited to:

1. The binders used in the study were limited to a blend of ordinary Portland cement (PC) CEM 1/52.5R and fly-ash (FA) 70/30 PC/FA and 50/50 PC/ground granulated blast furnace slag (GGBS). The FA and GGBS mixes prepared without tyre rubber content were used as reference, i.e. control mix, to compare the results of the mixes prepared by replacing fine aggregates with tyre rubber.
2. Two water-to-binder ratios of 0.40 and 0.55. The water/binder ratio of 0.40 was used to investigate the influence of less permeable concrete on the strength and durability properties of concrete. On the other hand, 0.55 water-to-binder ratio was selected to evaluate the influence of highly permeable concrete on the strength and durability properties of concrete.
3. 13.2mm andesite stones for coarse aggregate and a blend of crusher sand and varying percentage of shredded rubber for fine aggregate were used

respectively. The shredded rubber replaced fine aggregates partially in 7.5, 15, 25, 35 and 50%. Andesite stones are commonly available.

4. Strength testing was conducted at 7, 28 and 56 days and durability tests were carried out at 28 and 56 days.

1.7 Thesis Outline

The thesis is divided into five chapters as follows:

Chapter 1 gives a general introduction of the study.

Chapter 2 is a literature review. It provides an overview of the aggregates, effects of aggregate properties on concrete. As the study focusses on replacing aggregates with shredded rubber tyre, the characteristics of recycled tyre rubber and the properties of rubberized concrete are outlined.

Chapter 3 gives the details of experiments carried out in lab and methodology adopted in the study.

Chapter 4 discusses the experimental results of this study.

Chapter 5 present the conclusions based on the findings of this study and provides recommendations for future study.

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CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Aggregates are defined as ‘a granular material such as sand, gravel, crushed stone or iron blast-furnace slag, used with a cementing medium to form hydraulic-cement concrete or mortar’(ASTMC125, 2010). They constitute up to 70 to 80% of the volume of a typical concrete mix (Shetty, 2005b), and affect various properties of both fresh and hardened concrete. That is, mix proportioning of concrete is partly governed by the aggregate properties such as particle shape, grading, surface texture and water demand (Neville, 2008b). Hardened concrete properties such as durability, creep, and shrinkage are also affected by aggregate properties. Aggregates also play the role in reduction of the cost of concrete. Most importantly, aggregates transform the binder phase into a useful engineering material (Legg, 1974). It is therefore important to understand not only the properties of aggregates but also how they affect the properties of concrete.

Natural mineral aggregates (e.g. granites, andesites, etc) have been used to make concrete for many years. However, with the increasing demand for concrete as a construction material, natural aggregates are fast becoming depleted, and carefully selected alternative materials have had to be sought to partially or fully replace the natural aggregates. Some of these materials include mine tailings, recycled concrete and shredded rubber tyres. This study focuses on the use of the latter, specifically on shredded rubber tyres. The challenge of tyre disposal is a global issue, affecting both developed and developing countries like South Africa. Annually, Europe and the United States generate 245 million and 270 million waste tyres respectively (R Siddique and Naik, 2004) (Bjegovic and D.Lakusic, 2013). According to (Zvimba *et al.*, 2013), South Africa has over 60 million stockpiled waste tyres and this figure increases by about 10 million every year (Mushunje and Otieno, 2017).

These alternative materials used to replace natural aggregates in concrete should meet the requirements both in terms of material properties and concrete properties.

This chapter focuses on aggregates and alternatives for aggregates, their characteristics and influence on fresh and hardened properties of concrete.

2.2 Aggregates for concrete

Before discussing the properties of aggregate themselves and their performance in concrete, there is a need to discuss the origin and sources of aggregates, and how they become useful engineering materials.

Natural aggregates are derived from solid rock, that is naturally occurring geological sources, which is crushed or has been broken down by natural processes. Their properties are primarily governed by the chemical, mechanical and physical properties of the parent rock (Alexander and Mindess, 2005c). Other sources of aggregates are from recycled waste products such as slag, crushed building rubble, synthetic compounds such as polystyrene, heat treated minerals such as vermiculite and perlite (Grieve, 2009).

2.2.1 Natural aggregates

Rocks, defined as ‘naturally occurring inorganic substances of more or less definite chemical composition and usually of a specific crystalline structure’ according to ASTM C294c. They comprise of various minerals and are commonly classified as igneous, sedimentary or metamorphic, as per the method of their formation or origin. In terms of physical characteristics, such as particle size and bulk density, they are classified as coarse and fine aggregates, and normal weight, light weight and heavy weight aggregates respectively. It is also classified with respect to types and relative proportions of the minerals present, that is, petrologically.

Igneous rocks: they are formed through the cooling and solidification of magma and characterized by rate and condition of cooling. With respect to silica percentage, magma may be supersaturated, saturated, and undersaturated, i.e. if a silica mineral precipitates from magma, they are supersaturated. Free quartz is present in supersaturated rocks, while no free quartz are present in saturated and undersaturated rocks. Nepheline ($NaAlSiO_4$) and/or leucite ($KAlSi_2O_6$) are the normative minerals that occur in undersaturated rocks. Other general undersaturated minerals are Forsteritic Olivine ($MgSiO_4$, Sodalite ($3NaAlSiO_4.NaCl$), Nosean ($6NaAlSiO_4.Na_2SO_4$), Hauyne ($6NaAlSiO_4.(Na_2Ca)SO_4$, Perovskite ($CaTiO_3$), Melanite ($CaFe^{3+}SiO_3O_{12}$), Melilite ($(Ca,Na)_2(Mg,Fe^{2+},AlSi)_3O_7$). Saturated rocks contain olivine, or hypersthene + olivine minerals (Nelson, 2018).

Extruded volcanic (fine grained), shallow intruded (medium-grained) and plutonic (coarse-grained) rocks are the igneous rock types, formed by rapid cooling, slower cooling under great pressure and extremely slow cooling under great pressure. Table 2.1 lists the igneous rock types for concrete aggregates (ASTMC294, 2017) (Alexander and Mindess, 2005c)

Table 2. 1: Igneous rocks as origins for concrete aggregates(ASTMC294, 2017) (Alexander and Mindess, 2005c)

Igneous Rocks	Rock types for concrete aggregates
Extruded volcanic rocks	Andesite, Rhyolite, Felsite, Trachyte, Basalt, Glassy Volcanics, Obsidian, Perlite, Pumice, Scoria
Shallow intruded	Microgranite, Granophyre, Diabase, Dolerite
Plutonic rocks	Granite, Pegmatite, Syenite, Diorite, Norite, Gabbro, Peridotite

Sedimentary rocks: they are derived from physical or chemical degradation of other rocks, resulting in fragments which accumulate as deposits, usually under water. Strength of cementing agent determines the strength of the rock mass. And they are less liable to degradation. Conglomerate, Sandstone (Orthoquartzite, Greywacke, Arkose), Chert, Claystone, Siltstone, Argillite, and Shale Carbonates (Limestone and Dolomite, Marl and Chalk), Tillite, Various types of gravels and sands derived from parent rocks; are the sedimentary rock types for concrete aggregates (ASTMC294, 2017) (Alexander and Mindess, 2005c).

The individual particles making up the body of the sedimentary rock maybe large as in the case of conglomerate or very small as in mudstone, shale and limestone. As they are formed from the disintegration of older rocks, their chemical composition is generally simpler and less liable to break down with respect to igneous and metamorphic rocks. The durability of the rock itself therefore depends to a great extent on the nature of the cementing materials (Grieve, 2009).

Metamorphic rocks: they are derived from pre-existing igneous, sedimentary or from other metamorphic rocks, which are altered texturally, structurally or mineralogically in-situ by the actions of high temperature and/or pressure, often together with percolating waters or vapors (ASTMC294, 2017) (Alexander and Mindess, 2005c). Metamorphism improves strength, hardness, durability, toughness and similar desirable properties of sedimentary rocks, on other hand metamorphosed igneous rocks show very little improvement on these properties (Grieve, 2009).

Gneiss, Hornfels, Marble, Quartzite (Metaquartzite), Schist and slate are the rock types for concrete aggregates (Alexander and Mindess, 2005c).

In terms of physical characteristics, such as particle size, the particles larger than 4.75mm are coarse aggregates and particles equal to or less than 4.75mm are fine aggregates. In terms of density, most natural aggregates have granular bulk density of 1500 – 1700 kg/m^3, referred as normal weight aggregates. Those with bulk densities less than about 1120kg/m^3 and greater than about 2000kg/m^3 are termed as light weight and heavy weight aggregates (Alexander and Mindess, 2005c). Andesites used as coarse aggregates and fine aggregates in the study have a bulk density of 1605kg/m^3 and rubber tyres of 1150kg/m^3.

From petrological view, aggregates are divided into several groups of rocks having common characteristics irrespective of whether they are crushed or naturally reduced in size. The classification of natural aggregates according to rock type as per BS 812: Part 1 :1975, is as Basalt group, Granite group, Limestone group, Schist group, Flint group, Gritstone group, Porphyry group, Gabbro group, Hornfels group and Quartzite group is listed in Table 2.2 (BS 812-1, 1975) (Neville, 2008b). Although, they don't give reliable guide as to the engineering characteristics of the rock types. The standard requires an aggregate to be described in petrological terms, including the type of aggregate, that is, crushed rock, sand, or gravel or artificial; nominal particle size; and other deleterious materials, clay lumps and so on. In South Africa, aggregates are likewise as in USA and Canada, are classified in petrological terms, but more emphasis is given to the major sources with regard to abundance of use. For example, quartzites, sandstones, basic igneous rocks and so on.

Table 2. 2 Classification of natural aggregates according to rock type (BS 812: Part 1:1975)

Basalt group	Andesite, Basalt, Basic porphyrites, Diabase, Dolerites, Epidiorite, Lamprophyre, Quartz-dolerite, Spilite
Granite group	Gneiss, Granite, Granodiorite, Granulite, Pegmatite, Quartz-diorite, Syenite
Limestone group	Dolomite, Limestone, Marble
Schist group	Phyllite, Schist, Slate, All severely sheared rocks
Flint group	Chert, Flint
Gritstone group	Arkose, Greywacke, Grit, Sandstone, Tuff
Porphyry group	Aplite, Dacite, Felsite, Granophyre, Keratophyre, Microgranite, Porphyry, Quartz-porphyrite, Rhyolite, Trachyte
Gabbro group	Basic diorite, Basic gneiss, Gabbro, Hornblende-rock, Norite, Peridotite, Picrite, Serpentinite
Hornfels group	Contact-altered rocks of all kinds except marble
Quartzite group	Granister, Quartzitic sandstones, Re-crystallized quartzite

2.2.2 Recycled / manufactured aggregates and aggregates from waste products

BS 12620 (2013) defines recycled aggregate as aggregates resulting from the processing of inorganic or mineral material previously used in construction. In this standard, coarse recycled aggregates are classified according to the proportions of constituent materials including amounts of concrete/ concrete products/ mortar/ concrete masonry units; unbound aggregate/ natural stone/ hydraulically bound aggregate; clay masonry units/ calcium silicate masonry units/ aerated non-floating concrete; bituminous materials; glass; floating material in volume; and other materials including gypsum plaster; clay and soil, metals; and non-floating wood, plastic and rubber (BS EN12620, 2013) (Chindaprasirt and Cao, 2015).

Studies have been made to determine the suitability of recycled materials for use as aggregates in concrete (Buck, 1976) (Halverson, 1981). These studies are significant as they contribute both economically and environmentally. Recycled aggregates such as building rubbles contain deleterious amounts of brick, glass and gypsum; reactive or poor-quality aggregates and high chloride contents etc., therefore, it requires high caution on using recycled aggregates. The aggregates made from industrial or municipal wastes, recycled or marginal materials may possess a number of undesirable properties both physically and chemically (ACI221R-96, 2001) (ASTMC33/C33M-18, 2018).

In Australia, recycled aggregates are classified into following groups: recycled concrete aggregates, recycled concrete and masonry, reclaimed aggregates, reclaimed asphalt pavement, reclaimed asphalt aggregate, glass cullet sand, scrap tyre chips and crumb rubber aggregate and used foundry sand or spent foundry sand (Chindaprasirt and Cao, 2015). And in most parts of South Africa, high quality natural aggregates were available at a very low cost. This resulted in limited use of recycled aggregates. However, with the awareness of environmental damage caused by wastage, recycled aggregates is becoming an option (Kearsley and Mostert, 2012).

2.3 Properties of aggregates

The important characteristics of aggregates for concrete include resistance to abrasion and degradation, resistance to freezing and thawing, resistance to disintegration by sulfates, particle shape and surface texture, grading, fine aggregate degradation, uncompacted void content of fine aggregate, bulk density, relative density (specific gravity), absorption and surface moisture, compressive and flexural strength and aggregate constituents (Brown, 1998).

2.3.1 Physical properties of aggregates

Physical properties of aggregates are those that refer to the physical structure of the particles that make up the aggregate. The role of various aggregate properties in concrete characterization includes:

2.3.1.1 Aggregate particles porosity and absorption

*T*he volume of pores to the total volume of water is defined as porosity. They influence the particle density and absorption of the aggregate (Alexander and Mindess, 2005c). The porosity of aggregate, its permeability and absorption influence the bonding between aggregate and cement paste, the resistance of concrete to freezing and thawing and resistance to abrasion. The extent to which pores or voids are filled with liquid determines the absorption of aggregate. Absorption of aggregates adversely affects the workability and durability of concrete (Grieve, 2009). As per SABS 1083:1994, for water absorption, 1% is deemed a safe value.

2.3.1.2 Particle density and specific gravity

Density of the aggregates is required in mixture proportioning and to establish weight-volume relationships. The bulk density or unit weight of aggregate gives information about the shape and grading of the aggregate. For a given specific gravity, the angular aggregates show a lower bulk density. A sample giving minimum voids or maximum bulk density is taken as the right sample of aggregate for making economical mix. ***Specific gravity*** of aggregates is made use of in calculations of concrete mixes. With specific gravity of constituents known, its weight can be converted into solid volume and hence a theoretical yield of concrete per unit volume can be calculated (Grieve, 2009; Shetty, 2008).

Particle density influences concrete density in both the plastic and hardened states. They influence segregation, concrete strength, stiffness and thermal properties.

Bulk density and void content determine the paste or matrix volume required, thus influencing the relative proportions of concrete ingredients (Alexander and Mindess, 2005). Specific gravity is determined as per SANS5845:2006.

Andesites coarse aggregate and crushed andesite sand are used in the research. According to Chung et al., (Chung, Elrahman and Stephan, 2017), the characteristics of lightweight aggregates is very important for development of lightweight concrete. According to EU 206, light weight concrete is a material with density lower than 2000kg/m^3, and it contains natural or synthetic light weight aggregates or foam agent, to reduce the weight of material, this material can be used for both structural and non-structural purposes, i.e., insulation and concrete block (European Union, 1998).

2.3.1.3 Grading, fineness modulus and impurities (dust and clay content)

Grading is the particle size distribution of an aggregate as determined by sieve analysis. The common sizes of aggregates range from 75mm to 75 microns. The aggregate fraction from 75mm to 4.75mm are termed as coarse aggregate or stone. The aggregate fraction from 4.75mm to 75micron is termed as fine aggregates or sand. As per SABS sieve sizes, 75mm, 37.5mm, 19mm, 9.5mm, 4.75mm, 2.36mm, 1.18mm, 600micron, 300micron, 150micron and 75micron are the usual ranges (Grieve, 2009). Grading of sand and stone has a major influence on the workability, cohesiveness and bleeding of fresh concrete. An ideal (optimum) aggregate grading is not possible, however, best results on the properties of the fresh mix are shown by proportions of sand particles passing 75µm, 150µm and 300µm sieves. For example, if mix is deficient in fractions minus 150µm and 300µm, it tends to produce concrete that is harsh and inclined to segregate and bleed. On other hand, if in excess, it causes stickiness of the fresh concrete(Grieve, 2009). Thus, grading of stone has lesser influence on the properties of the fresh concrete than that of sand. Aggregate gradation consists of continuous graded, poorly graded and gap graded. An 'optimized grading', of coarse and fine aggregates results in an economical concrete with good workability. Depending on the characteristics of each aggregate, the placement method and the finish required, the proportions between coarse and fine aggregates changes. Gap-graded concrete or the use of single sized stone combines well with coarse crusher sands, provided the slump is kept reasonably low. But, for mixes of high workability having slumps of 100mm and over, the use of graded stone is preferable (Grieve, 2009). Gap-graded stone shows less likelihood of particle interference, on other hand continuously graded stone shows less segregation of wetter mixes. The need for gap graded aggregate is greater if the particle shape of the stone is poor (Shetty, 2005b). A well graded aggregate with minimum voids requires minimum paste to fill up the voids in the aggregate, which implies less quantity of cement and water, thereby, an increased economy, higher strength, lower shrinkage and greater durability (Grieve, 2009).

Optimization of gradation of aggregates, in order to reduce the paste volume required to fill the cavities between aggregate particles, is based on the availability of aggregates and requirements
of the concrete formed. It helps in determining the void space to be filled as well as the surface area which needs to be coated with paste. As per Alexander and Mindess, it is the combined effects of particle shape, surface texture and grading that largely govern the plastic properties of concrete. Sieve analysis is conducted as per SANS 1083 (Grieve, 2009) (Alexander and Mindess, 2005c) (SANS1083, 2017).

Fineness modulus, (FM) of fine aggregate according to ASTM C 125 is calculated by adding the cumulative percentages by mass retained on each of a specified series of sieves and dividing the sum by 100. The specified sieves for determining FM are: 150µm, 300µm, 600µm, 1.18mm, 2.36mm, 4.75mm, 9.5mm, 19.0mm, 37.5mm, 75mm and 150mm. It is the parameter introduced by Abrams for proportioning of concrete (Shetty, 2005b). It was found that any sieve analysis curve of aggregate that gives same fineness modulus, will require same quantity of water to produce a mix of the same plasticity and gives concrete of same strength, as long as it is not too coarse for the quantity of cement used. The parameter is an index of fineness of an aggregate. The material passing 75µm sieve is defined as dust and this content is limited to control drying shrinkage in the concrete (Grieve, 2009).

The test method followed for the determination of fineness modulus of aggregate is as per SANS 201 (SANS 201, 2008).

2.3.1.4 Organic material in aggregates

Presence of organic material in natural sand causes retardation of setting, discoloration of concrete or mortar. If the presence of unexpectedly high organic content does not show detrimental effect on compressive strength, the sand can be accepted (Grieve, 2009).

2.3.1.5 Particle shape and surface texture

Shape and texture of aggregates tend to influence the performance in fresh concrete as compared to that in hardened concrete (Brown, 1998). Particle shape of aggregate ranges from spherical to chunky or cubical to angular, flaky or flat. Sphericity measures whether the particle is compact in shape and roundness the relative sharpness of the particle edges and corners. Closer the particle resembles a sphere or cube, lower is the water requirements as compared to flakier or more angular or elongated particles. They result in better workability as well (ACI, 2002).

The relative degree to which particle surfaces are polished or dull, smooth or rough is measured based on ***surface texture*** (Shetty, 2008). As the surface smoothness increases, contact area decreases, resulting in a highly polished particle having lesser bonding area with the matrix than a rough particle of the same volume. Smoother particles require less mixing water and thus lesser cement paste than rough particles. Surface texture of aggregate beneficially influence the flexural strength, with the rough textured aggregates developing higher bond strength in tension than smooth textured aggregate (ACI, 2002) (Grieve, 2009).

Particle shape is determined as per SANS 1083:1994 (SANS1083, 2017). And they influence particle packing and internal aggregate interlock, on other hand particle surface texture affects surface frictional properties in a mix and therefore mix

2.3.1.6 Water soluble chemicals and other deleterious chemicals in aggregates

Aggregates contain harmful substances for concrete. Most of them include soluble substances that leached out of aggregates, which reacts with cement destroying its properties, and other substances that react with the alkali constituents of cement causing corrosion of the reinforcing steel in concrete (Grieve, 2009).

2.3.1.7 Other properties

Thermal properties of aggregates such as specific heat, conductivity, diffusivity influence concrete properties strongly. Concrete can crack under large temperature differentials. And they are important for massive sections (Alexander and Mindess, 2005c).

Aggregates impart structurally manageable thermal movement, compatible with steel and influences on reducing shrinkage and restraining creep of paste in concrete, by providing durability (Alexander and Mindess, 2005c). On the other hand, very different thermal properties of coarse and fine aggregates can cause failure of concrete (Grieve, 2009). Temperature movements, i.e., thermal contraction/expansion of aggregates is significant for concrete properties, as they result in strain incompatibility between aggregate and paste, which leads to cracking.

2.3.2 Mechanical properties of aggregates

The mechanical properties of aggregates such as strength, stiffness, fracture and resistance to abrasion or attrition are significant to engineers, as these properties of aggregate to an extent, affects the performance of concrete.

As per American Concrete Institute (ACI221R-96, 2001) (Neville, 2008b), the typical ranges of aggregate properties indicate that aggregate source materials are generally strong in compression and relatively weak in tension, thus identifying them brittle. For example, fine-grained massive rocks such as andesite, felsite, dolerite and dolomite generally gave superior unconfined compressive strength, on other hand, bedded and jointed rocks such as quartzite and silt-stone, and coarse-grained rocks such as granite had values near the lower end of the range.

2.3.2.1 Strength of aggregates

Strength of aggregate is easier to be measured in granular form. Even though this is treated as the 'standard' measure of aggregate strength, it has the drawbacks that they are usually limited to coarse aggregate particles. And the inherent strength of the material is not measured. They are affected by the aggregate shape, flakiness, inherent flaws or cracks, and inherent strength of the material. The strength measured gives an indication of the ease of crushing of the material and its performance during transporting and stockpiling, useful features to aggregate producers (Alexander and Mindess, 2005d). In South Africa and UK, per cent fines aggregate crushing test (FACT) is used.

2.3.2.2 Abrasion, wear resistance and hardness of aggregate

It is the resistance to degradation, a surface property, which is important in cases of surface wear, for example, in trafficked or erosion areas, dam spillways and stilling basins, concrete pavements and wear-resistant floors. The hard minerals in aggregates improve concrete resistance to abrasion and thereby improve surface frictional properties. These properties influence the abrasion resistance of concrete as a composite. With regard to aggregate breakdown during handling and mixing, the resistance to degradation play an important role (Alexander and Mindess, 2005d).

Fine aggregates are tested under wet abrasion conditions. The Micro-Deval attrition test determines aggregate abrasion loss in the presence of water and an abrasive charge, providing information for judging the suitability of a fine aggregate which is subject to weathering and abrasive action in the absence of service information.

2.3.2.3 Fracture properties

Fracture toughness and fracture energy are measurable fracture parameters of aggregates. The brittle materials such as rocks can be characterized by these parameters. The parameter relating to the critical stress field surrounding a crack is termed fracture toughness, on other hand, fracture energy refers to the energy required to cause a crack to grow in a material. Fracture tests can give the elastic modulus, taken as the average of compressive and tensile moduli (Alexander and Mindess, 2005d).

2.3.2.4 Elastic properties

Addition of aggregate to cement paste, increases the elastic modulus of the concrete.

2.3.3 Durability properties of aggregates

Aggregates are one of the source contributing undesirable constituents to the concrete mix. These include soluble salts, reactive minerals, soft and friable particles, organics etc. Thus, hindering the durability of concrete (Alexander and Mindess, 2005d). The various properties of aggregates are:

2.3.3.1 Soundness of aggregates

It is defined as the ability of aggregate to resist excessive changes in volume as a result of changes in physical conditions, such as freezing and thawing, thermal changes at temperatures above 0°C, and alternate wetting and drying. This relates to the physical durability or physical competence of the material; an important property affecting the performance of concrete (Alexander and Mindess, 2005d).

Surface scaling, pop-outs, and cracking of concrete are the result of unsound aggregates. Soundness is assessed indirectly from aggregate granular strength tests and sulphate soundness. SANS 5839 indicates that sulphate soundness tests on South African aggregates only gives a rough indication of the durability of an aggregate when it is subjected to aggressive environments. Any disruptions in the tests cannot be directly correlated with aggregates performance when subjected to these conditions. Therefore, when aggregate is to be evaluated based on this test, it should be done by comparison with an aggregate of similar mineralogical composition and geological history and which has a proven service record (Grieve, 2009).

2.3.3.2 Freeze-thaw soundness

Depending on the pore-structure of aggregates, their absorption and permeability properties, they are susceptible to freeze-thaw damage. Freezing of water in the pores of aggregates, under severe frost action, is the primary freeze-thaw damage mechanism.

The absorption, permeability, porosity and pore structure properties of aggregate, determines its frost resistance. Aggregates possess a critical particle size for any freezing rates, above which the particle will fail if frozen when critically saturated. This critical size is dependent upon the rate of freezing and the porosity, permeability, and tensile strength of the particle (Kosmatka, Kerkhoff and Panarese, 2002).

2.3.3.3 Wetting and drying properties

Wetting and drying results in weathering, which affects the durability of aggregates. Clay lumps and other friable particles tend to develop severe strain with alternate wetting and drying. And with certain types of rock this results in a permanent increase in volume of the concrete and eventual breakdown. The moisture swelling characteristics of certain aggregates like shales and clay balls, results in pop-outs (Kosmatka, Kerkhoff and Panarese, 2002).

2.4 Aggregates in concrete

The role of aggregate in concrete is that of a structural filler and of great importance. For example, they vary from one concrete mix to another. The characteristics and performance of concrete mixes are altered and influenced by changes in grading, maximum size, unit weight and moisture content of aggregates. The physical and chemical properties of aggregates influence the fresh and hardened properties of concrete. Fresh properties include workability and water demand, which is affected by particle shape, size, gradation, surface texture and absorption. Hardened properties include strength and elastic modulus, durability, shrinkage, creep, expansion and thermal movements, which is affected by strength, particle density, chemical composition, dimensional stability, presence of contaminants and fineness modulus. Larger the size of coarse aggregates, lesser the cement requirement, thus playing a role in reduction of cost.

2.4.1 Influence of aggregates on fresh properties of concrete

The consistency of the concrete mix should be such that concrete can be transported, placed, compacted and finished, sufficiently easily and without segregation. It is the fresh properties of concrete that helps obtain this objective (Neville, 2008a). Handling of concrete has practical implications for construction. The plastic or fresh state of concrete exists in its initial one hour (Alexander, 2014). Workability, consistence and its measurement, cohesiveness, bleeding, plastic cracking, slump loss, density and air content are the properties of fresh state of concrete. Full compaction and minimum segregation and bleeding of the fresh concrete are essential to achieve the required strength and durability in their hardened state (Kellerman and Crosswell, 2009).

2.4.1.1 Workability

With minimum loss of homogeneity, the effort required to manipulate a freshly mixed quantity of concrete is determined by workability of the mix. ASTM C 125-93 describes workability as "property determining the effort required to manipulate a freshly mixed quantity of concrete with minimum loss of homogeneity." Slump measure, grading and shape of aggregate influences workability of concrete. Shape of aggregates is influenced by the characteristics of the parent rock, type of crusher used and the reduction ratio (ratio of the size of material fed into crusher to the size of the finished product). Rough angular particles have lesser workability as compared to smooth rounded particles (Alexander and Mindess, 2005d). As rounded aggregates require lesser cement, cost wise it is preferred. Flaky aggregates make poorer concrete as compared to angular aggregates (Shetty, 2005a).

2.4.1.2 Consistence

It is described as the mobility or ease of flow of the mix, related to its wetness or dryness. Concretes of same consistence may vary in workability. It is sometimes used as an alternative particularly for mortars and is usually measured by the slump test (Alexander and Mindess, 2005b). SANS 5861 (2006), part1 outlines the standards for mixing fresh concrete and part 2 outlines the sampling of the mixed fresh concrete in the laboratory (SANS5861-1, 2006) (SANS5861-2, 2006). Consistence tests are carried out to control the water content and thereby the water: cement ratio, by specifying limits for test results. According to the results obtained, water content added are adjusted (Kellerman and Crosswell, 2009).

2.4.1.3 Segregation

It is the separation of coarse particles from the mortar phase of the concrete, and the collection of these particles at the perimeter or base of a concrete placement (Alexander and Mindess, 2005b). The other form of segregation occurs in the wet mix, which is the separation of grout, i.e. cement plus water, from the mix. Differences in the particle sizes and in the specific gravity of the mix constituents are the primary causes of segregation. The extent of segregation is controlled by the choice of suitable grading and by care in handling (Neville, 2008a).

2.4.1.4 Bleeding

It is a form of segregation, in which some of the water in the mix tends to rise to the surface of freshly placed concrete. This is the result of the inability of solid constituents of the mix to hold all of the mixing water when they settle downwards, water having the lowest specific gravity of all the mix constituents (Neville, 2008a). Bleeding of concrete continues until the cement paste has stiffened sufficiently, putting an end to the process of sedimentation. Formation of voids under aggregate particles and reinforcing steel, as water migrates upwards, sand steaking, surface laitance and settlement cracking are the defects attributed to excessive bleeding (Kellerman and Crosswell, 2009). By modifying the properties of cement or using finer cement, and adequate proportion of very fine aggregate particles, significantly reduces bleeding. Also, reduction of water content and usage of air-entraining admixture reduces bleeding (Kellerman and Crosswell, 2009) (Neville, 2008a).

2.4.2 Influence of aggregates on hardened properties of concrete

Concrete structures should be sufficiently strong and stable to resist applied loads. It must be stiff enough to provide load resistance without undue deformation. Surface texture, stiffness, shape, strength and toughness, and grading of aggregates affect the strength of concrete. It is the physical and mechanical properties of aggregates which is the real matter of concern, and not the aggregate type. This is because within the same aggregate type, physical and mechanical properties can vary extremely (Alexander and Mindess, 2005a).

2.4.2.1 Strength

Strength of concrete is dependent on the quality of cement paste and on the bond between the cement paste and the aggregate. The mechanical properties of the rock or aggregate also influence the strength of concrete. Therefore, even though strong aggregates cannot make strong concrete, for making strong concrete strong aggregates are an essential requirement (Grieve, 2009).

The particle surface texture strongly influences the density of the interfacial transition zone (ITZ), and by improving the density of ITZ, the aggregate strength can have a significant effect on concrete strength (Grieve, 2009) (Shetty, 2005b). Aggregates contribute directly and indirectly to concrete strength depending on their shape. The crushing value tests determines the strength of aggregate. Aggregate shape, flakiness, inherent flaws or cracks, and inherent strength of the material is affected (Alexander and Mindess, 2005d).

2.4.2.2 Dimensional properties

Drying shrinkage, wetting expansion, thermal volume change, elastic modulus and creep are some of the aggregate properties influencing the dimensional stability of the hardened concrete. Shrinkage is tested as per SANS 10100-1:2000 and BS 8110. Higher the shrinkage potential of aggregates, higher is the risk of deterioration of reinforced concrete. If shrinkage is the result of moisture loss, on subjecting the same to wet conditions causes microcracking of concrete, i.e. ***wetting expansion***. The stress to strain ratio, ***elastic modulus*** (Young's modulus) increases on the addition of aggregate to cement paste. According to Alexander and Mindess, aggregate modulus strongly influences the concrete modulus, on other hand lesser on concrete strength. ***Creep***, defined as the increase in strain (deformation) under a constant stress, is mainly the result of hardened cement paste and has very little to do with aggregates for normal concrete. Therefore, addition of aggregates reduces creep. The volume of concrete expanding and contracting with varying temperature is the result of mix proportions and constituents of concrete. Classified as the ***thermal movements,*** quantified by linear coefficient of thermal expansion (Grieve, 2009).

2.4.2.3 Durability

In hardened concrete, durability is affected due to sulphate soundness, freeze-thaw resistance and alkali-aggregate reactivity (AAR). The physical effects of excessive volume change of aggregates under weathering actions because of sulphate soundness, causes pop-outs resulting in severe cracking on concrete. AAR causes expansion and cracking in presence of moisture affecting the long-term performance of structures. According to Alexander and Mindess, these aggregate properties make them susceptible to deterioration, subsequently reduced durability.

2.5 Properties of concrete with recycled aggregates

The need to incorporate recycled materials in building products is becoming more important than ever, so as to avoid landfill and decrease the depletion of raw materials. The production of concrete with recycled materials in place of aggregates is the most desirable form of achieving it. The properties of the waste materials, and the influence of different percentages of these materials on the fresh and hardened properties of concrete, are the important elements to understand when considering recycled aggregates in concrete.

In U.S, 4.5billion tonnes of post-consumer wastes and industrial by-products generated annually are readily recyclable. Initially it was recycling old, demolished concrete as aggregate for new concrete. Since 1990s, other by-products have been successfully used in concrete (Naik, 2002). In South Africa, the National Waste Management Act (NWMA) of 1998, strategizes on diverting 25% of recyclables from landfill sites for re-use, recycling or recovery (StatsSA, 2018).

Many researchers have studied different recyclable material properties and their effect in concrete. A few of them include incorporating glass, plastic and demolished concrete (Batayneh, Marie and Asi, 2007); waste iron from industrial sector (Ismail and AL-Hashmi, 2008); foundry sand and cupola slag from metal casting industries, post-consumer glass, wood ash from pulp mills, saw- mills, and wood product manufacturing industries, sludge from primary clarifiers at pulp and paper mills, and de-inking solids from paper-recycling companies (Naik, 2002) ; scrap tyre-rubber (Panda, Parhi and Jena, 2012) (Rawahi and Waris, 2017) etc.

This paper concentrates on using scrap tyre-rubber in concrete. Shredded tyre-rubber is used to replace fine aggregates in concrete. The properties of the rubberized concrete and their application in non-structural concrete works such as footpaths, light industrial floors (unreinforced), unreinforced house foundations, boundary walls and freestanding retaining walls etc.

2.5.1 Characteristics of scrap-tyre rubber

In general, rubber tyres are made of styrene–butadiene polymers and a small amount of natural rubber. Rubber tires contain a large number of additives, reinforcing agents, and fillers, including large quantities of carbon black, hydrocarbon oils, vulcanizing agents(sulfur, zinc oxide, and organic peroxides),accelerators (e.g., thiazole compounds), retarders, and antioxidants (Niaounakis, 2017). According to Rubber Manufacturers Association of America, the typical composition of tyres consist of natural rubber (C_5H_8), styrene (C_8H_8), 1,3-Butadiene (C_4H_6), fabric, carbon black, steel (belt and bead), fillers and water (H_2O) (Boateng, 2016).

Shredding of scrap tyres results in products with different properties. Depending on factors such as type of rubber, method of grinding, particle size and shape, and specific area of particles, the properties and usefulness of ground rubber varies. Cryogenic technology and mechanical grinding at ambient temperature are the two technologies used to granulate the waste rubber. In cryogenic method, pretreated waste tyres are subjected to cooling using liquid nitrogen and grinding thereafter. On the other hand, mechanical grinding consists of mechanical cutting and grinding of waste tyres (Gaweł *et al.*, 2011). Shredded / Chipped rubber (13 – 73 mm), Crumb rubber (0.425 – 4.75 mm) and Ground rubber (0.075 – 0.425 mm) are the three main classes of waste tyre rubber particles (Mushunje, Otieno and Ballim, 2018).

The large differences between Young's moduli of rubber and cement matrix, results in major differences in the mechanical properties between concrete with conventional natural aggregate and with concrete containing rubber. With increasing rubber content, there is a significant loss in compressive and tensile strength as well as stiffness. The rubber particles constitute of weak inclusions, and are also responsible for significant tensile stresses in the cement matrix, which lead to earlier cracking and failure. As a result, high strength loss is to be expected. According to El-Dieb et.al (2001), and Senouci (1993), this loss is as high as 80% (El-Dieb, Abdel-Wahab and Abdel-Hameed, 2001) (Eldin and Senouci, 1993). However, rubber particles have a restraining effect on crack propagation, which leads to a significant increase in strain capacity, ductility, and energy absorption capacity (El-Dieb, Abdel-Wahab and Abdel-Hameed, 2001) (Meyer, 2008).

The main purpose of using rubber as an aggregate in concrete, is to increase the concretes ductility and capacity to absorb energy (Topçu and Unverdi, 2018). A reduction in mechanical properties and increase in durability properties of concrete such as freeze-thaw and abrasion resistance are anticipated by using waste rubbers in concrete.

The use of rubber as an engineering material depends on the proper understanding of its behavior under rapidly changing stress such as vibrations, shocks etc., and the dependence of this behavior on factors such as temperature and rate of deformation.

The dynamic properties of rubber and the methods of measuring them are studied by Research Association of British Rubber Manufacturers (1960), mostly with respect to automobile industry. They help summarize the characteristics of rubber in engineering. As rubber undergoes stress, the strain always lags slightly. The property which makes rubber a good shock or vibration absorber. Rubber hardness can be defined as the resistance to elastic indentation. Poisson's ratio of rubber, nearly 0.5 makes the material virtually incompressible (Payne and Scott, 1960).

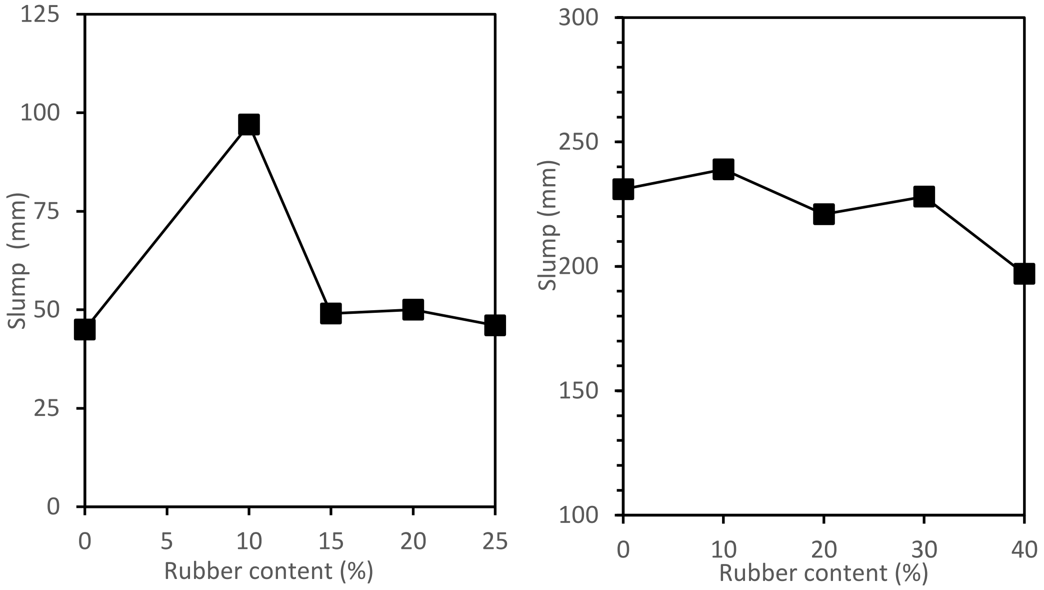
2.5.2 Rubberized concrete properties

Introducing rubber particles to replace natural aggregates, affects both fresh and hardened properties of concrete. There are various research works investigating the properties of rubberized concrete.

Slump: Al-Fadhli and Alhumoud (2017) in their study stated that fresh rubberized concrete exhibits lower unit weight. Further, they report acceptable workability when used to replace mineral aggregates up to 25%. Figure 2.1(i) shows the slump trend with different rubber replacement percentage. Large reductions in the strength and tangential modulus of elasticity with a demonstration of significant decrease in the brittle behavior of concrete with increased rubber content were also concluded from their study (Al-Fadhli and Alhumoud, 2017).

According to the investigations of Khatib and Bayomy (1999) on the workability of rubberized concrete, there is a decrease in slump with increase in rubber content as a percentage of total aggregate volume. They further noted that at rubber contents of 40%, slump was almost zero and concrete was not workable manually. It was also observed that mixtures made with fine crumb rubber were more workable than those with coarse tire chips or a combination of tire chips and crumb rubber (Khatib and Bayomy, 1999).

The studies on workability of controlled low strength rubber concrete (CLSRC) mixtures at different contents of rubber by Wang, Chen and Wu (2013), where natural sand was partially replaced with rubber (size 4.75 mm) at levels of 0%, 10%, 20%, 30% and 40%, by volume is plotted in figure 2.1(ii). The results showed an increase in the slump value with the inclusion of 10% rubber sand, whilst a slump value reduction was noted with the inclusion of 20%, 30% and 40% rubber, respectively (Wang, Chen and Wu, 2013).



(i) *(ii)*

Figure 2. 1: Variation of slump with rubber replacement % (i) (Al-Fadhli and Alhumoud, 2017) (ii) % (Wang, Chen and Wu, 2013)

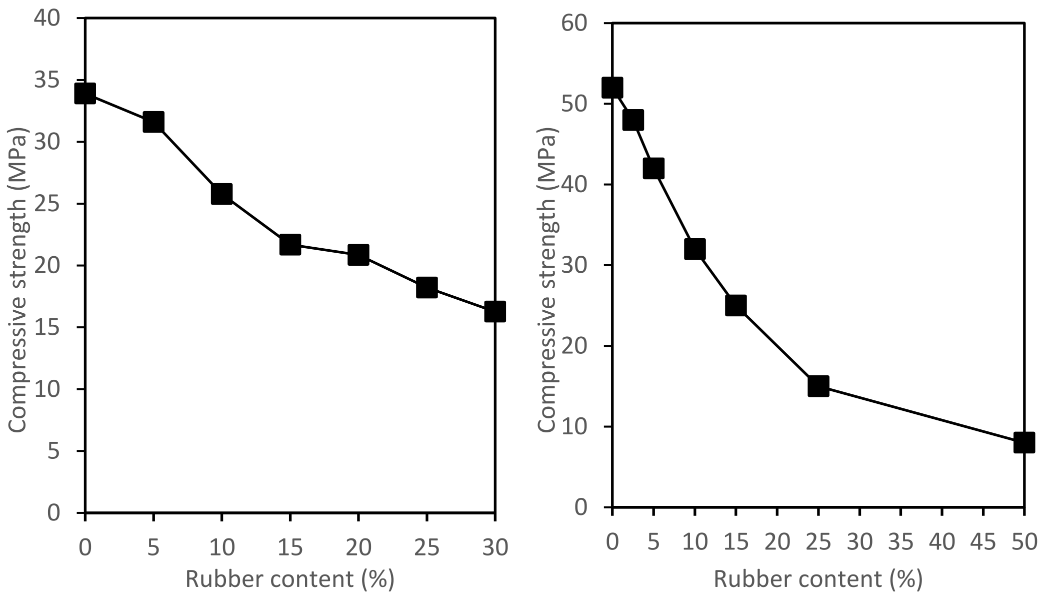
(Batayneh, Marie and Asi, 2007, 2008) have reported reduced workability on the inclusion of rubber tyre particles. On the contrary, few other studies (Rashad, 2016), have reported positive effect of rubber particles on workability. However, a few other studies believed that some rubber sand contents reduced workability, whilst other contents increased it (Khaloo, Dehestani and Rahmatabadi, 2008; Sallam *et al.*, 2008; Pelisse *et al.*, 2012).

Air content: Rubber particles has a tendency to entrap air in their rough surfaces. This would be the result of the non-polar nature of rubber. When added to concrete mixture, it may attract air as it has the tendency to repel water. Therefore, increasing rubber content results in higher air contents in rub-crete mixtures, thereby decreasing the unit weight of the mixtures (Rafat Siddique and Naik, 2004). Fedroff et.al (1996), Khatib and Bayomy (1999) have observed higher air contents in rubberized concrete than in control mixture (Fedroff et al., 1996) (Khatib and Bayomy, 1999).

.***Unit weight***: Rubber particles have a low specific gravity. With increasing percentage of rubber content, the unit weight of mixtures with rubber decreases. Also, increasing rubber content increases air content, which in turn decreases the unit weight (Rafat Siddique and Naik, 2004). According to Khatib and Bayomy (1999), when rubber content is lower than 10-20% of total volume of aggregate, the decrease in unit weight of rubcrete is negligible (Khatib and Bayomy, 1999).

Compressive and tensile strength: Results of various studies indicate that the size, proportions and surface texture of rubber particles, substantially affect the compressive strength of rubberized concrete. Several authors (Jamdar and Ansari, 2018), (Eldin and Senouci, 1993),(Guneyisi, Gesoglu and Ozturan, 2004), (Meddah, Beddar and Bali, 2014),(Rashad, 2016) have reported the compressive strength of rubberized concrete. They report a reduction in compressive strength. Figures 2.2 (i) and (ii) shows the effect of rubber content on compressive strength of concrete from the studies carried out by Meddah et.al (2014) and Guneyisi et.al (2004).

According to Eldin and Senouci (1993), the concrete mixtures with tire chips and crumb rubber aggregates exhibited lower compressive and splitting tensile strengths than regular portland cement concrete. As coarse aggregate was fully replaced by coarse crumb rubber chips, there was approximately 85% reduction in compressive strength and 50% reduction in splitting tensile strength. On other hand, when fine aggregate was fully replaced by fine crumb rubber, a reduction of about 65% in compressive strength and up to 50% in splitting tensile strength was observed. However, both of these mixtures demonstrated a ductile failure and had the ability to absorb a large amount of energy under compressive and tensile loads (Eldin and Senouci, 1993) (Rafat Siddique and Naik, 2004). Similar observations were reported by Khatib and Bayomy, and (Topçu, 1995), when coarse rubber chips and fine aggregate were used. However, opposite trend were indicated in the results reported by Ali et.al and Fattuhi and Clark (Ali, Amos and Roberts, 1993) (Fattuhi and Clark, 1996).



(i) *(ii)*

Figure 2. 2: Effect of rubber content on compressive strength of concrete (i) (Meddah, Beddar and Bali, 2014) (ii) (Guneyisi, Gesoglu and Ozturan, 2004)

If rubber particles have rougher surface or are given pretreatment i.e., varying from washing rubber particles with water to acid etching, plasma pretreatment and various coupling agents, then better and improved bonding may develop with the surrounding matrix (Naik and Singh, 1991).

Shrinkage: Preliminary studies by Raghvan et.al (1998) on two different shaped rubber particles as constituents of mortar, reported reduced plastic shrinkage cracking compared to control mortar. Also, the crack width for control mix was averaged to be greater than specimens with rubber particles as shown in figure 2.3 (Raghavan, Huynh and Ferraris, 1998). They also reported that the onset time of cracking was delayed by the addition of rubber content. Mortar mixes without rubber content showed first crack in 2 minutes, while mortar with 5% mass fraction of rubber sheds cracked after 30 minutes and those mortars with 10 and 15% rubber sheds after one hour.

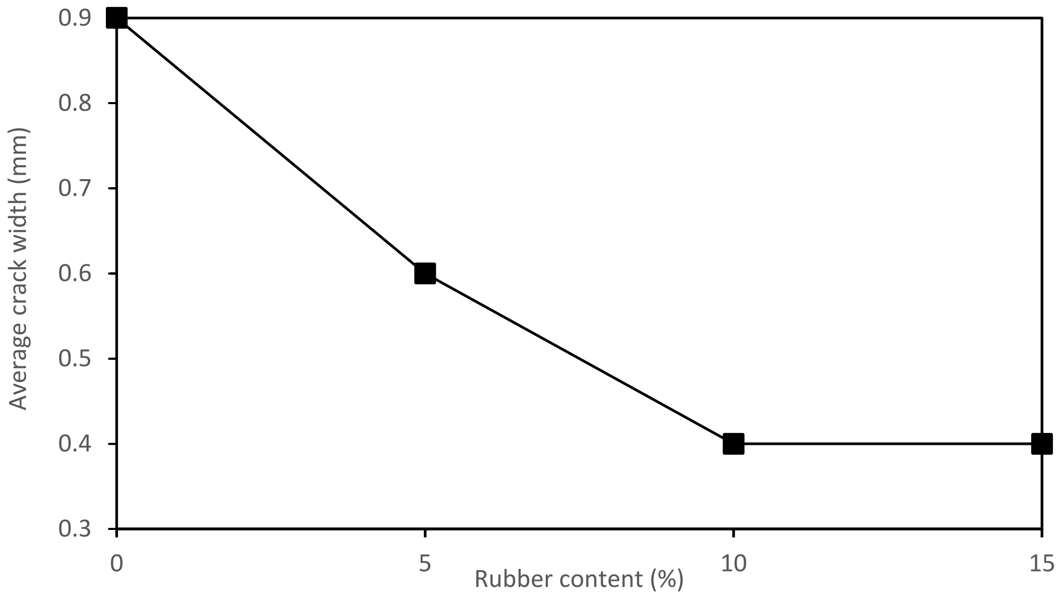


Figure 2. 3: Plastic shrinkage crack width after 3 hours (Raghavan, Huynh and Ferraris, 1998)

Toughness and impact resistance: Toughness or energy absorption capacity of rubberized concrete were found to be higher than the control concrete mixtures according to Tantala.et.al (1996). The rubber replacement of 5% and 10% by volume of coarse aggregate were carried out, and the toughness of 10% buff rubber mixture showed lower toughness compared to 5%. This is because of decrease in compressive strength (Tantala, Lepore and Zandi, 1996). Khaloo et.al (2008) reported a maximum increase in the toughness with the inclusion of 25% rubber, by total aggregate volume. Beyond this level, decrease in toughness, as shown in figure 2.4, was reported due to systematic reduction in strength (Khaloo, Dehestani and Rahmatabadi, 2008). Impact resistance of concrete increased when rubber was incorporated in the concrete mixture, due to the enhanced ability of the material to absorb energy (Topcu and Avcular, 1997). Similar results were reported by (Eldin and Senouci, 1993) and (Topçu, 1995).

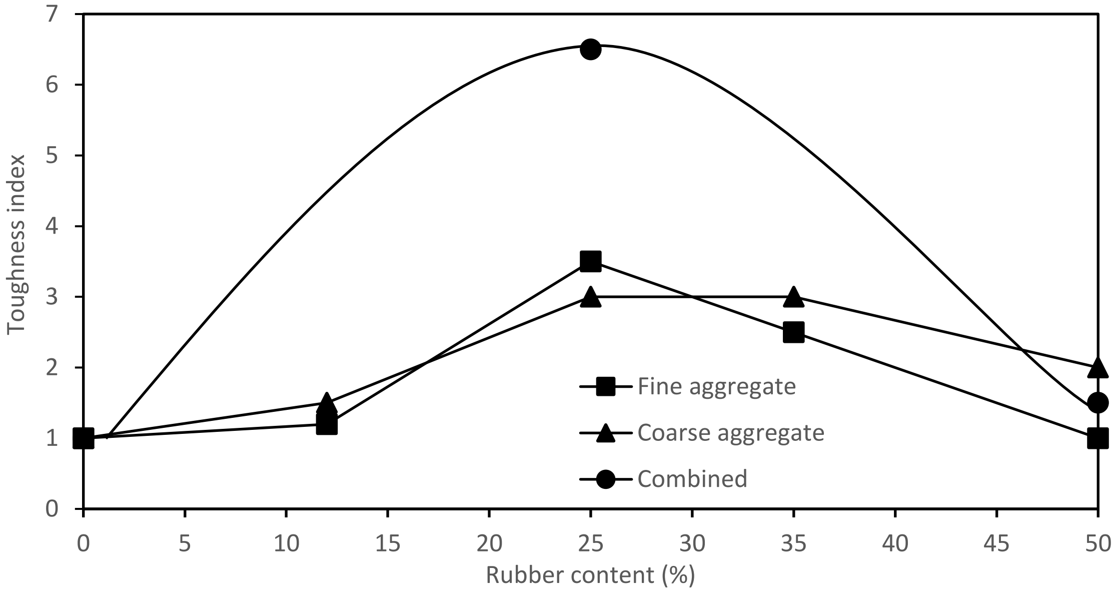


Figure 2. 4: Effect of rubber aggregate content on the toughness index values (Khaloo, Dehestani and Rahmatabadi, 2008)

Freezing and thawing resistance: To study the freezing and thawing durability of rubber concrete, Savas et.al (1997) in his study, incorporated 10-30% ground rubber by weight of cement used for control mixture. It was reported that mixtures with 10% and 15% ground rubber exhibited higher durability factors and those with 20% and 30% did not meet ASTM standard (Savas, Ahmad and Fedroff, 1997). Similarly Al-Akhras et.al (2004) reported higher freeze and thaw resistance for mortars containing rubber ash as natural sand replacement at levels of 5% and 10% by weight, *(Al-Akhras and Smadi, 2004),* as shown in figure 2.5.

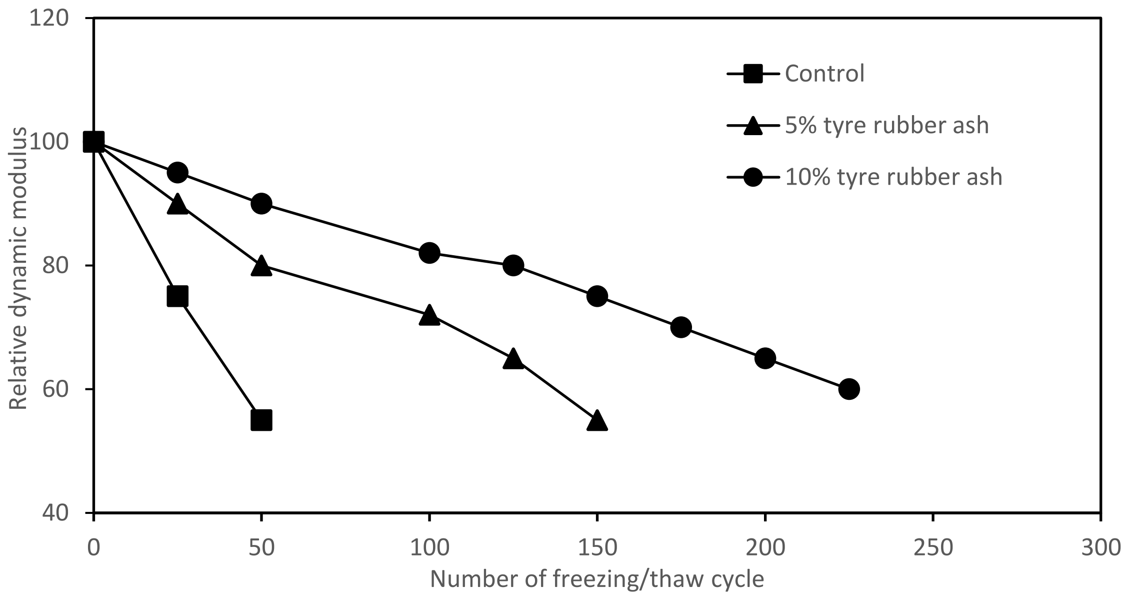


Figure 2. 5: Effect of rubber ash content on the variation of relative dynamic modulus with a number of freezing and thawing cycles (Al-Akhras and Smadi, 2004)

2.5.3 Uses of rubberized concrete

Rubber concrete because of its light unit weight, or being a light density concrete, ranging from 900 -1600kg/m^3, maybe suitable for architectural applications such as nailing concrete, false facades, stone backing and interior construction (Rafat Siddique and Naik, 2004).

(Fattuhi and Clark, 1996) have suggested that rub-crete can be used in areas where vibration damping is needed, such as foundation pad for rotating machinery and in railway stations; for trench filling and pipe bedding, pile heads, and paving slabs and where resistance to impact or blast is required such as in railway buffers, jersey barriers and bunkers.

In highway construction, (Topcu and Avcular, 1997) suggested they can be used as a shock absorber, in sound barriers as a sound absorber, and, also in buildings as an earthquake shock-wave absorber. However, research is required before definite recommendations is made.

Low strength concrete (<25MPa), is suitable for unreinforced foundations (single storey only); mass fill; infill concrete in masonry. Concrete of 20MPa and 25MPa can be used for house slabs, driveways, footings and footpaths. Hence making the application of rubberized concrete in such non-structural concrete construction works feasible.

2.6 Summary of literature review

Aggregates are the major constituents of a concrete mix. It affects the mix proportioning, fresh and hardened properties of concrete, also in cost reduction of concrete. They transform the binder phase into a useful engineering material. Therefore, it is important to focus on aggregates and alternatives used for aggregates in concrete. The origin and sources of aggregates and how they become useful engineering materials are summarized. With respect to the method of formation or origin, they are classified as igneous, sedimentary or metamorphic rocks. In terms of physical characteristics, they are classified as coarse and fine aggregates; and normal, light or heavy aggregates respectively.

Physical properties of aggregates contribute to the characterization of concrete. The porosity of aggregate, its permeability and absorption influence the bonding between aggregate and cement paste. And the absorption of aggregate adversely affects the workability and durability of concrete. Aggregate particle density influences concrete density, segregation, concrete strength, stiffness and thermal properties. An optimized grading of coarse and fine aggregates results in an economical concrete with good workability. Presence of organic material in aggregates, particle shape and surface texture, water soluble chemicals and other deleterious chemicals in aggregate, thermal properties of aggregates are other properties which influence the concrete characteristics.

Mechanical properties of aggregates such as strength, stiffness, fracture and abrasion resistance, elastic properties and aggregate hardness also affect the performance of concrete. Soundness of aggregates, freeze-thaw soundness, wetting and drying affects the durability properties of concrete

Aggregates in concrete plays the role of a structural filler. The physical and chemical characteristics of aggregates influences the fresh and hardened properties of concrete. The fresh and hardened properties of concrete include workability, consistence and its measurements, cohesiveness, bleeding, plastic cracking, slump loss, density, air content, strength, elastic modulus, durability, shrinkage, creep, expansion and thermal movements. To avoid landfill and depletion of raw materials, it is important to incorporate recycled materials in building products. When recycled aggregates are considered in concrete, it is important to understand the properties of waste materials, and the influence of different percentages of these materials on the fresh and hardened properties of concrete. This study uses the waste material scrap tyre-rubber in concrete.

Rubber tyres are made of styrene-butadiene polymers and a small amount of natural rubber. The properties and usefulness of ground rubber varies depending on factors such as type of rubber, method of grinding, particle size and shape, and specific area of particles. Shredded rubber, crumb rubber and ground rubber are the main classes of waste tyre rubber particles.

The huge difference in mechanical properties between conventional concrete containing natural aggregate and rubberized concrete with rubber aggregate, is the result of large differences between Young's moduli of rubber and cement matrix. With increasing rubber content, there is a significant loss in compressive strength, tensile strength and stiffness; and a significant increase in strain capacity, ductility and energy absorption capacity. A reduction in mechanical properties and increase in durability properties of concrete are anticipated by using waste rubbers in concrete. However, there is a gap in the use of rubberized concrete in non-structural construction work by maximum replacement of aggregate.

According to literature review, increase in rubber content, workability of concrete decreased. Few other researchers report positive effect on workability. The tendency of rubber particles to entrap air in their rough surfaces, results in higher air contents in rub-crete concrete, thereby decreasing unit weight of mixture. The compressive strength and splitting tensile strength decreased with increasing rubber content. However, these mixtures possessed ability to absorb a large amount of energy under compressive and tensile loads according to few researchers. Few others reported opposite trends as well. Reduced plastic shrinkage cracking and lesser average crack width of rubberized mortars were reported compared to control mortar. Toughness and impact resistance of concrete were reported to increase when rubber was incorporated in the concrete mixture. Good freezing and thawing durability were exhibited with up to 15% rubber content. These properties of rubberized concrete make its application suitable for architectural applications such as nailing concrete, false facades, stone backing and interior construction.

Non-structural concrete with 20MPa strength can be used for house slabs, driveways, footings and footpaths. Other applications include unreinforced foundations in single storey; mass fill and infill concrete in masonry etc. This study concentrates on the compressive strength and durability properties of the concrete specimens prepared with replacement of fine aggregate by rubber content up to 50%. The durability assessments account for the interaction between concrete as a system and its exposure environment. The indices obtained relates to the degree of aggressiveness that the concrete must withstand and to the service life prediction models.

The next chapter will cover details on the experimental methods used in this study.

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CHAPTER 3

METHODOLOGY

3.1 Introduction

This chapter outlines the details of the laboratory procedures performed during the study. The investigation was carried out by comparing conventional concrete mixes prepared, with concrete prepared by partial replacement of fine aggregates with rubber wastes.

According to previous researchers, it is recommended to replace not more than 30% of fine aggregate by rubber particles. The particle size ranging from super fine (500 to 600 micron) enhances the sound insulation and thermal conductivity properties of resulting concrete rubber (Sukontasukkul and Wiwatpattanapong, 2009). The larger the rubber particle size, the higher the effect on compressive strength and as percentage replacement increased the compressive strength decrease (Rawahi and Waris, 2017). The use of tyre replacements in structural concrete is likely to face loss in strength, hence, its application to non-structural concrete is more suitable. The objective of the study is to analyze the mechanical and physical properties of a non-structural concrete made with shredded rubber tyres and different binder types. The study focuses on understanding the effect of replacing natural fine aggregate at different percentage by shredded rubber tyres. The laboratory experiments carried out in the study is to evaluate the feasibility of using shredded rubber tyres in place of fine aggregate in concrete. An outline of the details of variables introduced, test specimens cast and methods adopted in the study are schematically represented in Figure 3.1.

RESEARCH AIM
To evaluate the performance of concrete by replacing crusher andesite sand aggregates with varying percentages of rubber content.

TEST SET-UP	
Experimental variables	Experimental constants
Binder type	Andesite stones
Rubber percentage	
w/b ratio	

TEST SPECIMENS	
Specimen types:	100mm cubes, 70mm by 30mm discs
w/b ratio:	0.40 and 0.55
Binder types:	50/50 OPC/GGBS, 70/30 OPC/FA
Aggregates:	Coarse – 13.2mm andesite stone Fine – Andesite crusher sand and rubber wastes at 0, 7.5, 15, 25, 35 and 50% replacement

TEST METHODS	
Aggregate	Grading, Fineness modulus, Specific gravity, Consolidated bulk density
Concrete	Workability, Compressive strength, Oxygen permeability index, Water sorptivity index, chloride conductivity index

EXPECTED OUTCOME
Understanding the behavior of concrete to changes in type of aggregate. This leads to solving the issues of depleting natural resource and to provide better waste management of shredded rubber tyres disposed as 'waste'.

Figure 3. 1: Methodology flowchart

3.2 Materials

3.2.1 Binder type

Cement manufacturing is a highly energy consuming process where temperatures of 1400 - 1450^{o}C is involved (Grieve, 2009b), with significant amount of carbon dioxide released into atmosphere. Therefore, by using supplementary cementitious materials (SCMs) such as fly ash (FA) in the production of concretes will help preserve environment and enhance the properties of concrete. The use of blended binders exhibit better performance compared to ordinary Portland cement(OPC) (Motsieloa, 2012). Hence, FA and ground granulated blastfurnace slag (GGBS) are extenders selected for blended binders along with OPC. PC/GGBS: 50/50 and PC/FA: 70/30 are the selected binder type. This is because the two mixes of PC/GGBS 50/50 and PC/FA 70/30 would result in similar strength at maturity. Also, it is the recommended cement types for concrete exposed to non-polluted, polluted, corrosive and highly corrosive atmosphere.(Ballim, Alexander and Beushausen, 2009).

3.2.1.1 Portland cement

Type 1 PC with a strength of 52.5 R, denoted as CEM 1 52.5R contains Portland cement and only 5% minor additional constituents as per the South African standards (Grieve, 2009b). CEM 1 52.5R cement are used were high early strength of concrete or reduction of curing time is required (Crosswell, 2009). Thus, curing is carried out for a maximum of 56 days, before which they are tested for compressive and tensile strength. Portland cement mixes prepared were used against the supplementary cementitious materials (SCM) to compare impact.

The chemical composition of South African Portland cement binder, in terms of oxides (Grieve, 2009b) is presented in Table 3.1.

3.2.1.2 Ground granulated blastfurnace slag

GGBS is a byproduct of the blast furnace process used to produce iron. Replacing cement partially with GGBS affects the fresh and hardened properties of concrete. The concrete made with GGBS sets slower than concrete made with ordinary cement. It improves the workability as well (Grieve, 2009b). GGBS contributes to strength by its gradual hydration reaction which refines the pore structure over time. GGBS with its latent hydraulic binding properties, results in a more gradual development of its cementing properties (Angelucci, 2013). Fresh concrete containing GGBS tends to require less energy for movement, which makes it easier to place and compact, especially when pumping and or using mechanical vibrators. Hence GGBS slows down the development of strength and increases the later age strength, example at 56 days (Suresh and Nagaraju, 2015).

Karri, Rama Rao and P.M Raju (2016) studied strength and durability on GGBS concrete and concluded that compressive strength increased when cement is replaced by GGBS. And at 40% cement replacement, concrete attained maximum strength(Kumar Karri, Rao and Raju, 2016). Oner and Akyuz (2007) studied on optimum level of GGBS content for maximizing strength is about 55-59% of the total binder content (Oner and Akyuz, 2007). The study uses a blend of 50/50 PC/GGBS as per the recommended practices of the concrete industry for various applications.

The chemical composition of South African GGBS binder, in terms of oxides (Grieve, 2009b) is presented in Table 3.1.

3.2.1.3 Fly ash

Fly ash is finely divided residue resulting from the combustion of powdered coal and transported by the flue gases and collected by electrostatic precipitator. The spherical form of the fly ash particles improves the flowability and reduces water demand (Shetty, 2005). American Society for Testing and Materials (ASTM C618) classifies fly ash into class F (CaO <5%) and class C (CaO >10%) depending on the percent of calcium oxide. Class F fly ashes are pozzolanic in nature while class

C are both pozzolanic and hydraulic in nature (Shetty, 2005). The four main constituents of fly ash are silica dioxide (SiO_2), aluminium oxide (Al_2O_3), iron oxide (Fe_2O_3) and calcium oxide (CaO). The range of these constituents are dependent on the selected coal type namely lignite, sub bituminous, bituminous and anthracite.

The chemical composition of South African FA binder, in terms of oxides (Grieve, 2009b) is presented in Table 3.1.

15 to 25 percent by mass of the Portland cement is replaced with fly ash, with even higher percentages used for mass concrete placements. Higher levels of FA have been used in large structures to control temperature rise. High FA content produce concrete with good mechanical and durability properties. The guidelines provided by Cement and Concrete Institute for a mix design is followed in the study (Addis and Goodman, 2009).

Table 3. 1: Chemical composition of South African binders PC, GGBS and FA in terms of oxides (Grieve, 2009b)

Oxide	% by mass		
	PC	GGBS	FA
CaO	63.0-69.0	32.0-37.0	4.0-7.0
SiO_2	19.0-24.0	34.0-40.0	48.0-55.0
Al_2O_3	4.0-7.0	11.0-16.0	28.0-34.0
Fe_2O_3	1.0-6.0	-	2.0-4.0
MgO	0.5-3.6	10.0-13.0	1..0-2.0
K_2O	0.20-0.80	0.8-1.3	1.0-2.0

3.2.2 Aggregates

Natural mineral aggregates (e.g. granites, andesites, etc) have been used to make concrete for many years. However, with the increasing demand for concrete as a construction material, natural aggregates are fast becoming depleted (Indian Roads Congress. *et al.*, 2013), and carefully selected alternative materials have had to be sought to partially or fully replace the natural aggregates. Some of these materials include shredded rubber tyres, glass, recycled concrete and mine tailings (Cresswell, 2007).

3.2.2.1 Mineral aggregates

Andesite is a basic igneous rock, used in South Africa for the manufacture of concrete. It is a kind of extrusive rock with an appearance of dark gray (Yang, 2015). 13.2 mm size andesite stones are used in the study as coarse aggregate for it is commonly available. Andesite crusher sand is considered as the fine aggregate for the study as it is commonly available. The fineness modulus (FM) of the sand is determined in accordance with SANS (SANS 201, 2008). FM indicates the mean particle size in sand. The FM of crusher sand used as fine aggregate is 3.1. Shredded rubber of FM 1.9 is used to replace crusher sand partially in the study. The FM of fine aggregates used, complies to the range 1.2 to 3.5 as per SANS1083 (Grieve, 2009a) (SANS1083, 2017).

The stones comply with SANS 1083, and the extremes of the range of gradings used for the study trial mixes is given in Figure 3.2.

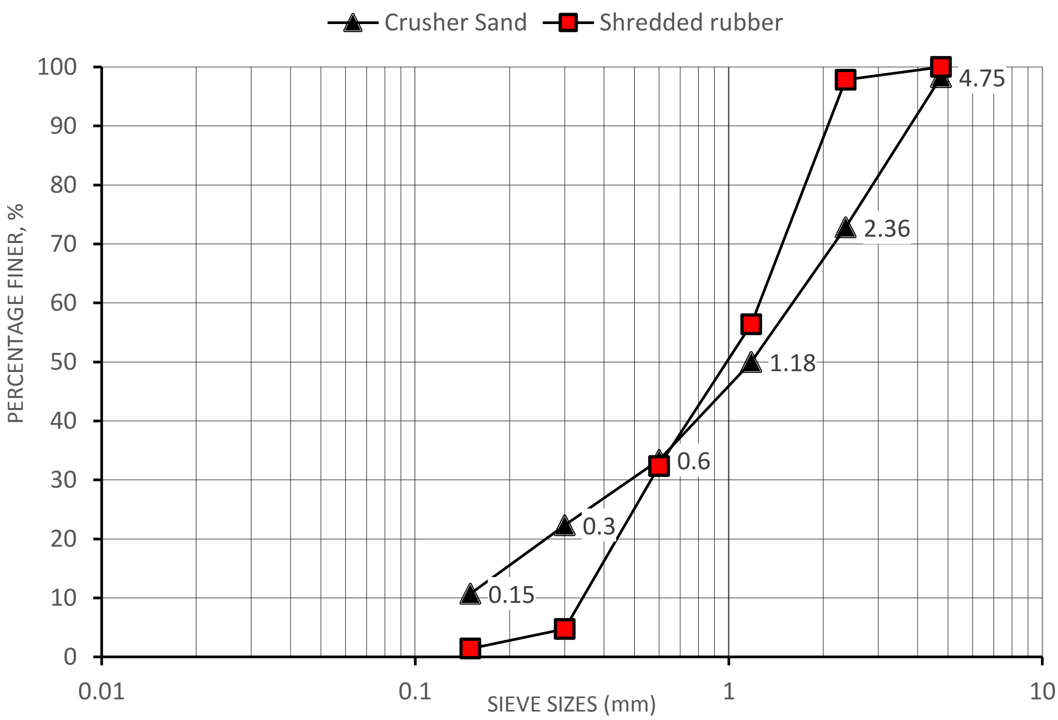

Figure 3. 2: Grading curve of fine aggregate

3.2.2.2 Shredded rubber tyre

In the study, shredded rubber is used to partially replace fine aggregate in 7.5, 15, 25, 35 and 50% in concrete mix. The objective of this replacement level is to find the feasible rate at which natural fine aggregate can be replaced in concrete by shredded rubber, to fulfill the aim of using the concrete in non-structural concrete works.

3.3 Aggregate replacement levels

The percentage replacement of fine aggregates with shredded rubber tyre used in this study is based on volume. The mass of shredded rubber tyres required for a mix was calculated using specific gravity of fine aggregate and shredded rubber tyres. Equation (1) and (2) were used to calculate the amounts of aggregate and shredded rubber tyres for a replacement fraction "α".

$$M_{\alpha,tyre} = M_{agg} * \alpha * \frac{SG_{tyre}}{SG_{agg}} \quad \text{Equation 1}$$

$$M_{\alpha,agg} = (1 - \alpha) * M_{agg} \quad \text{Equation 2}$$

Where,

α = fractional replacement of aggregate considered (0.075, 0.15, 0.25 and 0.35);

Mα,tyre = mass of shredded rubber tyre to be replaced with α replacement;

Mα,agg = mass of aggregate in the mix having $'\alpha'$ replacement;

Magg = total mass of aggregate in the control mix;

SG_{tyre} and SG_{agg} are specific gravity of shredded rubber trye and aggregate respectively.

With regard to rubber content and size, (Khatib and Bayomy, 1999); (Khatib and Bayomy, 1999); (Ishtiaq Alam, Umer Ammar Mahmood, 2013) have suggested a maximum rubber content of not more than 20% total aggregate volume and a size no bigger than crumb rubber size. The soft nature of rubber could be imagined acting like air voids within concrete, thus decreasing its ability to withstand loads, thereby leading to points of weakness within concrete mix.

3.4 Water-to-binder ratio

In engineering practice, water to binder ratio and degree of compaction are the two primary factors, the strength of concrete at a given age and cured at a prescribed temperature is assumed to depend upon (Neville, 2008). It is also described as the determining factor for penetrability of the hydrated cement paste and refers to the ratio of the total water to the total binder utilized (Grieve, 2009b). The microstructure of the hydrated cement paste and the associated mechanical properties depend largely on this ratio. According to Czernin (Czernin, 1980), at a w:b ratio of 0.40, the overall volume of hydrate exactly fills the space originally occupied by cement and water. Mixes with low water to binder ratio gains strength more rapidly than mixes with higher water to binder ratio, because in the former case, the cement grains are closer to one another and a continuous system is established more rapidly (Neville, 2008). When rubber is added to a concrete mixture, it may attract air as it has the tendency to repel water and then air may adhere to the rubber particles (Topçu and Unverdi, 2018). Hence, a low and high water/binder (w/b) ratios of 0.40 and 0.55 were utilized in the study.

3.5 Concrete mix proportions

Mix proportions of rubber-based concrete are prepared by replacing different percentage of coarse aggregate (or fine aggregate) in conventional concrete. Shredded rubber used as replacement material is a lightweight aggregate. And lightweight aggregates occupy more volume if replaced on weight basis. Therefore, replacement by volume is carried out. The method of mix design described by the Cement & Concrete Institute of South Africa is adopted in the study. It is derived from ACI Standard 211.1-91 (ACI, 2002) (Addis and Goodman, 2009). In this method, the mix constituents are calculated by mass, and the absolute volumes of each constituent should equate to the total absolute volume of concrete. The method is based on the following factors:

- Strength, at a given age for fully compacted concrete is governed by water/binder ratio and the type of cementitious material.
- Amount of water required per unit volume of concrete
- Optimum stone content depending on the stone and sand characteristics
- Volume of compacted concrete produced through a combination of materials

The mix design steps carried out in the study is guide lined by the following flow chart in figure 3.3, as adapted from Preez (Preez, 2019).Addis and Goodman (Addis and Goodman, 2009).

A total of twenty-four mix combinations were prepared with GGBS and FA for the study. The adopted methodology of specimen preparation involved replacement of variable amount of fine aggregate in 0%, 7.5%, 15%, 25%, 35 and 50%.

Summary of the mix proportions prepared are given in the following tables and the mix design calculations are presented in Appendix A. Table 3.2 and 3.3 gives the mix proportions with PC/GGBS and PC/FA respectively.

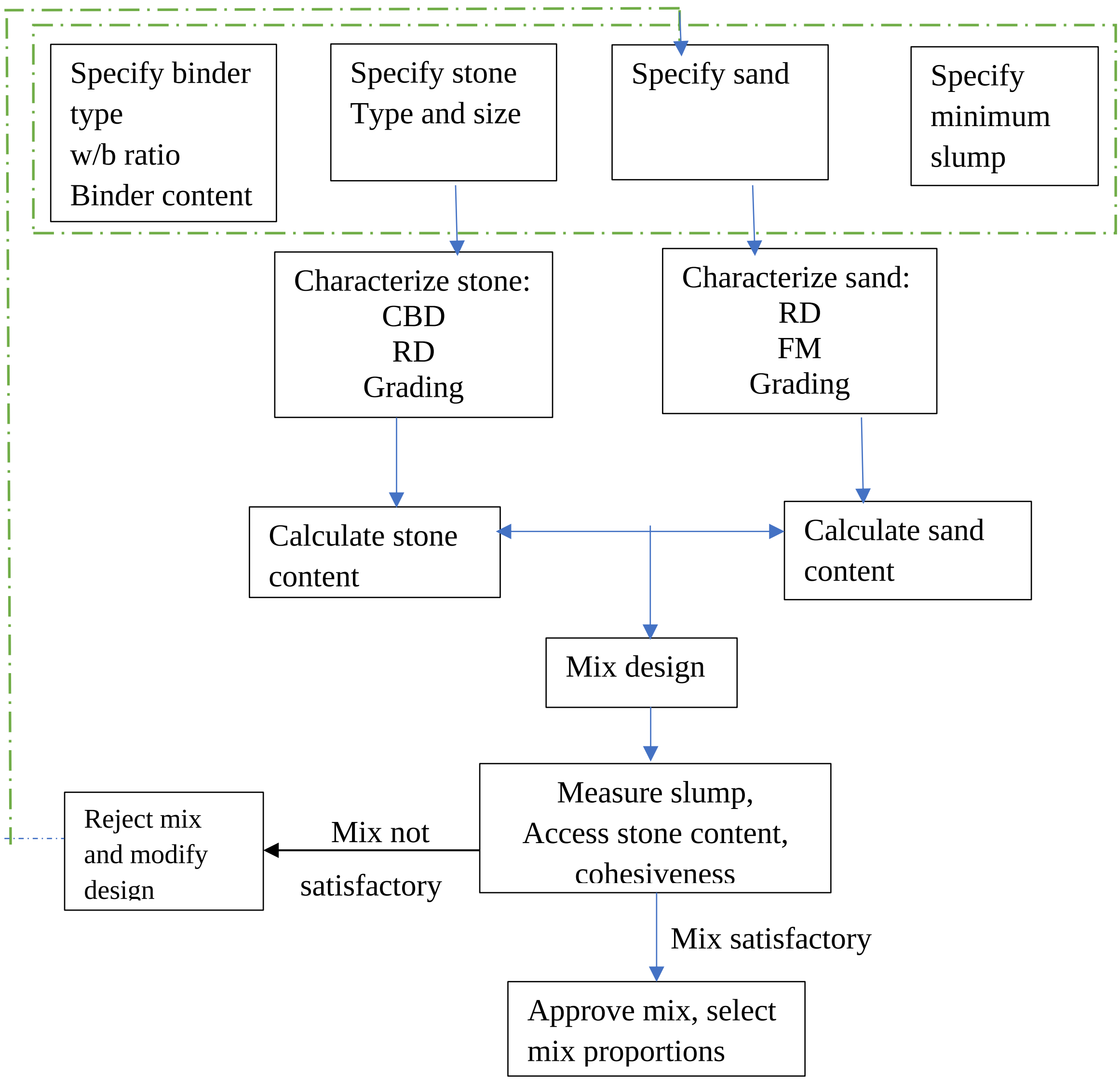

Figure 3. 3: Concrete mix design adapted from Preez (Preez, 2019)

Table 3. 2: Summary of mix proportions with GGBS

Material (kg/m^3)	50/50 PC/GGBS											
	0%sr		7.5%sr		15%sr		25%sr		35%sr		50%sr	
Mix identifier	Ga00	Gb00	Ga7.5	Gb7.5	Ga15	Gb15	Ga25	Gb25	Ga35	Gb35	Ga50	Gb50
w/b ratio	0.40	0.55	0.40	0.55	0.40	0.55	0.40	0.55	0.40	0.55	0.40	0.55
Combined binder	513	373	513	373	513	373	513	373	513	373	513	373
PC	256	186	256	186	256	186	256	186	256	186	256	186
GGBS	256	186	256	186	256	186	256	186	256	186	256	186
Fine aggregate	637	773	590	715	542	657	478	580	414	502	319	386
Shredded rubber(sr)	-	-	19	23	38	46	63	76	88	107	126	152
Coarse aggregate	1188	1188	1188	1188	1188	1188	1188	1188	1188	1188	1188	1188
Water Content	205	205	205	205	205	205	205	205	205	205	205	205

*Note: *Shredded rubbers(sr) replacing fine aggregate in 0, 7.5, 15, 25, 35 and 50%*

**a and b notations in mix identifiers represents 0.40 and 0.55 w/b ratio respectively.*

Table 3. 3: Summary of mix proportions with FA

Material (kg/m³)	70/30 PC/FA											
	0%sr		7.5%srG		15%srG		25%srG		35%srG		50%srG	
Mix identifier	Fa00	Fb00	Fa7.5	Fb7.5	Fa15	M-Fb15	Fa25	Fb25	Fa35	Fb35	Fa50	Fb50
w/b ratio	0.40	0.55	0.40	0.55	0.40	0.55	0.40	0.55	0.40	0.55	0.40	0.55
Combined binder	513	373	513	373	513	373	513	373	513	373	513	373
PC	359	261	359	261	359	261	359	261	359	261	359	261
FA	154	112	154	112	154	112	154	112	154	112	154	112
Fine aggregate	605	749	560	693	514	637	454	562	393	487	302	375
Shredded rubber(sr)	-	-	18	22	36	44	60	74	83	103	119	148
Coarse aggregate	1188	1188	1188	1188	1188	1188	1188	1188	1188	1188	1188	1188
Water Content	205	205	205	205	205	205	205	205	205	205	205	205

*Note: * shredded rubbers(sr) replacing fine aggregate in 0, 7.5, 15, 25, 35 and 50%*

**a and b notations in mix identifiers represent 0.40 and 0.55 w/b ratios respectively.*

3.6 Specimen type and sample size

A total of 216 cube specimens are prepared for testing compressive strength. Three 100 mm cubes are prepared per mixture to test at 7, 28, and 56. The specimens were tested at 7 and 28 days to determine the effect of the parameters on early and medium-term strength of the materials. Curing and testing was conducted up to 56 days to take account of the slow-maturing pozzolanic reactions of fly ash and latent hydraulic reactions of GGBS. The durability index tests required four 70mm by 30mm disc specimens per mixture combination equating to 120 tests per durability index. The same specimens were used for oxygen permeability and water sorptivity tests. The outline of the specimens is given in the following table 3.4:

Table 3. 4: Specimen details

<table>
<tr><th rowspan="2">Test</th><th rowspan="2">No: of Mixes</th><th rowspan="2">Curing Period (days)</th><th rowspan="2">Quantity per test</th><th colspan="2">Total Quantity</th></tr>
<tr><th>Disc</th><th>Cube</th></tr>
<tr><td rowspan="3">Compressive strength</td><td rowspan="3">24</td><td>7</td><td>3</td><td>-</td><td>72</td></tr>
<tr><td>28</td><td>3</td><td>-</td><td>72 (216)</td></tr>
<tr><td>56</td><td>3</td><td>-</td><td>72</td></tr>
<tr><td rowspan="2">Oxygen permeability</td><td rowspan="2">24</td><td>28</td><td>4</td><td>96 (192)</td><td>48 (96)</td></tr>
<tr><td>56</td><td>4</td><td>96</td><td>48</td></tr>
<tr><td rowspan="2">Water sorptivity</td><td rowspan="2">24</td><td>28</td><td>4</td><td>96 (192)</td><td rowspan="2">-</td></tr>
<tr><td>56</td><td>4</td><td>96</td></tr>
<tr><td rowspan="2">Chloride conductivity</td><td rowspan="2">24</td><td>28</td><td>4</td><td>96 (192)</td><td>48 (96)</td></tr>
<tr><td>56</td><td>4</td><td>96</td><td>48</td></tr>
<tr><td></td><td></td><td></td><td></td><td colspan="2">Total: 408</td></tr>
</table>

Note:

(i) Cube specimen size: 100mm.

(ii) Disc specimen size: 70±2mm in diameter and 30±2mm thick

(iii)Specimens from oxygen permeability test are reused for water sorptivity test.

3.7 Specimen casting and curing

Three 100mm cubes per mixture were prepared in accordance to SANS (SANS 5861-3, 2006). After casting, the specimens are demoulded, labelled and placed into a water bath at 23±2^{o} C for curing. The curing of concrete is described as keeping the concrete moist and warm enough so that the hydration of cement can continue. Cubes were cured for:

(i) 7, 28 and 56 days to obtain compressive strength and
(ii) 28 and 56 days to obtain durability indexes.

Compressive strength was determined by performing standard cube crushing test in accordance with SANS (SANS 5863, 2006) at 7, 28 and 56 days. This was carried out to determine the effect of parameters on early, medium and later term strength of the materials. For the durability index tests, test specimens considered are circular discs prepared by coring and cutting concrete cubes in the laboratory. The specimens are 70±2mm in diameter and 30±2mm thick concrete discs. Coring of cubes shall take place at 28±3 and 56±3 days after casting. After coring, the specimens are kept in the oven, maintained at 50±2^{o}C, for not less than 7 days and not more than 8 days. After drying, specimens are removed from oven and cooled in desiccator to 23±2^{o} C, for a minimum of 2 hours and a maximum of 4 hours. The specimens are tested within 30 minutes, once they are removed from the desiccator (DI-Manual, 2018).

3.8 Specimen testing

3.8.1 Compressive strength

The compressive strength of the concrete was determined according to SANS 5863 (SANS 5863, 2006). The specimens were tested at 7, 28 and 56 days to determine the range of strength development. The tests were conducted using an automated hydraulic compression machine at a loading rate of 0.25MPa per second up to failure and SANS (SANS 5863, 2006) recommends a loading rate of 0.3±0.1 MPa per second.

SANS 10100-Part 2 and 3 specifies a minimum characteristic cube strength of 15MPa for plain concrete and 20MPa if the concrete is to include embedded metal. According to SANS 10100-2, section 14.3.3.1, acceptance criteria for strength test results for site mixed concrete should meet the criteria that the mean of the three valid results shall exceed the specified characteristic strength by at least 2 MPa (SANS 10100 - 2, 2014; Viljoen, 2015).

3.8.2 Durability index tests

Durability is associated with the deterioration of the material over the intended service life of the structure in a given environment (Ballim, Alexander and Beushausen, 2009).

Chloride conductivity index, oxygen permeability index and water sorptivity index are the three durability index tests developed to measure the resistance of concrete to the transport mechanism of different deteriorating substances (Ballim, Alexander and Beushausen, 2009). Each test measures different transport property of fluids or ions through the cover layer of concrete. Oxygen Permeability Index (gaseous diffusion), Water Sorptivity Index (water absorption) and Chloride Conductivity Index (ionic diffusion) respectively. It is the resistance of these cover concrete to these transport processes that governs the deterioration of concrete and embedded steel and therefore defines the potential durability of concrete (Gouws, Alexander and Maritz, 2001).

The basis of the durability tests lies in the theory that durability can only be improved, when it is possible to measure parameters that control durability performance. The durability indices obtained from these tests methods are related to service life prediction models (Ballim, Alexander and Beushausen, 2009). For example, OPI is used as a predictive tool for estimating carbonation depths in structures, while CCI is used for chloride ingress estimations. These tests have proved sensitive to durability related aspects of concrete construction, such as materials, compaction and curing (Gouws, Alexander and Maritz, 2001). Specimen age have a significant effect on the test results, depending on the type of concrete and the curing procedure. It is not recommended that the test be performed before 28 days after casting. The specimens were tested at 28 and 56 days.

Determination of durability indices are relevant to both structural and non-structural concrete construction works. As these tests are sensitive to material, environmental and constructional factors which influences concrete durability. It is applied in quality control of site concrete, optimization of mix proportion of concrete, performance-based specifications and long-term performance predictions (Alexander, Mackechnie and Ballim, 1999). From the overview of OPI test results, the DI performance-based approach can be implemented in practice. Various issues arising in construction industry can be identified and addressed (Nganga, Beuhausen and Alexander, 2017). The different materials and their proportions used in the mixes will influence physical characteristics of the fresh and hardened concrete. The type of aggregates used may influence the water sorptivity. If concrete is to be designed for freeze/thaw exposure class, the freeze/thaw resistance will be high if the capillary action is low, but resistance will be low if the pores in the concrete are saturated (Zhang *et al.*, 2017)(PCA, 2002). Hence, knowing sorptivity index for non-structural concrete works is important. Chloride conductivity test is relevant as it is used in engineering practice for construction quality control, as it is sensitive to quality-related parameters such as water/binder ratio, binder type, curing and compaction (Streicher, 1997).

Table 3.5 shows the suggested ranges for durability classification of concretes for the three index tests, based on site and laboratory data, i.e. performance-based specification.

Table 3. 5: Suggested ranges for durability classification using index (Alexander, Mackechnie and Ballim, 1999)

Durability class	OPI (log scale)	Sorptivity (mm/hr$^{0.5}$)	Conductivity (mS/cm)
Excellent	>10	< 6	< 0.75
Good	9.5 – 10	6 – 10	0.75 – 1.50
Poor	9.0 – 9.5	10 – 15	1.50 – 2.50
Very poor	< 9.0	>15	>2.50

Local authorities like South African Roads Agency Limited (SANRAL), compares the results of durability parameters of concrete used for specific structures to limiting index values as given in tables 3.8 and 3.9. According to Alexander and Beushausen (2009), deemed-to-satisfy approach is considered an adequate approach to specifying durability index for most of the reinforced concrete construction. Table 3.6 gives the requirements of durability criteria specified for SANRAL project - New England Road Bridge subjected to carbonation induced corrosion (Ronny, 2011). Table 3.7 gives the maximum chloride conductivity values.

Table 3. 6: Durability parameters acceptance ranges (SANRAL project target-New England Road Bridge) (Angelucci, 2013) (Ronny, 2011)

Acceptance level	**Oxygen Permeability (Log Scale)**	**Water Sorptivity (mm/$\sqrt{hr}$)**
Concrete made, cured and tested in the laboratory (average of 2 tests)	> 9.80	OPI * 1.1 (max. 12)
Full acceptance of in-situ concrete	> 9.70	12
Conditional acceptance of in-situ concrete (with remedial measures approved by the Engineer)	8.75 – 9.70	12 - 15
Rejection	< 8.75	>15

Table 3. 7: Maximum chloride conductivity values (mS/cm) for different exposure classes and binder types: deemed to satisfy approach – common structures, cover = 50mm (Alexander et al., 2008) (Angelucci, 2013) (SANRAL)

EN 206 class	**Description**	**70:30 CEM I : Fly Ash**	**50:50 CEM I : GGBS**
XS 1	Exposed to airborne salt but not in direct contact with seawater or inland saline waters	3.00	3.50
XS 2a	Permanently submerged in sea (or saline water)	2.45	2.60
XS 2b	XS 2a + Exposed to abrasion	1.35	1.60
XS 3a	Tidal, splash and spray zones	1.35	1.60
XS 3b	XS3a + Exposed to abrasion	1.10	1.25

3.8.2.1 Chloride conductivity test

Chloride conductivity index is a quality control parameter used to assess the resistance of concrete to chloride ingress (Otieno and Alexander, 2015).

Diffusion is the process by which liquid, gas or ions move through a porous material under the action of a concentration gradient. In chloride conductivity (CC) test, chloride ions move through all pores of sufficient size. The test, therefore, provides a good indication of the overall diffusivity of the material, the test being sensitive to changes in the pore structure and cement chemistry. In marine environment, chloride diffusion is the main process by which chloride enters concrete (Gouws, Alexander and Maritz, 2001).

A detailed test procedure is available in the Durability Index Testing Manual (DI-Manual, 2018)(SANS3001-CO3-3, 2015). In summary, the cored specimens (discs) of 70 ± 2 mm diameter, 30 ± 2 mm thick are prepared from concrete cubes. Four specimens are tested for each concrete. Prior to testing, the specimens are dried in an oven at 50°C for not less than seven days and not more than eight days. The specimens are then vacuum-saturated for 24 hours with 5M NaCl solution before being placed in a test rig with a cell filled with the same salt solution on either side. A 10 V potential difference is applied across the specimen, and the current through the specimen is measured in a very short time (typically 10 seconds). Fig 3.4 gives the schematic representation of the test set up. The chloride conductivity index (CCI) is then calculated using the formula (Otieno and Alexander, 2015):

$$\sigma = \frac{[i]*[d]}{[V]*[A]} \qquad \text{Equation 3}$$

where,

σ = chloride conductivity index of the specimen (mS/cm)

i = electric current through the specimen (mA)

V = voltage difference across the specimen (V)

d = average specimen thickness (cm)

A = cross-sectional area of the specimen (cm^2)

In chloride conductivity test, chloride ions move through all pores of sufficient size, therefore, it provides a good indication of the overall diffusivity of the material. The test is sensitive to changes in pore structure and cement chemistry. Chloride conductivity index is a quality control parameter used to assess the resistance of concrete to chloride ingress (Otieno and Alexander, 2015).

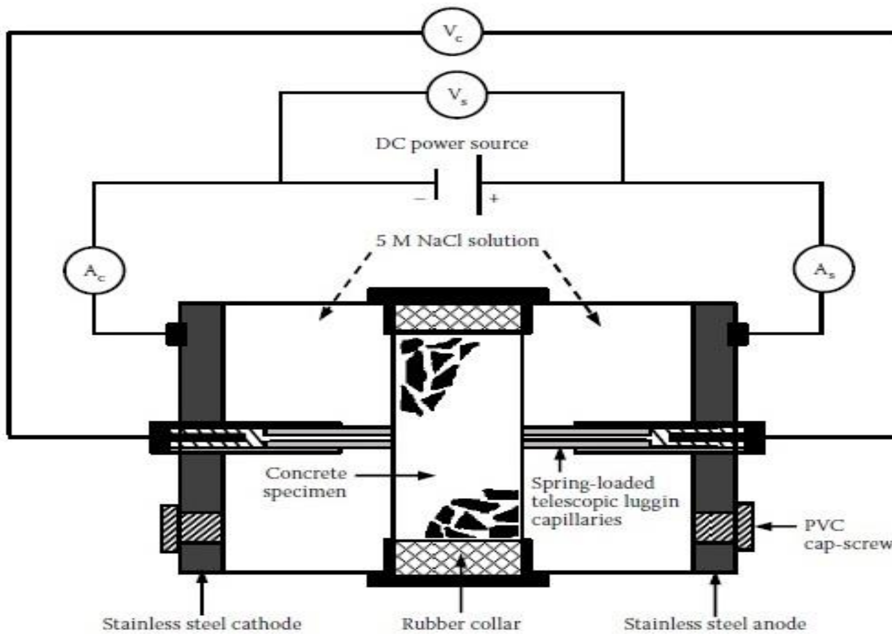

Figure 3. 4: Schematic of Chloride Conductivity Test (SANS3001-CO3-3, 2015)

Typical chloride conductivity index values for concrete range from very poor (>2.50mS/cm) to excellent (<0.75 mS/cm). Lower the index, better the potential durability of the concrete (Gouws, Alexander and Maritz, 2001). CCI values focus on binder type.

3.8.2.2 Oxygen permeability test

This test method consists of measuring the pressure decay of oxygen through the concrete disc specimens placed in a falling head permeameter. Figure 3.5 shows the apparatus arrangement. The tests are conducted on concrete disc specimens of 70±2mm diameter and 30±2mm thick. As per the Durability Index Testing Manual (DI-Manual, 2018) they are preconditioned by oven drying at $50 \pm 2°C$ for not less than 7 days and not more than 8 days. After drying the specimens are removed from the oven and placed in a desiccator to cool to $23 \pm 2°C$ for a minimum of 2 hours and a maximum of 4 hours. The specimens are then weighed, and the thickness and diameter measured. Afterwards the specimens are placed in compressible rubber collars and inserted into a falling head permeameters. The collars ensure that all oxygen losses occur through the specimen. The apparatus is charged with oxygen, at an initial pressure of 100 ± 5kPa, and allowed to decay to 50kPa or up to a period of 6 hours, whichever occurs first. During this period, the pressure drop is recorded at 15-minute intervals. Fig 3.5 gives the schematic representation of the test set up.

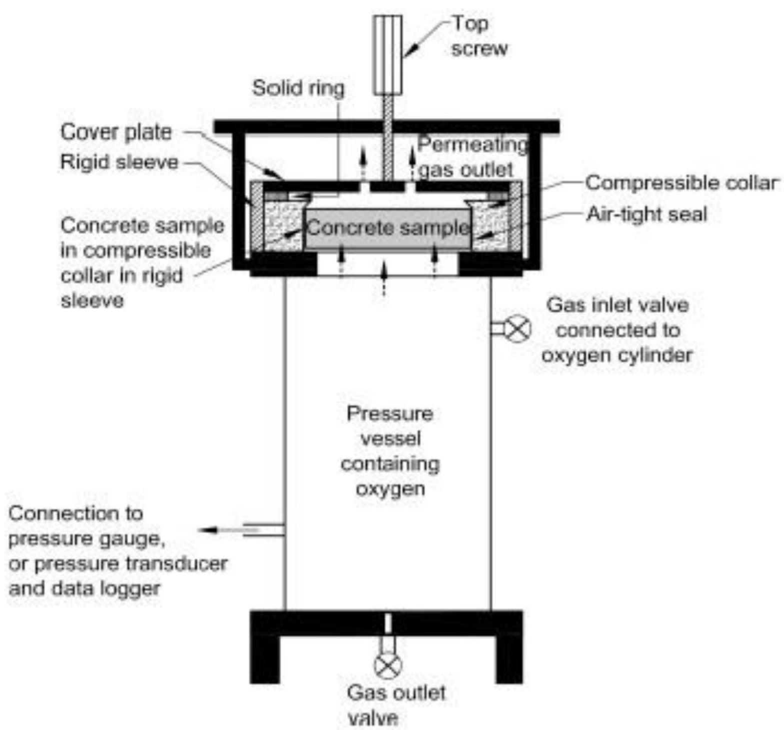

Figure 3. 5: Schematic of Oxygen Permeability Test (SANS3001-CO3-2, 2015)

The oxygen permeability index is calculated from the D'arcy coefficient of permeability. It is determined from the following equation:

$$k = \frac{\omega V g d}{R A \Theta t} \ln \left(\frac{P_o}{P_t}\right) \quad \text{Equation 4}$$

where,

k = coefficient of permeability (m/s)

ω= molecular mass of permeating gas (0.032 kg/mol)

V = volume of the pressure cylinder (m^3)

g = acceleration due to gravity (9.81 m/s^2)

d = specimen thickness (m)

R = universal gas constant (8.313 Nm/Kmol)

A = cross-sectional area of specimen (m^2)

Θ = absolute temperature (K)

t= time (sec)

P_o, P_t = pressure at start of test and at time t respectively (kPa)

The best fit line is determined using linear regression of ln $\left(\frac{P_o}{P_t}\right)$ against time t and forcing the regression line through the (0,0) point. The coefficient of correlation (r^2) should be greater than 0.99 and it is calculated from the equation:

$$r^2 = 1 - \frac{\sum[t_i - t_{p,i}]^2}{\sum [t_i^2] - \left[\frac{(\sum t_i)^2}{n}\right]} \quad \text{Equation 5}$$

where,

t_i = time at any given pressure reading, recorded to the nearest minute, in seconds (s);

$t_{p,i}$ = predicted time at the same pressure reading (based on the linear regression), in seconds (s);

n = number of data points being considered.

The value of $t_{p,i}$ can be calculated from:

$$t_{p,i} = \frac{ln\left({}^{P_o}/_{P_t}\right)}{z} \quad \text{Equation 6}$$

where,

P_0 and P_t is same as defined above

z = slope of linear regression, defined as:

$$z = \frac{\sum\left[\ln\left(\frac{P_o}{P_i}\right)\right]^2}{\sum\left[ln\left(\frac{P_o}{P_t}\right)\ t\right]} \qquad \text{Equation 7}$$

In order to simplify the above coefficient, the "oxygen permeability index" is defined as the negative log of the k value as follows:

$$OPI = -\log_{10} k \qquad \text{Equation 8}$$

Cement paste is a porous material, permitting the diffusion of gases into concrete along with fluids. The resistance of cement-based materials against oxygen ingress under pressure is determined. Durability of cementitious materials is strongly related to permeability, i.e. the determination of material constant under defined conditions. Therefore, the assessment of resistance against ingress of gases, for example CO_2, which can react with cement is relevant, making oxygen permeability index determination important (Loser, 2017).

Higher the index, less permeable is the concrete (Gouws, Alexander and Maritz, 2001).

3.8.2.3 Water sorptivity test

Sorptivity is defined as the rate of movement of a wetting front through a porous material. The water sorptivity test involves the uni-directional absorption of water into one face of a pre-conditioned concrete disc specimen. At predetermined time intervals the sample is weighed to determine the mass of water absorbed, the sample is then vacuum-saturated with water and the mass determined. The same specimens used in the OPI test were used for WSI. The sorptivity of the material is determined from the plot of mass of water absorbed versus square root of time. The lower the water sorptivity index, the better is the potential durability of the concrete (Gouws, Alexander and Maritz, 2001).

Concrete disc specimens 70 ±2mm diameter, 30 ±2mm thick with low moisture contents at the start of the test needs to be ensured. The test specimens are exposed to a saturated calcium hydroxide solution and the specimens are weighed at 3, 5, 7, 9, 12, 16, 20- and 25-minutes intervals to determine the mass of absorbed solution (DI-Manual, 2018).

The water sorptivity of the specimen (S), in mm/$\sqrt{h}$, is given by:

$$S = \frac{((F)*(d))}{(M_{sv}) - (M_{s0})} \quad \text{Equation 9}$$

where,

F = slope of best fit line (g/$\sqrt{h}$)

d = average specimen thickness to the nearest 0.02mm, (mm)

M_{sv} = vacuum saturated mass of the specimen, (grams)

M_{s0} = mass of the specimen at initial time t_0, (grams)

The slope of the line of best fit, F is calculated by linear regression analysis:

$$F = \left[\frac{\sum(\sqrt{t_i} - T)(M_{wti} - \acute{M}_{wt})}{\sqrt{\sum(\sqrt{t} - T)^2 \sum (M_{wti} - \acute{M}_{wt})^2}}\right]^2 \quad \text{Equation 10}$$

$$\acute{M}_{wt} = \frac{\sum M_{wti}}{n} \quad \text{Equation 11}$$

$$T = = \frac{\sum\sqrt{t_i}}{n} \quad \text{Equation 12}$$

$$M_{wti} = M_{st} - M_{s0} \quad \text{Equation 13}$$

$$M_{wt} = F\sqrt{t} \quad \text{Equation 14}$$

where,

t = time in hours after a specimen was first exposed to water on its lower face, (hours);

M_{wti} = mass gain at any given time, (grams);

t_i = time corresponding to the mass gain Mwti, (hours);

M_{st} = mass of the specimen at particular time t, (grams);

M_{s0} = mass of the specimen at the initial time t_0, (grams);

M_{wti} = mass gain

Lower water sorptivity values indicate higher quality of concrete cover. Typical water sorptivity values range from 3.5 mm/$\sqrt{h}$ to 9.5mm/$\sqrt{h}$.

The next chapter gives the details of the results obtained from the laboratory tests and analyses the conclusions drawn from these results.

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CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 Introduction

This chapter analyses and presents the results of the experimental works carried out as outlined in Chapter 3. Compressive strength and durability indices of the specimens containing rubber particles are investigated and discussed, with respect to the w/b ratio and binder type.

The trends of the resulting data are analyzed and the practical implications of the mix design with replacement of fine aggregates by shredded tyre rubber particles are outlined. The error bars indicated on the figures, are graphical representations of variability of the data. They indicate how far from the reported measurement the true value might be. They represent the 95% confidence interval, refer to Appendix A for calculation details.

4.2 Workability

4.2.1 A revisit of the workability requirements for non-structural concrete

Slump requirements vary appropriate to the type of construction or within such other limits as maybe required by the Engineer. Slump is an important factor in construction, since it affects the ease with which the concrete can be placed and consolidated to minimize honeycomb and bugholes. For non-structural concrete, no or lesser amount of reinforcement are required compared to reinforced structural concrete works. Hence, less workable concrete mix can be readily used for non-structural works as they can be easily vibrated. For example, in accordance with SANS 5862, the slump measured should be between 20mm and 60mm for plain footings and substructure walls in vibration compaction.

4.2.2 Effect of w/b ratio and binder type on workability

Figure 4.1 shows slump values of GGBS and FA rubberized concrete at different w/b ratio. It can be noted that slump showed values below 15mm. According to BS EN 12350-2, slump test is sensitive to changes in the consistence of concrete which corresponds to slumps between 10mm and 210mm. Beyond these extremes slump is considered unsuitable (BS-EN12350-2, 2019). It was observed that there was very little change in consistency of the mix designs. Hence, other methods, for example Vebe test; of determining consistence should be considered. In general, GGBS and FA binder tends to improve workability (Kellerman and Crosswell, 2009). FA binder type control mix concrete shows better workability compared to GGBS binder type control mix concrete. FA binder improves workability due to its microscopic, spherical-shaped particles which create a more flowable, easier-to-finish concrete (Rosenberg, 2010). As explained earlier in chapter 3, fresh concrete containing GGBS tends to require less energy for movement, which makes it easier to place and compact, especially when pumping and or using mechanical vibrators. According to Page and Fanourakis (2008), who investigated the influence of GGBS of fineness 5000 cm^2/g or GGBS (5000) and 3600 cm^2/g or GGBS (3600), on the workability of concrete, revealed that the finer slag , GGBS (5000), exhibited higher viscosity and hence lower workability than GGBS (3600), due to its relatively higher surface area (Page and Fanourakis, 2008). Addition of shredded tyre rubber particles influences the workability of concrete mixes. This is explained in section 4.2.3.

Figure 4.2 shows the comparisons of slump values, for GGBS and FA binder type specimens prepared using different w/b ratios. Data points on figure 4.2 falls below the line of equality, indicates higher slump values for 0.55 water/binder (w/b) ratio as compared to 0.40 w/b ratio. As water increases, workability of the concrete improves. The figure further shows that in few mixes, slump values are sensitive to changes in rubber content percentage and w/b ratio in GGBS binder type concrete mix when compared to FA binder type, as data points further offset from the line of equality for GGBS binder type concrete mixes.

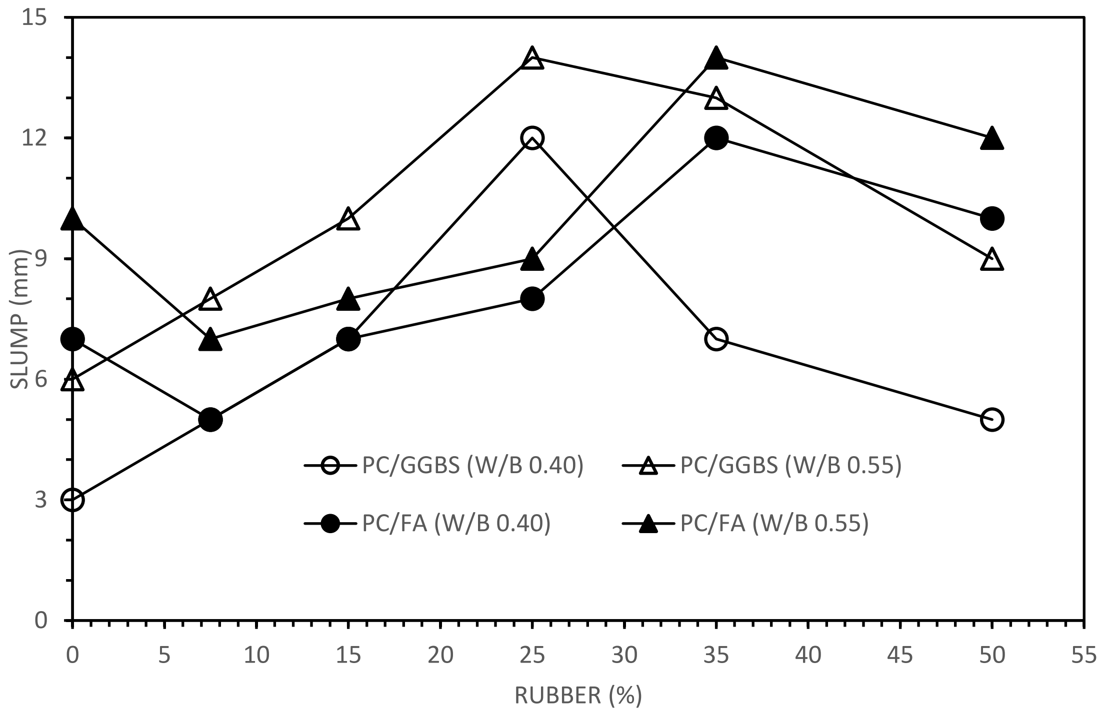


Figure 4. 1: Slump values of PC/GGBS and PC/FA concrete

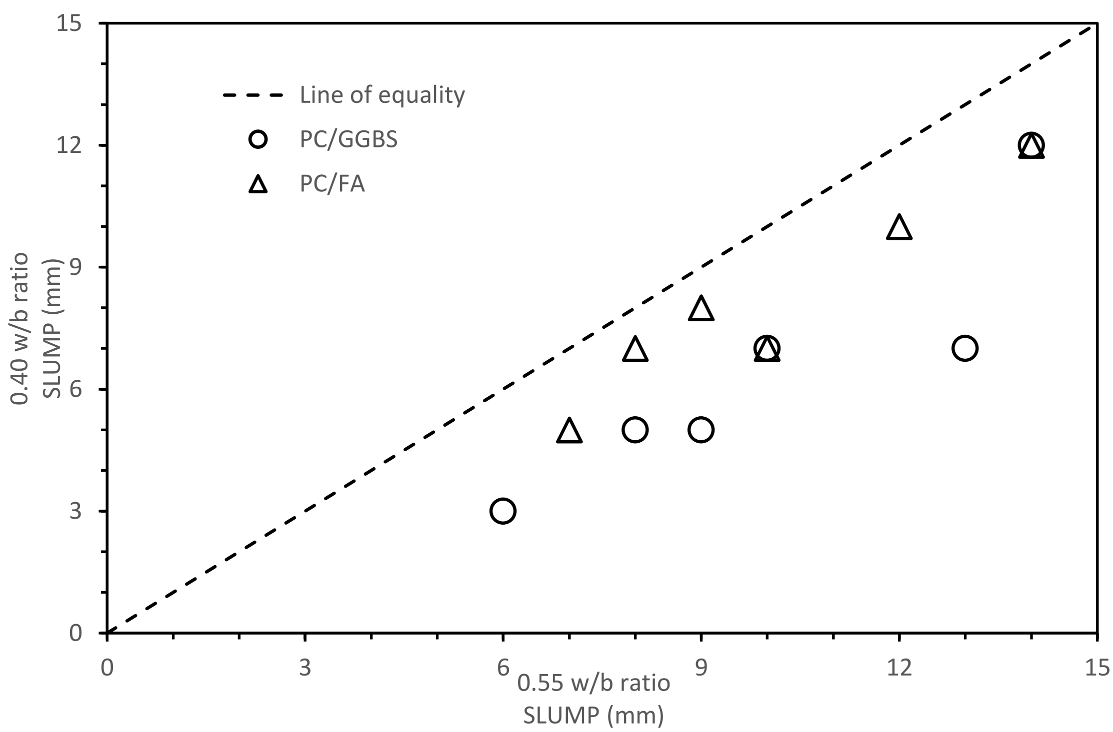
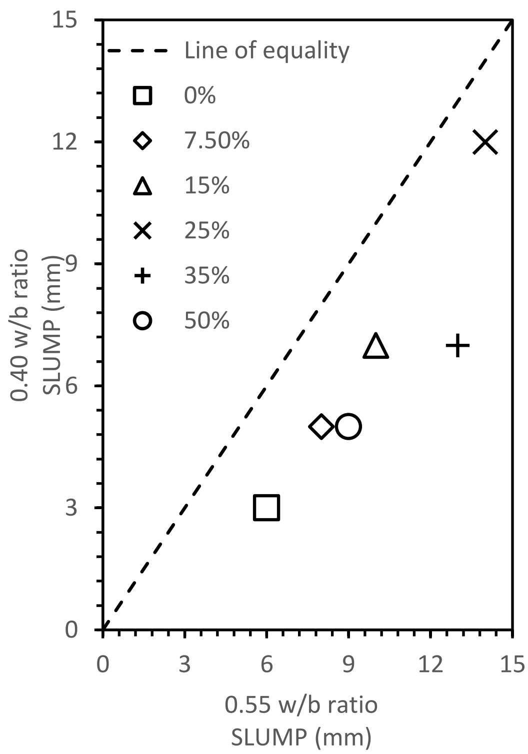
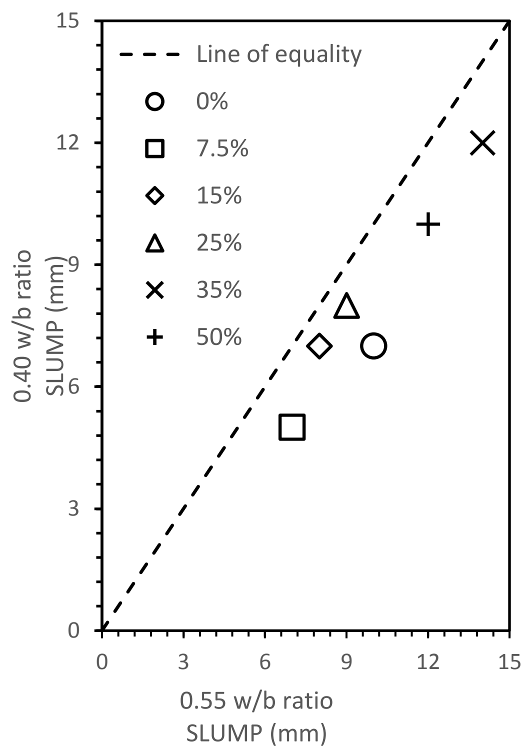


Figure 4. 2: Effect of w/b ratio on workability of different binder type concrete

4.2.3 Effect of rubber content on workability

Figure 4.3 (i) and (ii) plots the slump at w/b ratios of 0.40 and 0.55, for GGBS and FA concrete mixes with respect to increase in percentage of shredded tyre rubber particles replacing fine aggregate by volume.





4.3(i) GGBS binder *4.3 (ii) FA binder*

Figure 4. 3: Effect of varying rubber percentage on workability at different w/b ratios for (i)GGBS binder concrete (ii)FA binder concrete

It can be noted that slump showed values below 15mm. BS EN 12350:2 recommends slump test suitable only for slumps between 10mm and 210mm. It was observed that in GGBS binder type mixes with lower w/b ratio of 0.40, slump showed value greater than 10mm, only in concrete mix containing 25% rubber content. With an increase in w/b ratio to 0.55, concrete mixes with 25% and 35% rubber particles showed a slump value greater than 10mm. Similarly, in FA binder type mixes, it is observed that at comparatively lower w/b ratio of 0.40, only those mixes containing 35% rubber content showed slump values greater than 10mm. At higher w/b ratio of 0.55, mixes containing both 35% and 50% rubber content showed values greater than 10mm. However, it can be noted that as per BS EN 12350:2, slump test showed slight sensitiveness to changes in the consistence of concrete with the introduction of shredded tyre rubber content in both GGBS and FA binder type concrete mix as compared to control. The increase in workability between 25% and 35% rubber content can be due to improvement of the fine and coarse aggregate grading.

The variations in slump values can be quantified with respect to control mix at lower water binder ratio of 0.40 in GGBS binder type rubberized concrete, as a slump increase by 300% at 25% rubber content. At higher water binder ratio of 0.55, slump values increased to 133.3% and 116.7% at 25% and 35% rubber content with respect to control mix. However, increase in rubber content shows decrease in slump value beyond 25% rubber content. In FA binder type it was observed that, at lower w/b ratio slump value increased by 71.4% at 35% rubber content with respect to control mix. At higher water binder ratio, slump values increased to 40% at 35% rubber content and 20% at 50% rubber content with respect to control mix. Increase in rubber content shows decrease in slump value beyond 35% rubber content. The reduction in workability observed in GGBS and FA binder type concrete mixes as percentage of tyre rubber particle increases could be the result of surface features of rubber. The inclusion of waste rubber sand in the mixture reduced the workability which could be related to the higher water absorption of rubber sand compared to natural sand (Rashad, 2016). This observation supports the studies on workability of controlled low strength rubber concrete (CLSRC) mixtures at different contents of rubber by Wang, Chen and Wu (2013), where natural sand was partially replaced with rubber (size 4.75 mm) at levels of 0%, 10%, 20%, 30% and 40%, by volume. The results showed an increase in the slump value with the inclusion of 10% rubber sand, whilst a slump value reduction was noted with the inclusion of 20%, 30% and 40% rubber, respectively (Wang, Chen and Wu, 2013).

Khatib and Bayomy (1999) investigated the workability of rubber concrete and reported that there was a decrease in slump with increase in rubber content as a percentage of total aggregate volume. They further noted that at rubber contents of 40%, slump was almost zero and concrete was not workable manually. (Batayneh, Marie and Asi, 2007, 2008)(Khatib and Bayomy, 1999) have reported reduced workability on the inclusion of rubber tyre particles. On the contrary, few other studies (Rashad, 2016), have reported positive effect of rubber particles on workability. However, a few other studies believed that some rubber sand contents reduced workability, whilst other contents increased it (Khaloo, Dehestani and Rahmatabadi, 2008; Sallam *et al.*, 2008; Pelisse *et al.*, 2012).

4.2.4 Practical implications for rubberized concrete on the workability of non-structural concrete

It was observed that, even though results exhibited less than 10mm slump values for no and lower percentage of rubber content, mix concretes with comparatively higher percentage of rubber content exhibited sensitivity to slump test. Addition of tyre rubber particles slightly improved the workability of GGBS and FA concrete at 25% and 35% rubber content. However, beyond these replacement levels, workability started decreasing in both GGBS and FA binder type concrete mixes. Irrespective of binder type and percentage of rubber content, increasing w/b ratio, increased the slump of the mix. From a practical viewpoint, the workability of the concrete mix designed is significant.

For non-structural concrete, compaction of concrete is not as difficult as in the reinforced concrete constructions with high percentage of steel reinforcement, where compaction is difficult to be carried out with vibrators or other equipment. In these cases, workability of concrete should be such that concrete flows to each part of the member. For concrete members where it is easier to compact the concrete, low workability concrete can also be used. Therefore, a reduction of workability with addition of shredded tyre rubber particles does not hinder its application in non-structural concrete.

Workability needs can be catered using water reducing admixtures where needed. As per Cement & Concrete Institute (C&CI) method, derived from ACI Standard 211, for concrete mix design with slump range less than 25mm, the placing of concrete requires heavy vibration. The required slump can be attained using different admixtures such as high range superplasticizers and water reducing chemical admixtures. For example, certain modified polycarboxylate polymers are high range water reducing superplasticizer which can be added to the mix design to attain the required slump, for better workability of the mix.

4.3 Compressive strength

4.3.1 A revisit of the strength requirements for non-structural concrete

Depending on the nature of use in non-structural work, concrete strength requirements varies. SANS 10100-Part 2 and 3 specifies a minimum characteristic cube strength of 15MPa for plain concrete and 20MPa if the concrete is to include embedded metal. According to SANS 10100-2, section 14.3.3.1, acceptance criteria for strength test results for site mixed concrete should meet the criteria that the mean of the three valid results shall exceed the specified characteristic strength by at least 2 MPa.

4.3.2 Effect of w/b ratio and binder type on compressive strength

Figures 4.4 and 4.5 presents the average early and long-term compressive strength results for rubberized concrete mixes at w/b ratios of 0.40 and 0.55, with binder types of GGBS and FA. Figure 4.6 and 4.7 presents the average strength development of the GGBS and FA binder type concrete mixes, up to 56 days.

It can be observed that compressive strength decreases with increasing rubber content. These reductions correspond to the results observed by many researchers (Khatib and Bayomy, 1999)(Rashad, 2016; Luong *et al.*, 2017)(Pardeshi *et al.*, 2017). Improvements were noted in strength as the concrete age increased, irrespective of binder type, w/b ratio and percent of rubber content. This beneficial role of prolonged curing in increasing strength, can be attributed to the ongoing hydration process of the cementitious material which leads to the decrease in porosity and interconnectivity of pores with time (Ballim, 1993).

During hydration reaction between Portland cement and water, or pozzolanic reaction between hydration product Portlandite and cement extenders, the microstructure of paste depends largely on the w/b ratio of the mixture. FA undergo a pozzolanic reaction in presence of water with cement hydration product Portlandite to produce CSH gel, the main strength contributor of the hardened cement paste. Strength development starts off slowly in FA concrete but matures over time. GGBS on other hand undergoes its own hydration reaction when activated by Portland cement. GGBS contributes to strength by its gradual hydration reaction which refines the pore structure over time. Therefore, with prolonged curing, concrete strength characteristics are enhanced over time with GGBS and FA binder type (Angelucci, 2013).

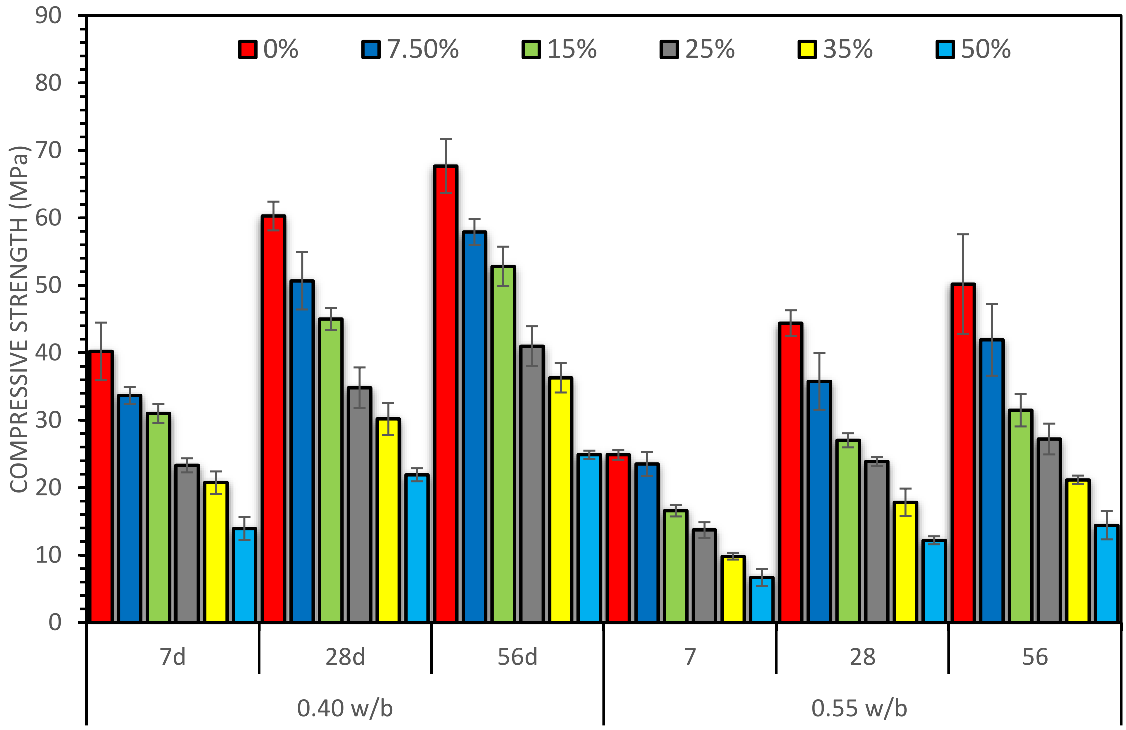


Figure 4. 4: Average compressive strength at 7, 28 and 56 days with GGBS binder for varying rubber content

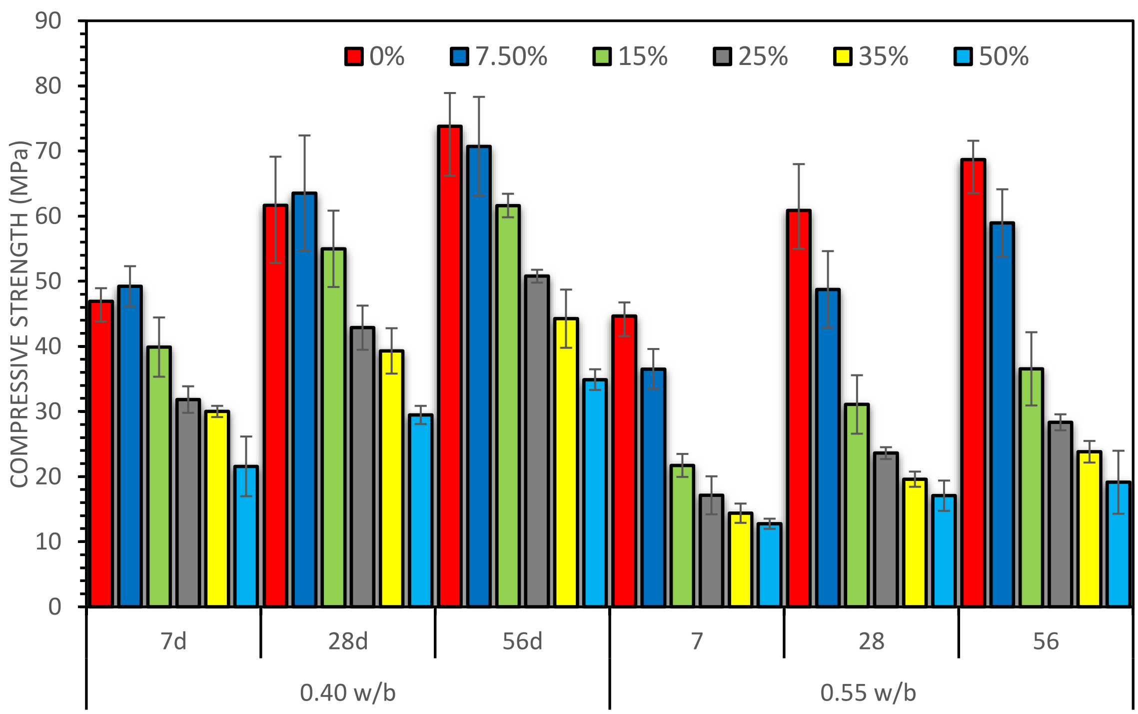


Figure 4. 5: Average compressive strength at 7, 28 and 56 days with FA binder for varying rubber content

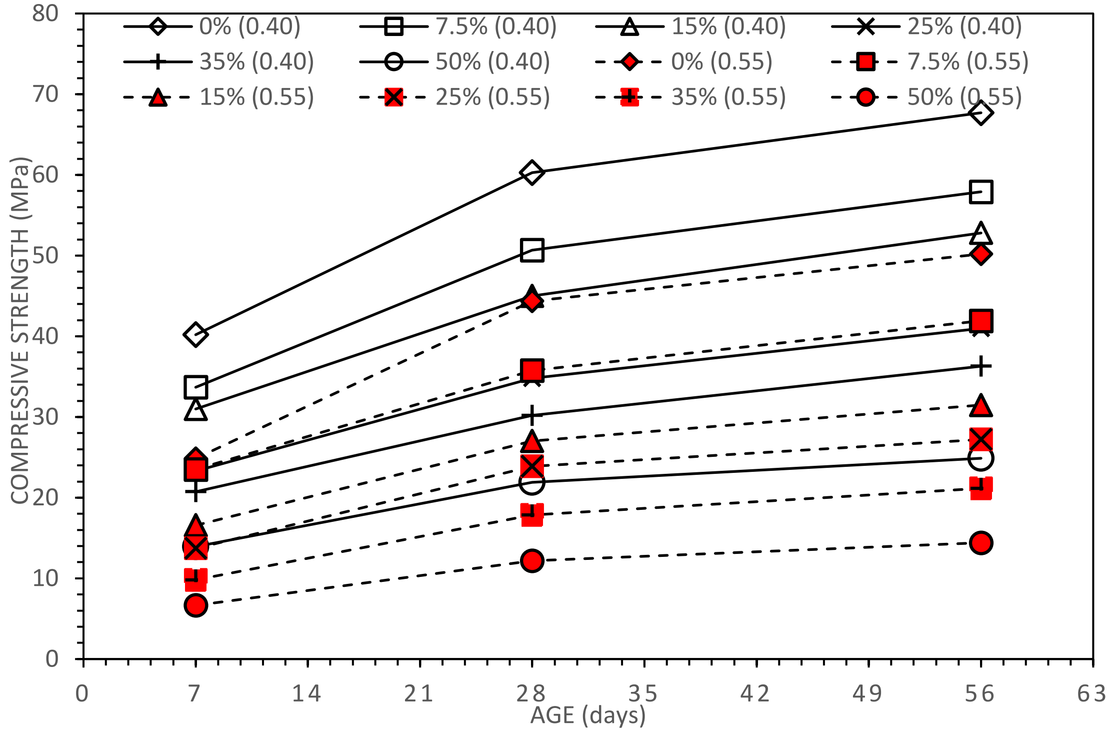


Figure 4. 6: Average compressive strength development of GGBS binder type concrete

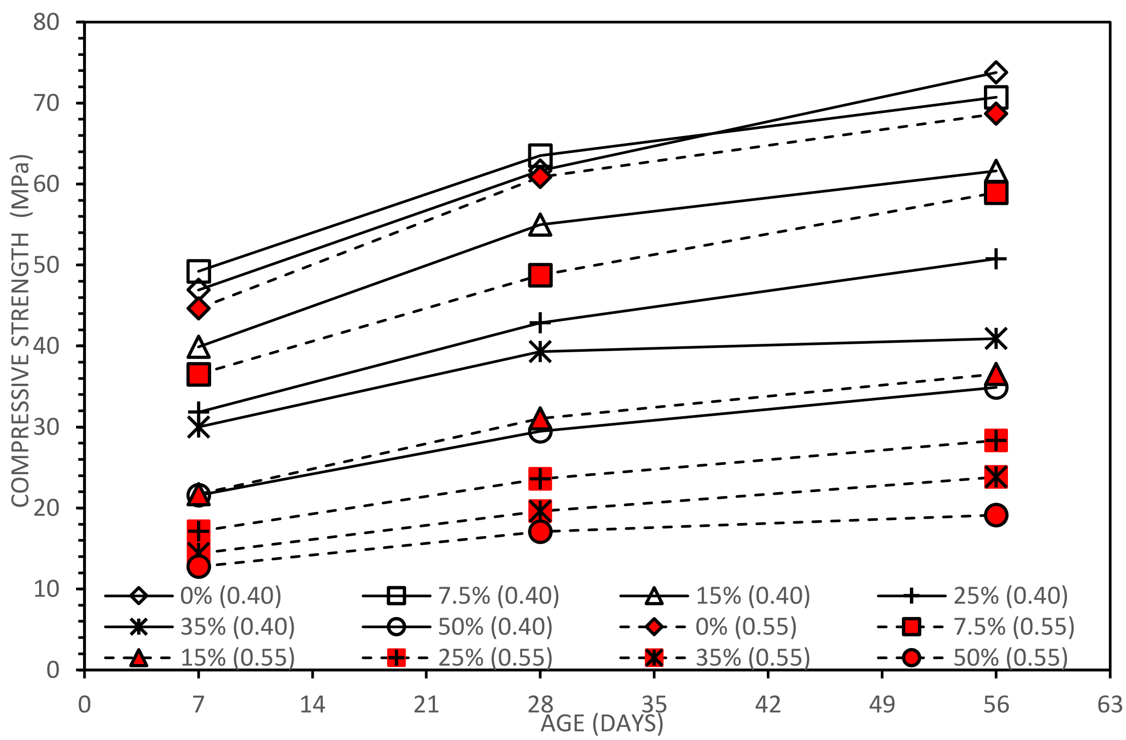


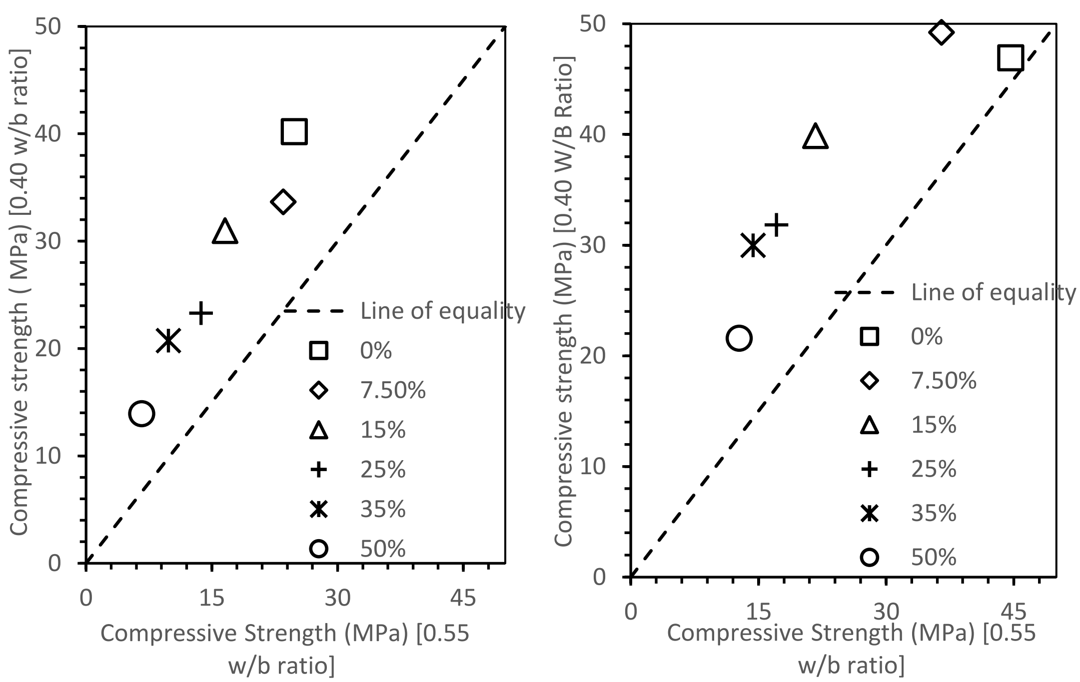
Figure 4. 7: Average compressive strength development of FA binder type concrete

Figures 4.8 (i) and (ii) to 4.10 (i) and (ii) depicts the effect of w/b ratio on compressive strength of concrete mix at 7, 28 and 56 days for two binder types respectively. Data points falls above the line of equality, which indicates higher compressive strength for 0.40 w/b ratio as compared to 0.55 w/b ratio. These graphs further show that FA mixes are more sensitive to changes in w/b ratio compared to GGBS binder type concrete mix, as data points further offset from the line of equality for FA binder type concrete mixes. This indicates that changes in w/b ratio have greater influence on workability of FA binder type concrete mixes.

When evaluating the influence of the w/b ratio on the compressive strength results, for a given binder type, the 0.40 w/b ratio mixes yielded higher strength values than the 0.55 w/b ratio mixes, these reductions in strength with increasing w/b ratio correspond to numerous literature sources (Angelucci, 2013)(Preez, 2019)(Mukadam, 2014). At lower w/b ratio, distance between binder particles reduces, thus reducing the distance between hydrates of neighboring particles. This will result in a dense hydrated cement microstructure. As w/b ratio increases, the larger binder inter-particle spacing that leads to the hydration products extending further to interact with one another and results in a less dense hydrated cement microstructure (Mukadam, 2014; Preez, 2019).

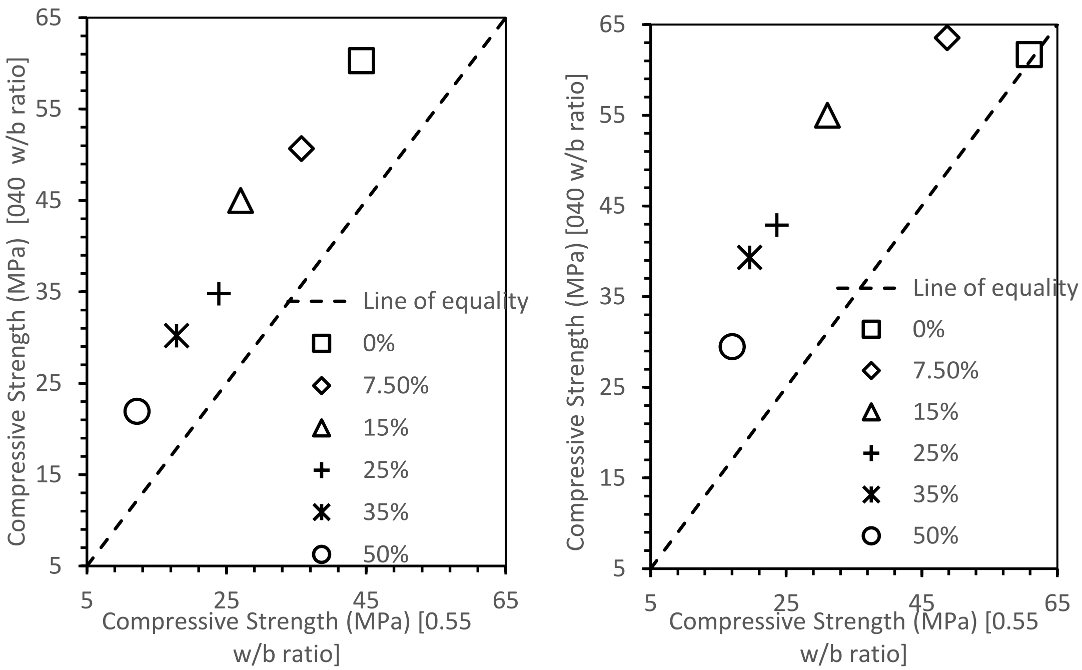
It is observed that at 56-days, with an increase in w/b ratio from 0.40 to 0.55, there is a strength decrease of 26% in PC/GGBS binder type control mix, and of 7% in PC/FA binder type control mix. Similarly, in rubberized concretes with varying percent of rubber content, a decreasing pattern in compressive strength is noticed with increase in w/b ratio. Irrespective of binder type, strength decreases with an increase in w/b ratio. However, mix concrete with FA binder type gives better strength than GGBS binder (Angelucci, 2013). GGBS binder type have a higher Blaine fineness or specific surface area (SSA) compared to FA. As the particle size decreases, the strength increases, because of the increase in specific surface area which is required for the pozzolanic reaction to take place (Nagendra, Sashidhar and Kumar, 2018). The incorporation of FA improves the performance, because of its unique spherical shape, and chemical properties (Rycroft, 2017). The improvement of strength of concrete by FA is the result of its pozzolanicity as well as the ability of the very small fly ash particles to fit in between cement particles (Neville, 2008a).

The FA and GGBS binder rubberized concrete mixes, containing shredded rubber up to 50%, meets the specified minimum strength requirement of concrete for non-structural applications with and without reinforcement at 28 days for lower w/b ratio of 0.40 and surpasses the minimum requirement at prolonged curing. On other hand it is observed that for higher w/b ratio of 0.55, GGBS and FA concrete meets the specified target strength requirement of 17 MPa with up to 35% and 50% rubber replacement respectively for non-structural plain concrete works. On other hand, for non-structural applications including reinforced concrete, concrete with both binder type satisfies the specified target strength requirement of 22MPa only with maximum 25% rubber content. However, and FA with 35% maximum rubber content respectively. However, on prolonged curing, FA surpasses the condition at 35% maximum rubber content.



4.8 (i) GGBS binder concrete *4.8 (ii) FA binder concrete*

Figure 4. 8: Effect of w/b ratio on compressive strength at 7 day for different binder types

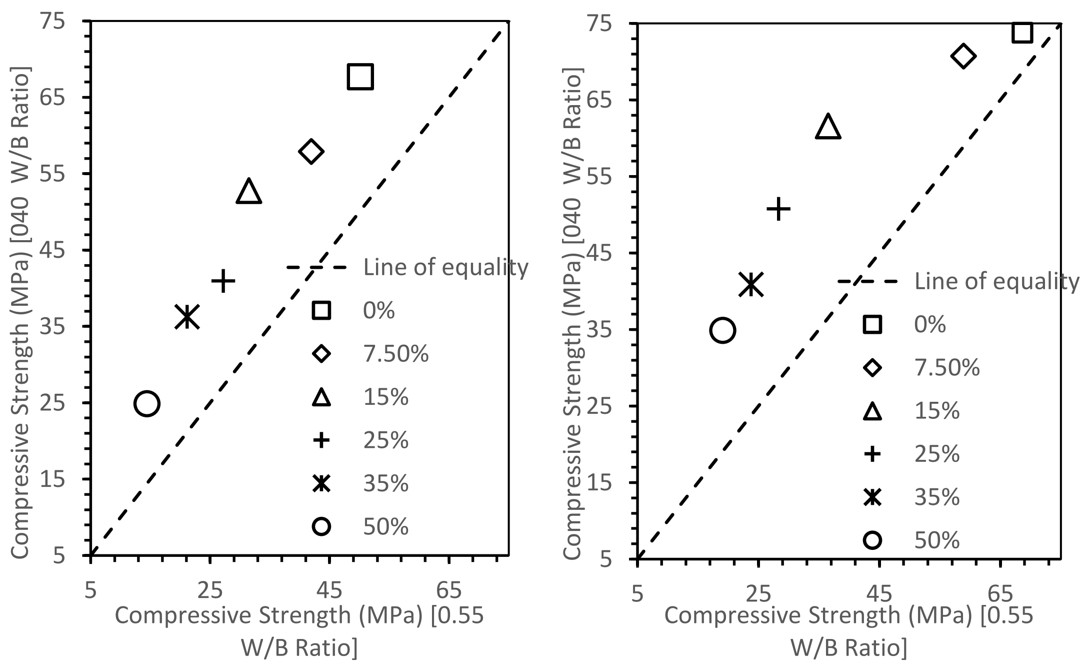


Compressive Strength (MPa) [0.55 w/b ratio]
Line of equality
0%
7.50%
15%
25%
35%
50%

4.9 (i)GGBS binder concrete

4.9 (ii) FA binder concrete

Figure 4. 9: Effect of w/b ratio on compressive strength at 28 day for different binder types



Compressive Strength (MPa) [040 W/B Ratio]
Compressive Strength (MPa) [0.55 W/B Ratio]
Line of equality
0%
7.50%
15%
25%
35%
50%

4.10 (i) GGBS binder concrete

4.10 (ii)FA binder concrete

Figure 4. 10: Effect of w/b ratio on compressive strength at 56 day for different binder types

4.3.2 Effect of rubber content on strength

Figures 4.11 to 4.13 depicts the effect of rubber content on compressive strength of concrete mix at 7, 28 and 56 days for two binder type and two water binder ratios respectively.

It is observed that as the percentage of rubber increases compressive strength decreases. Rubber has a lower stiffness compared to natural aggregates and the presence of rubber particles in concrete reduces concrete mass stiffness and lowers its load bearing capacity. At higher w/b ratio, compressive strength further decreases. As w/b ratio increases, the larger binder inter-particle spacing, leads to the need of hydration products to extend further to interact with one another resulting in a less dense hydrated cement microstructure (Mukadam, 2014; Preez, 2019). Therefore, with an increase in w/b ratio and rubber content, compressive strength decreases.

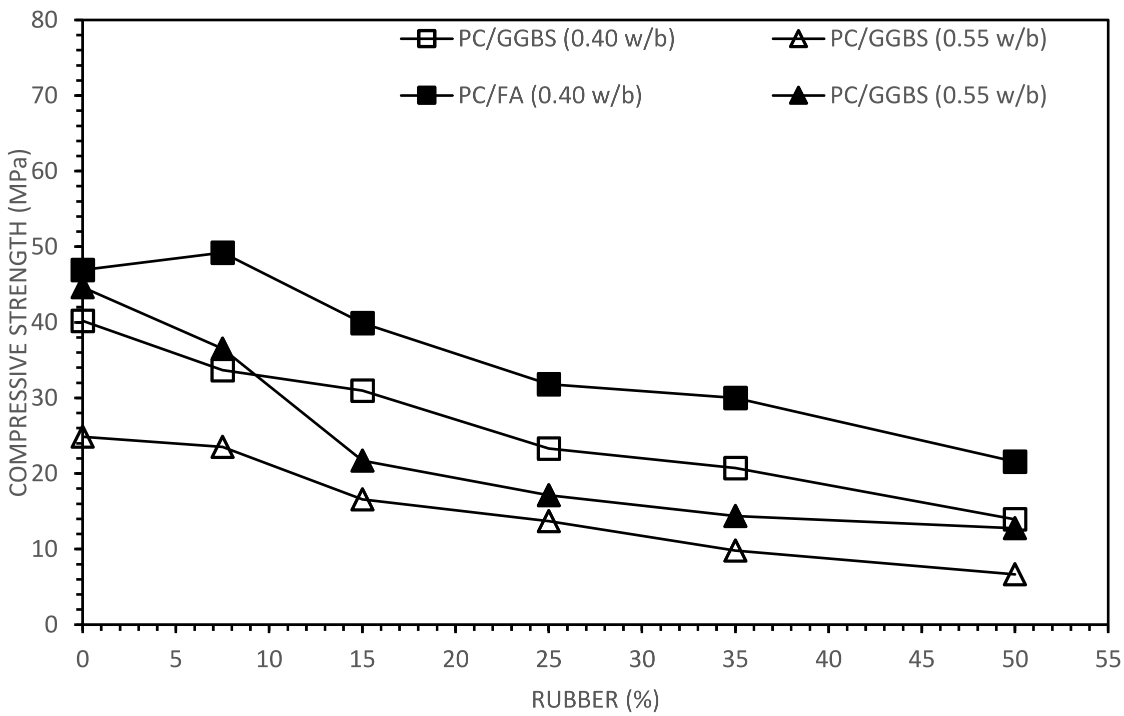


Figure 4. 11: Effect of rubber content on compressive strength of concrete at 7 day

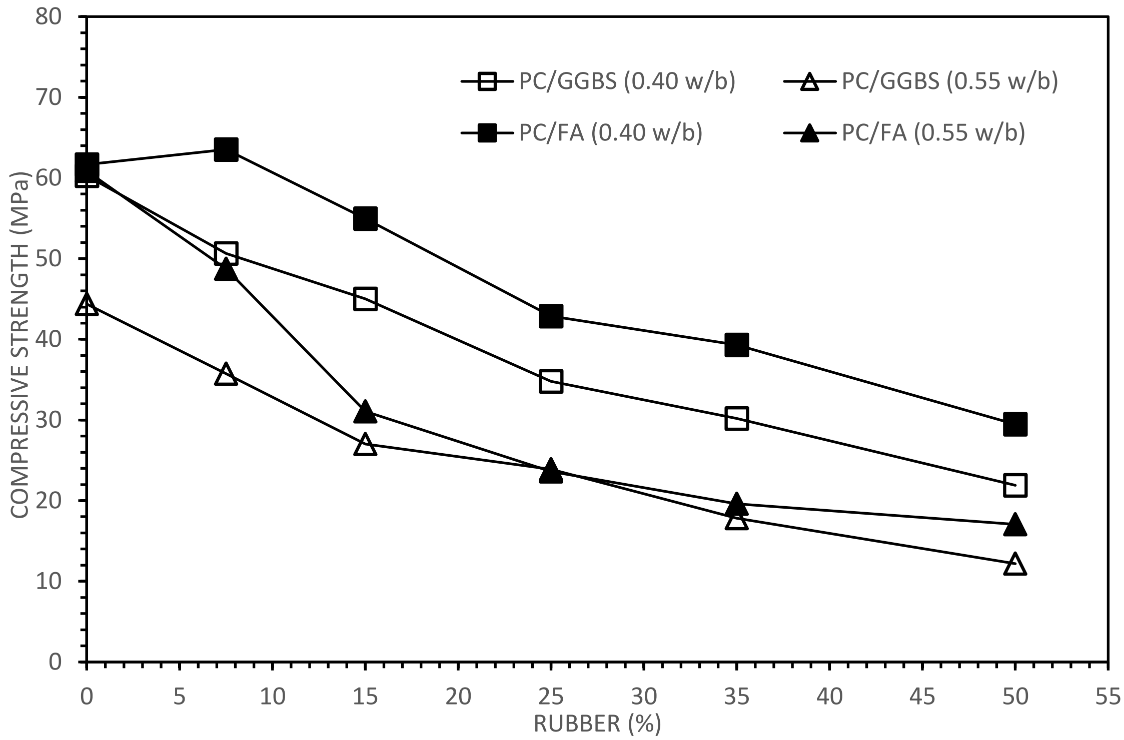


Figure 4. 12: Effect of rubber content on compressive strength of concrete at 28 day

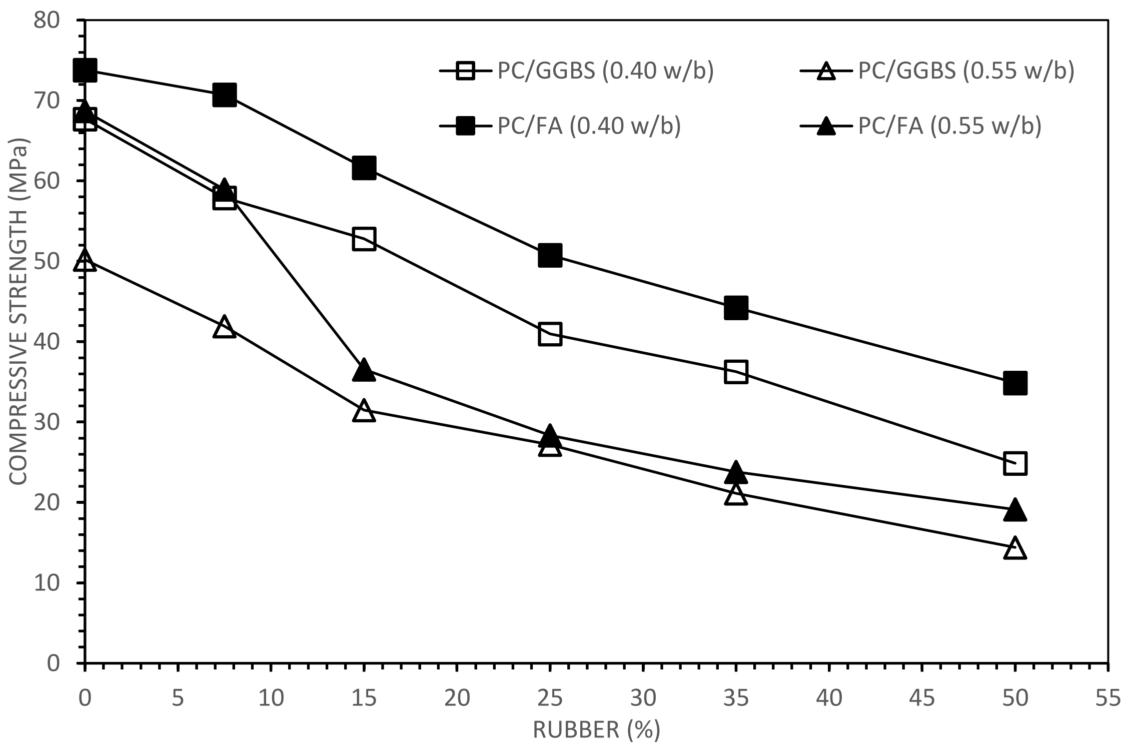


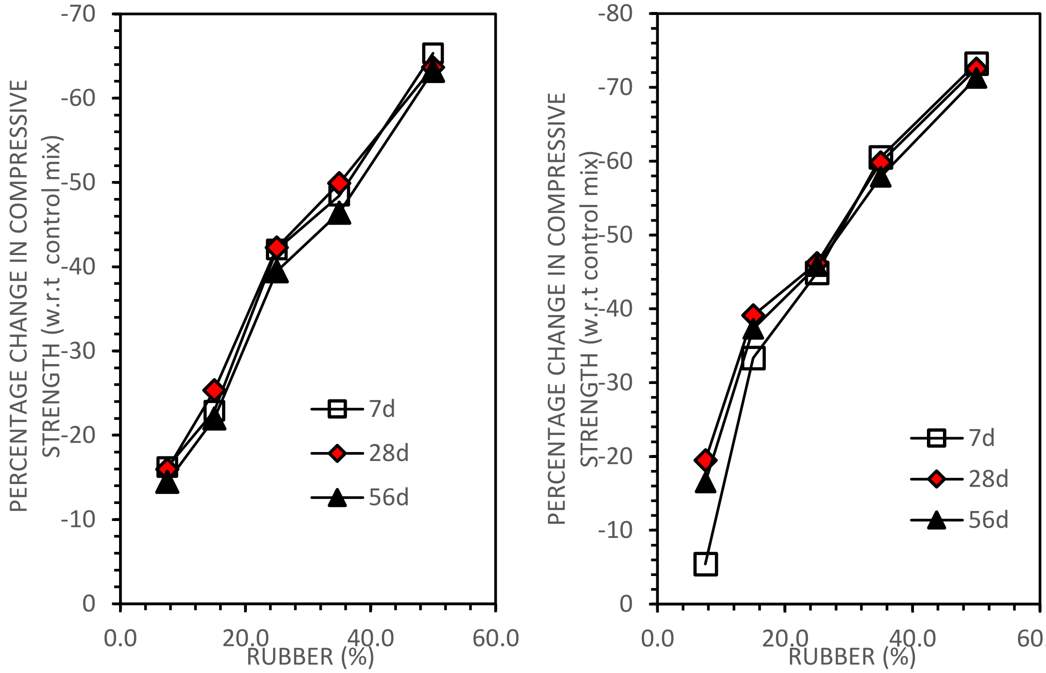
Figure 4. 13: Effect of rubber content on compressive strength of concrete at 56 day

Irrespective of binder type and w/b ratio, compressive strength decreases with increase in rubber content. However, rubberized concrete mix with FA binder type shows better strength when compared to GGBS binder type rubberized concrete mix design. The incorporation of fly ash improves the performance, because of its unique spherical shape, and chemical properties. The spherical shape of aluminum and silica compounds which form the majority of fly ash components, are used for fillers in rubber products (Rycroft, 2017).

Compressive strength decreased as the rubber concentration increased, especially because the waste rubber displays little adhesion to the cement paste (Pczieczek *et al.*, 2019). However, the addition of fly ash in concrete increases compressive strength compared to GGBS binder mix, this is due to increased formation of hydrated calcium silicate (C-S-H).

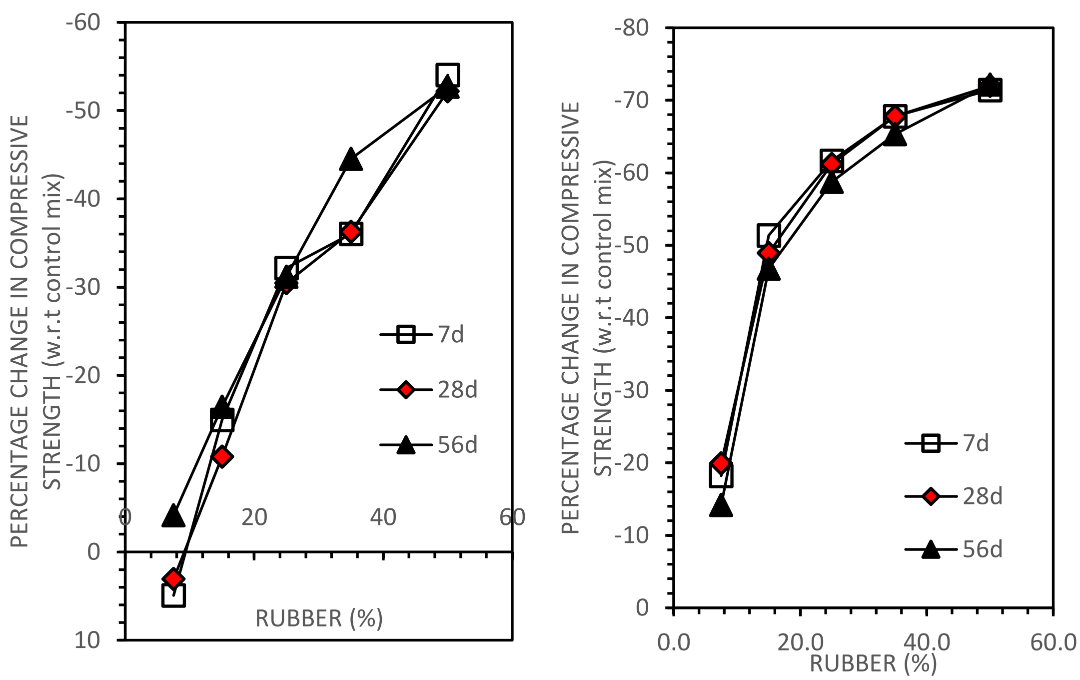
Compressive strength of mix concrete reduced significantly with the increase in rubber content. The strength of concrete depends on the bonding between cement and aggregates, size and hardness of aggregates (Eldin and Senouci, 1993) (Jamdar and Ansari, 2018). As a result of partially replacing fine aggregates with shredded rubber particles, the bonding between cement and aggregates weakened and hence strength reduced. The results obtained from the compressive strength agrees with the existing works reported by Eldin and Senouci (1993) and Jamdar and Ansari (2018).

Figures 4.14 (i) and (ii) and 4.15 (i) and (ii) quantifies the percentage reduction in compressive strength values with the introduction of rubber content with respect to control mix at lower and higher w/b ratio of 0.40 and 0.55 in GGBS and FA binder type rubberized concrete.



(i)0.40 w/b ratio *(ii) 0.55 w/b ratio*

Figure 4. 14: Percentage change in compressive strength of GGBS concrete with respect to control mix



(i)0.40 w/b ratio *(ii) 0.55 w/b ratio*

Figure 4. 15: Percentage change in compressive strength of FA concrete with respect to control mix

4.3.4 Practical implications for rubberized concrete on the compressive strength of non-structural concrete

It is observed that at lower w/b ratio of 0.40, concrete specimens containing up to 50% replacement of fine aggregates with shredded tyre rubber particles, attained 2MPa above minimum characteristic strength of 20MPa for non-structural plain and reinforced concrete works at 28-d curing, with both GGBS and FA binder type. It is observed that with an increase in w/b ratio to 0.55, the strength requirement for both non-structural plain and reinforced concrete works was attained with the maximum of only up to 25% replacement percentage of fine aggregate with tyre rubber particles in both GGBS and FA binder type concrete. However, with prolonged curing, FA binder type surpassed the requirement for reinforced non-structural concrete with 35% rubber content. GGBS and FA binder type rubberized concrete attained the target strength requirement for plain concrete non-structural works with 35% and 50% rubber content respectively. It should be noted that, at w/b ratio of 0.55, FA binder type rubberized concrete attained the requirement for reinforced non-structural work, only with prolonged curing to 56-day. Therefore, from a practical view, it is very crucial that depending on the type of non-structural construction work, i.e., whether in plain or reinforced concrete be identified and accordingly the w/b ratio and rubber content selected with respect to binder type used. Selection of binder type and curing period is important, as they have different effects with same rubber content and w/b ratio. FA binder type rubberized concrete exhibits better strength properties when compared to GGBS binder type at both w/b ratio with increasing rubber content

In the past the gain in strength beyond the age of 28 days was regarded merely as contributing to an increase in the factor of safety of the structure; but the 1975 Code of Practice for Reinforced Concrete in Buildings (CP114) allows the gain in strength to be taken into account in the design of structures, provided it will not be subjected to load until a later age (Neville, 2008b). Hence at prolonged curing, FA concrete mixes at higher w/b ratio of 0.55, with a maximum of 35% rubber content may be suitable for non-structural reinforced concrete wall works such as mullions, spandrels and sidewalls, rubber content must be limited to 25% in GGBS. However, at lower w/b ratio of 0.40, both GGBS and FA concrete with 50% rubber content may be used. FA concrete mixes with a maximum of 50% rubber content and GGBS concrete containing up to 35% rubber content at higher w/b ratio of 0.55 may be suitable for plain non-structural purposes such as lightweight concrete walls, building facades, and architectural units. However, at lower w/b ratio of 0.40, GGBS and FA concrete containing up to 50% rubber content maybe used. Fire hazards are of major concern and need to be thoroughly investigated before recommendations for practical implementation are drawn.

Literature review indicates better bonding of rubber particles with the surrounding matrix, if rubber particles possess a a rougher surface or if they are pretreated, which results in higher compressive strength. According to Eldin and Senouci (1993), concretes containing water washed rubber particles showed 16% higher compressive strength. Studies by Rostami (1993), showed an improvement in compressive strength by 57% when rubber aggregates were pretreated with carbon tetrachloride, a cleaning fluid used in industry as a degreasing agent (Siddique and Naik, 2004). Hence, pretreating the rubber particles would improve the strength of rubberized concrete, allowing a greater replacement percentage of rubber content, in non-structural concrete works.

4.4 Durability Index Tests

4.4.1 A revisit of the durability requirements for non-structural concrete

Durability relates to the concept of material performance, and a concrete durable in one environment may not be durable in another (Ballim, Alexander and Beushausen, 2009). The basis of DI approach is to enable the quantification of relevant parameters, through the measurement of transport-related properties of cover layer to yield a suitable concrete quality. The measurements characterize concrete quality through the use of durability indices, which are quantifiable parameters that are sensitive to material, processing and environmental factors, thus highlighting both material potential and construction quality (Alexander *et al.*, 2008). DI tests are sensitive to material, environmental and constructional factors which influences concrete durability. It is applied in quality control of site concrete, optimization of mix proportion of concrete, performance-based specifications and long-term performance predictions (Alexander, Mackechnie and Ballim, 1999). Hence, it is important durability indices be considered in concrete for non-structural construction purposes concrete.

Table 4.1 shows the suggested ranges for durability classification of concretes for the three index tests, based on site and laboratory data, i.e. performance-based specification.

Table 4. 1: Suggested ranges for durability classification using index values (Alexander, Mackechnie and Ballim, 1999)

Durability class	OPI (log scale)	Sorptivity (mm/hr$^{0.5}$)	Conductivity (mS/cm)
Excellent	>10	< 6	< 0.75
Good	9.5 – 10	6 – 10	0.75 – 1.50
Poor	9.0 – 9.5	10 – 15	1.50 – 2.50
Very poor	< 9.0	>15	>2.50

According to Alexander and Beushausen (2009), deemed-to-satisfy approach is considered an adequate approach to specifying durability index for most of the reinforced concrete construction. Tables 4.2 and 4.3, gives the deemed-to-satisfy values for SANRAL projects. There is no focus on binder type for OPI and WSI values. Chloride conductivity index is a quality control parameter used to assess the resistance of concrete to chloride ingress (Otieno and Alexander, 2015). Lower the index, better the potential durability of the concrete (Gouws, Alexander and Maritz, 2001). CCI values focus on binder type.

Table 4. 2: Durability parameters acceptance ranges (SANRAL project target-New England Road Bridge) (Angelucci, 2013) (Ronny, 2011)

Acceptance level	**Oxygen Permeability (Log Scale)**	**Water Sorptivity (mm/√*hr*)**
Concrete made, cured and tested in the laboratory (average of 2 tests)	> 9.80	OPI * 1.1 (max. 12)
Full acceptance of in-situ concrete	> 9.70	12
Conditional acceptance of in-situ concrete (with remedial measures approved by the Engineer)	8.75 – 9.70	12 - 15
Rejection	< 8.75	>15

Table 4. 3: Maximum chloride conductivity values (mS/cm) for different exposure classes and binder types: deemed to satisfy approach – common structures, cover = 50mm (Alexander et al., 2008) (Angelucci, 2013) (SANRAL)

EN 206 class	**Description**	**70:30 CEM I : Fly Ash**	**50:50 CEM I : GGBS**
XS 1	Exposed to airborne salt but not in direct contact with seawater or inland saline waters	3.00	3.50
XS 2a	Permanently submerged in sea (or saline water)	2.45	2.60
XS 2b	XS 2a + Exposed to abrasion	1.35	1.60
XS 3a	Tidal, splash and spray zones	1.35	1.60
XS 3b	XS3a + Exposed to abrasion	1.10	1.25

4.4.2 Oxygen permeability index

The oxygen permeability index gives the negative logarithm of coefficient of permeability (k). Low value of coefficient of permeability, implying high OPI values, indicates less permeable concrete, i.e., better quality of concrete. Whereas, high value of coefficient of permeability and low OPI values, indicates more permeable concrete.

4.4.2.1 Effect of w/b ratio and binder type on OPI

Figure 4.16 and 4.17 presents the average OPI results of GGBS and FA concrete mixes at 28 days and 56 days for two w/b ratios of 0.40 and 0.55, by replacing fine aggregates with 0, 7.5, 15, 25, 35 and 50% shredded tyre rubber particles by volume.

These results show that as w/b ratio increases, concretes display higher permeability which in turn results in low OPI values. This is explained by the excess water present in the hydrated cement paste, for higher w/b ratios, which later evaporates and leaves large, well connected capillary pores, resulting in high permeability in concrete when compared to lower w/b ratios (Mukadam, 2014). It is observed that FA binder type concrete shows higher OPI values at both w/b ratio as compared to GGBS binder type. This is due to the fact that the pozzolanic reaction of FA refines the microstructure of the hydrated cement paste by transforming larger pores into finer ones, increasing the matrix density (Preez, 2019). The slight variation in trend observed for FA binder type control mix at 0.40 w/b ratio as compared to GGBS binder type can be due to poor compaction and presence of impurities. The error bar plotted for compressive strength in previous section 4.3.2 shows a higher standard deviation, indicating variations in data, for FA concrete at 0.40 w/b ratio. The low values of GGBS concretes at all ages is due to the low Blaine fineness of the GGBS, as compared to FA specimens, which influences the increased rate of carbonation due to consumption of calcium hydroxide during pozzolanic reaction and resulting permeability (Divsholi, Lim, Darren Yang and Teng, 2014; Mukadam, 2014; Preez, 2019). This supports the findings of Mukadam (2014) and Preez (2019). Numerous studies have been carried out dealing with the durability traits imparted on concrete microstructure by the use of extenders

(Angelucci, 2013)(Antiohos, Maganari and Tsimas, 2005). GGBS reduces permeability through the cementing reaction that refines the pore systems. The slow-maturing nature of its hydration reaction makes it a lengthy process though.

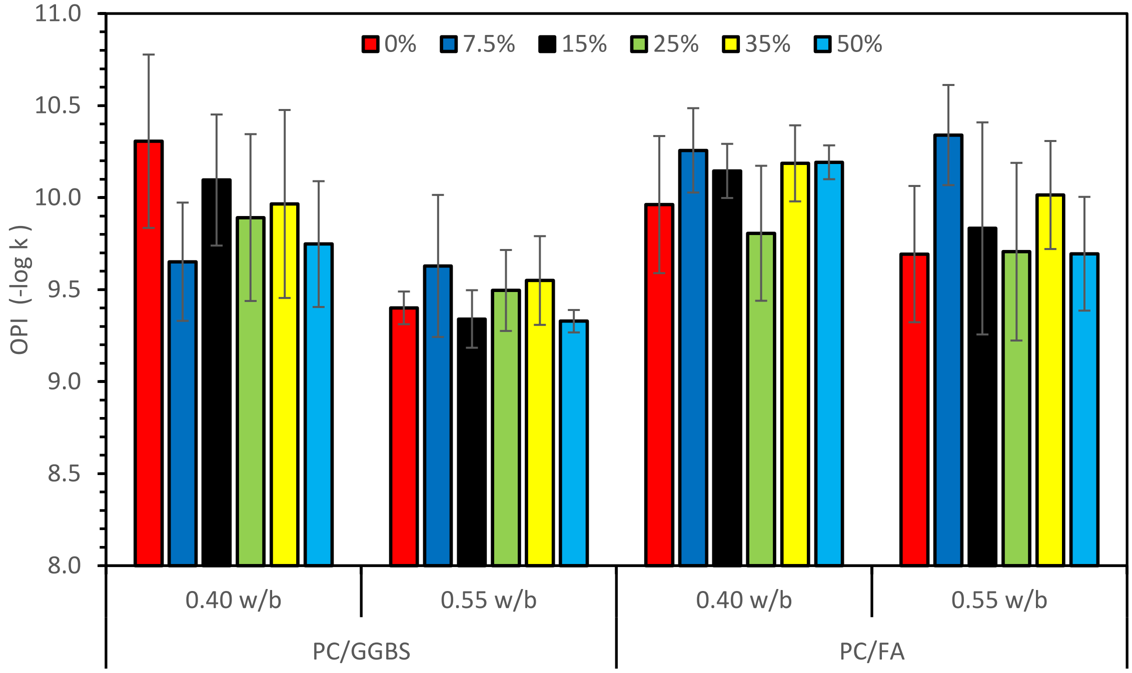


Figure 4. 16: Average OPI results at 28 day

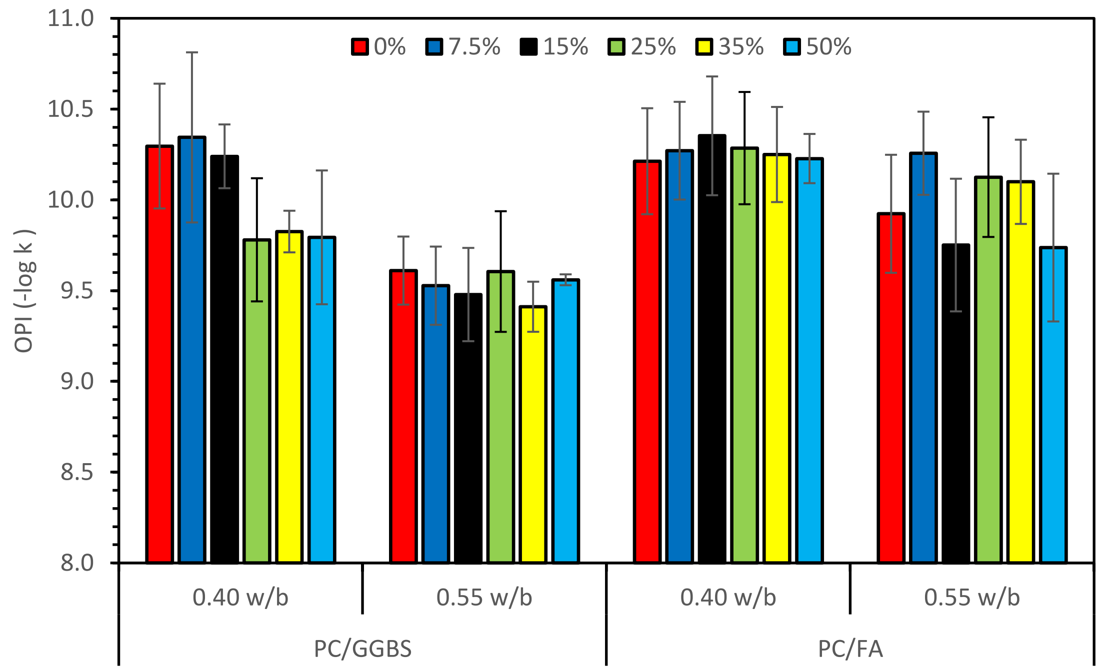


Figure 4. 17: Average OPI results at 56 day

Figure 4.18 and 4.19 illustrates effect of w/b ratio on oxygen permeability values for rubberized concretes, with binder types GGBS and FA, at varying rubber content. Data points lies above the line of equality, indicating that higher OPI values were obtained for the lower water binder ratio of 0.40 as compared to higher w/b ratio of 0.55. At higher w/b ratios, the excess water present in the hydrated cement paste, would later evaporate leaving large and connected capillary pores, resulting in higher permeability as compared to lower w/b ratios (Preez, 2019).

Figures 4.20 and 4.21 gives the effect of binder type on OPI values at 28 and 56 days of curing, with respect to a change in rubber content from 0% to 50%. Permeability improvements were noted with increased curing age. As with compressive strength, permeability also improves with prolonged curing. FA concrete exhibits better permeability, which implies better quality of concrete. GGBS concrete exhibits lower OPI values due to the low Blaine fineness, as compared to FA concrete specimens, which influences the rate of reaction of the concrete and the resulting permeability (Preez, 2019).

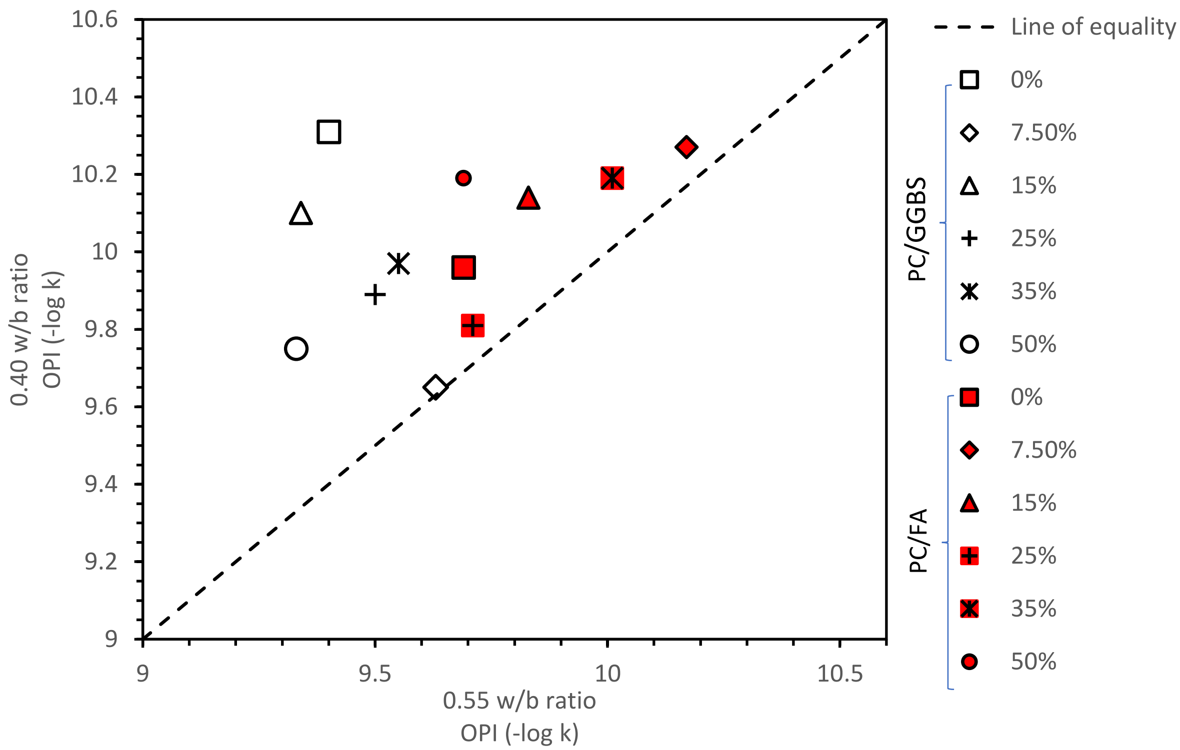


Figure 4. 18: Effect of w/b ratio on OPI at 28 day for varying rubber content

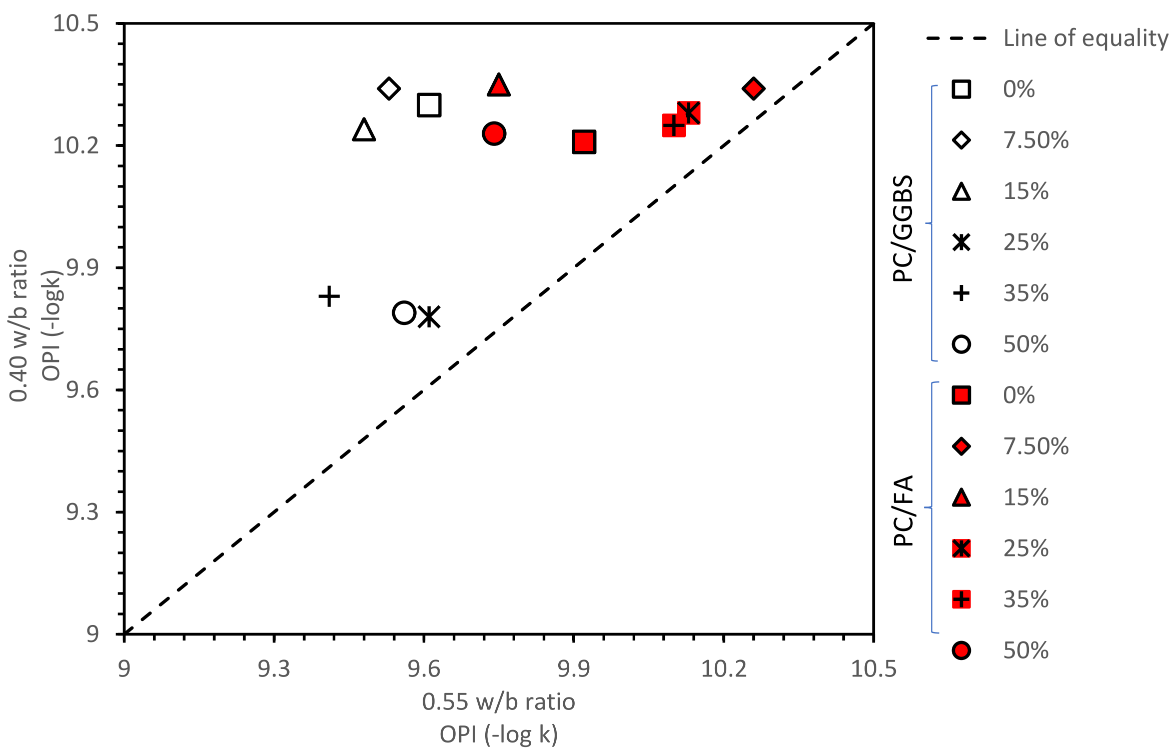


Figure 4. 19: Effect of w/b ratio on OPI at 56 day for varying rubber content

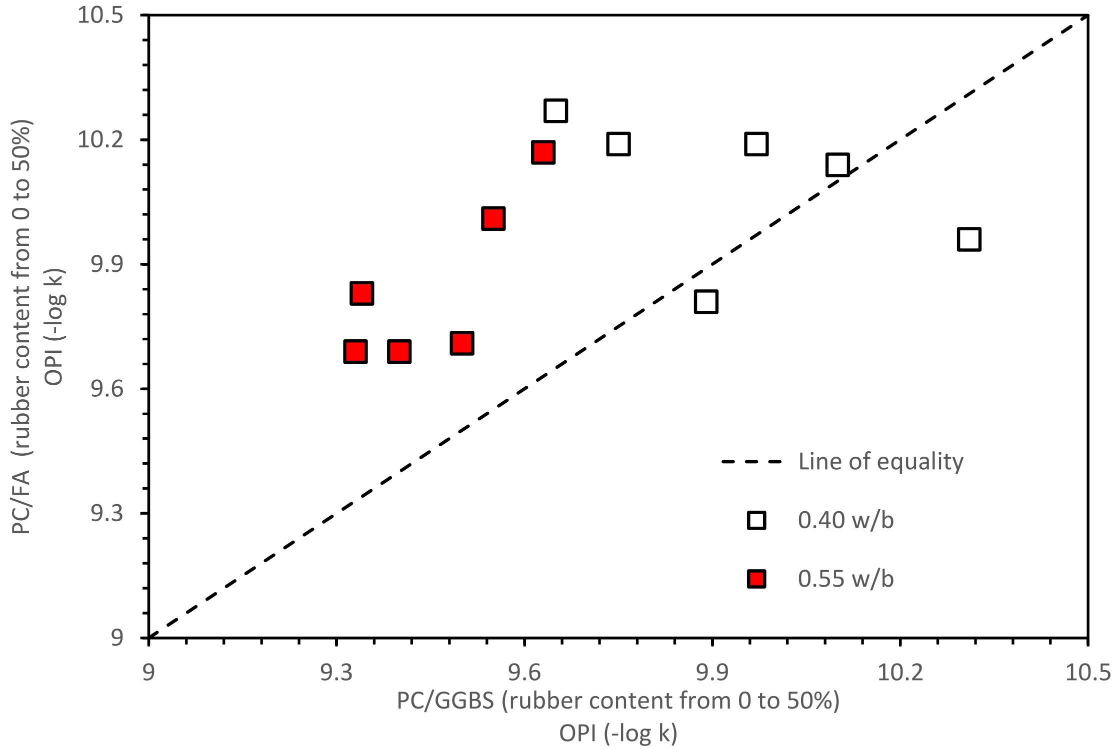


Figure 4. 20: Effect of binder type on OPI at 28 day

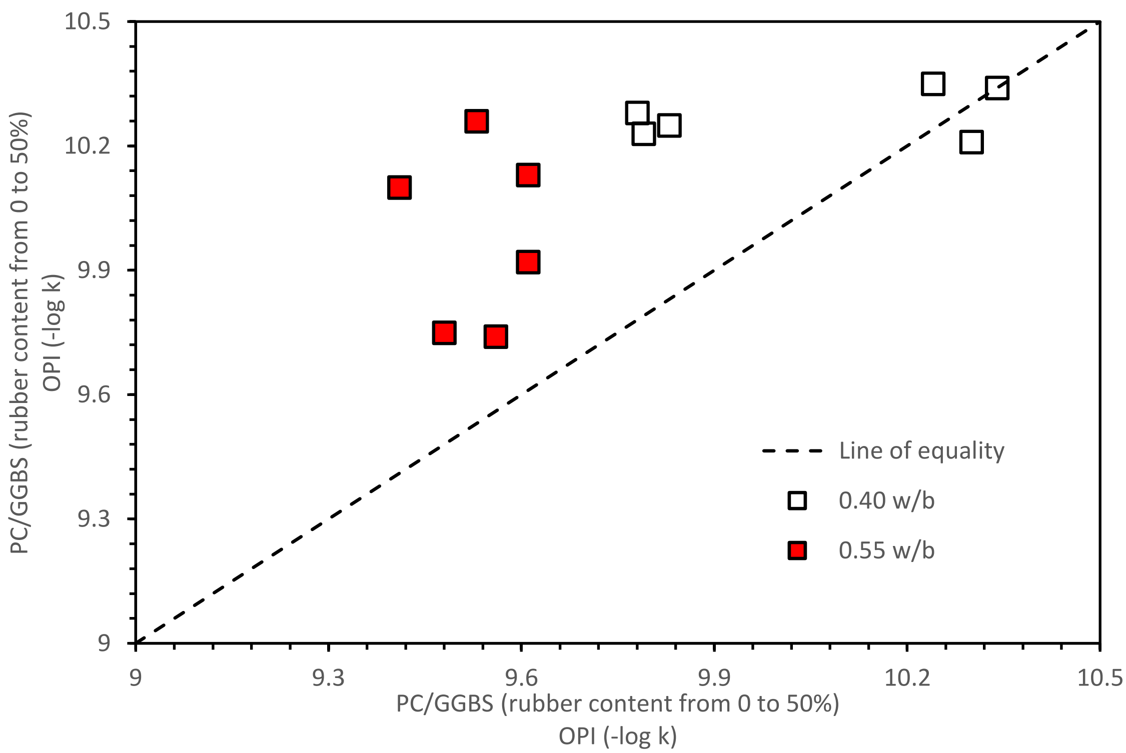


Figure 4. 21: Effect of binder type on OPI at 56day

4.4.2.2 Effect of rubber content on OPI

Figures 4.22 and 4.23 presents the effect of rubber content on OPI values, at 28 and 56 day, for two different binder types GGBS and FA and two w/b ratios of 0.40 and 0.55.

It is observed that addition of rubber content decreased the OPI value in GGBS binder type concrete with respect to the control mix at lower w/b ratio of 0.40 as compared to higher w/b ratio of 0.55, where OPI values showed more or less the same values with the introduction of rubber content. Permeability is a function of the internal structure of the rubber mixture. These results support the findings of Bjegovic (2011) conducted on gas permeability with addition of rubber particles in order to determine dependencies between permeability properties of concrete and presence of rubber particles in concrete, which showed no improvement in gas permeability. Presence of high amount of air voids on rubber/binder paste interface results in lack of proper bonding between rubber particles and surrounding matrix influencing on gas permeability results (Bjegović *et al.*, 2011). This could possibly be the reason for more or less little effect on addition of rubber content in GGBS concrete. On other hand, in FA concrete mixes, addition of rubber content had a positive impact on OPI values compared to control mix at both the lower and higher w/b ratio of 0.40 and 0.55. This can be reasoned as the result of spherical shape of fly ash particles in voids created by rubber particles. The incorporation of fly ash, because of its unique spherical shape of aluminum and silica compounds, the majority of fly ash components, are used for fillers of voids in rubber products (Rycroft, 2017). A slight variation observed in FA binder type rubberized concrete at 0.55 water binder ratio, for 25% and 50% rubber content can be reasoned as presence of impurities and lack of proper compaction. Also, rubber has low permeability towards non-polar gases such as oxygen, due to the presence of polar groups in the rubber chain (Chandrasekaran, 2010).

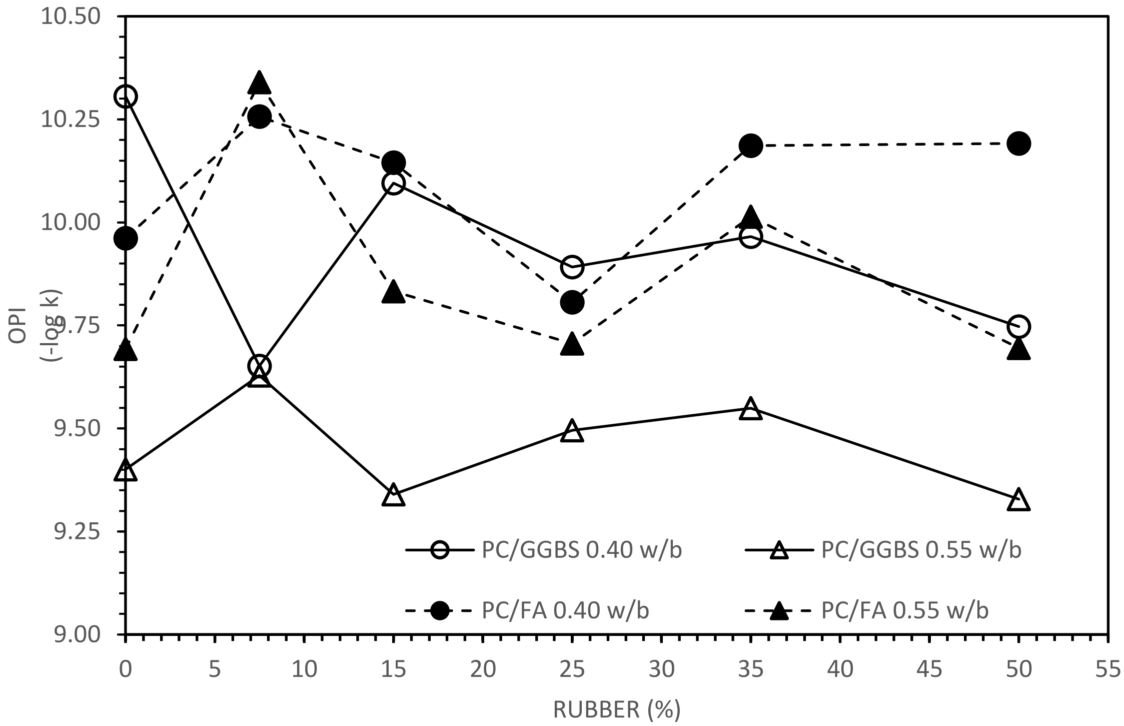


Figure 4. 22: Effect of rubber content on OPI at 28day

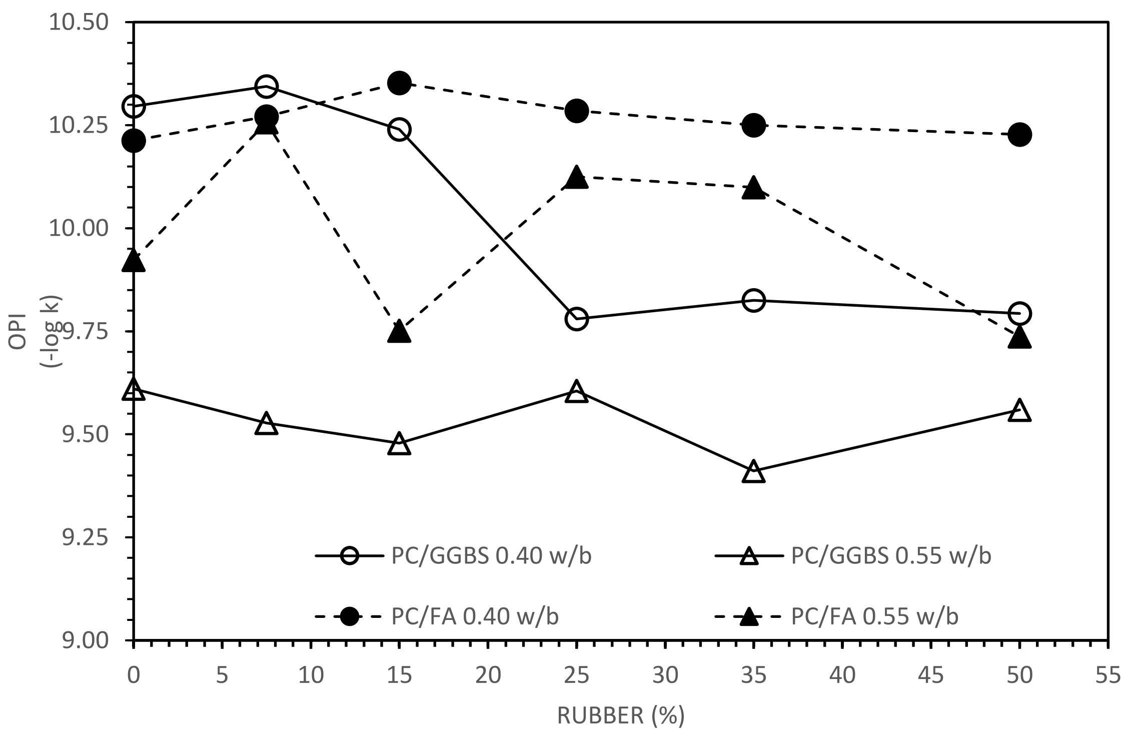


Figure 4. 23: Effect of rubber content on OPI at 56day

4.4.3 Water Sorptivity Index

Water sorptivity test ensures uni-directional absorption of water into preconditioned concrete specimens. The test is found sensitive to the nature and extent of early curing of concrete (Ballim, Alexander and Beushausen, 2009). Sorptivity is defined as the rate of movement of a wetting front through a porous material under the action of capillary forces. This is influenced by the pore geometry including the orientation and connectivity (Preez, 2019). Lower the sorptivity value, better is the quality of concrete as it indicates more resistance towards the ingress of moisture.

4.4.3.1 Effect of w/b ratio and binder type on WSI

Figures 4.24 and 4.25 presents the average WSI results of concrete mixes at 28 and 56 days, for two different binder types GGBS and FA, at two different w/b ratios of 0.40 and 0.50, by replacing fine aggregates with 0, 7.5, 15, 25, 35 and 50% shredded tyre rubber particles by volume.

On evaluating the influence of w/b ratio on sorptivity, it can be noted that in GGBS control mix, increasing w/b ratio shows an increase in water sorptivity value. This supports the results from the studies made by Angelucci (2013). According to the work of Angelucci, water sorptivity was negatively affected by increase in w/b ratio for GGBS concretes, and for FA concrete the trends were unclear (Angelucci, 2013). The increase in sorptivity values for increasing w/b ratio, can be attributed to the increased pore refinement as the w/b ratio decreases (Preez, 2019). However, it can be observed that in GGBS control concrete mix and in mixes with rubber content up to 7.5%, sorptivity showed better values at lower w/b ratio. However, at higher w/b ratio, this was not observed. Slight variations are observed in this trend, and this is reasoned as a result of improper compaction and contaminations present.

Figures 4.26 to 4.27 illustrates the effect of w/b ratio on WSI at 28 and 56-day, for GGBS and FA binder type concrete with a varying rubber percentage from 0% to 50%. It is observed from the WSI values that GGBS binder type concrete provide better sorptivity compared to FA binder type. And this is supported by the findings of Ballim, Alexander and Heushausen (2009), of typical test results for water sorptivity on water cured samples in relation to w/b ratio and binder type (Ballim, Alexander and Beushausen, 2009). There is not much significant change in sorptivity value at 28 and 56-day curing.

Irrespective of w/b ratio, it can be noticed that GGBS binder type produced better control mix and rubberized concrete, as compared to FA binder type concrete.

Figures 4.28 and 4.29 presents the effect of binder type on WSI when concrete specimens with varying rubber content from 0% to 50% in mixes are prepared for two different w/b ratio of 0.40 and 0.55 and subjected to a curing of 28 and 56 days. The data points that fall below on the line of equality indicates that higher WSI values are obtained for GGBS binder type concrete and for those data points which lie above the line of equality, higher WSI is obtained for FA binder type concrete. It can be observed that FA binder type concrete yields higher sorptivity results, compared to GGBS binder type. Hence a better durability quality is exhibited by GGBS binder type concrete. This can be reasoned as the result of pozzolanic reaction of FA, it refines the capillary pores, thereby lowering the sorptivity of the concrete. Furthermore, the fine filler effect of FA leads to an improved packing configuration of the particles in the hardened cement paste. This results in a denser microstructure, which reduces the overall concrete sorptivity (Preez, 2019).

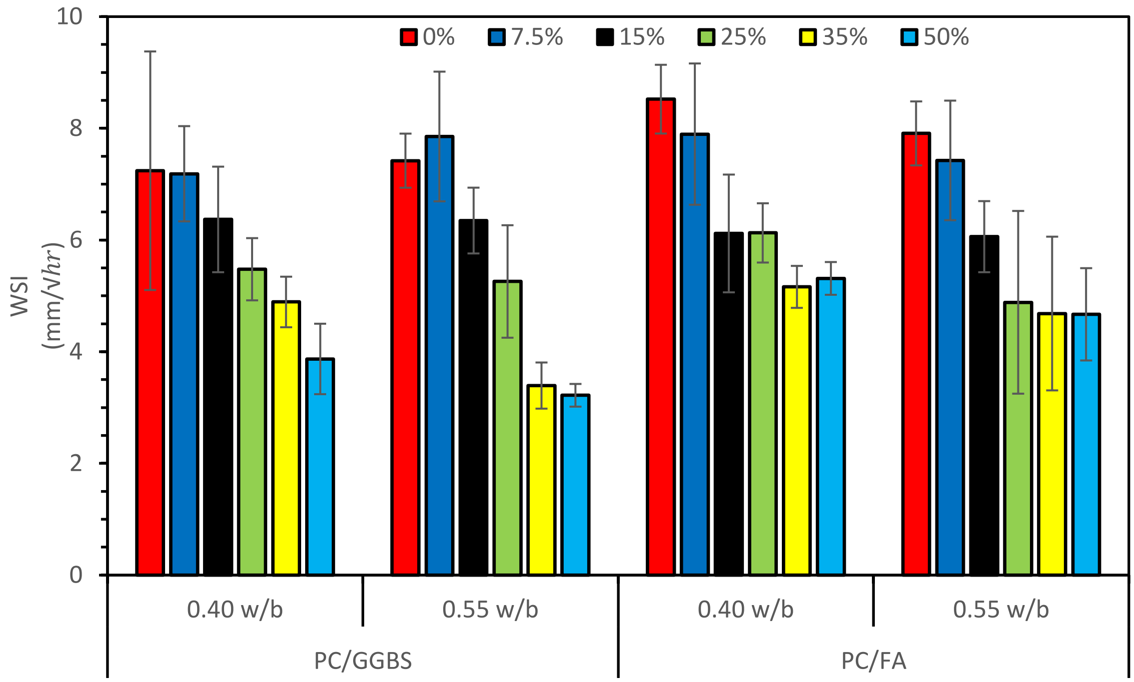


Figure 4. 24: Average WSI results at 28 d

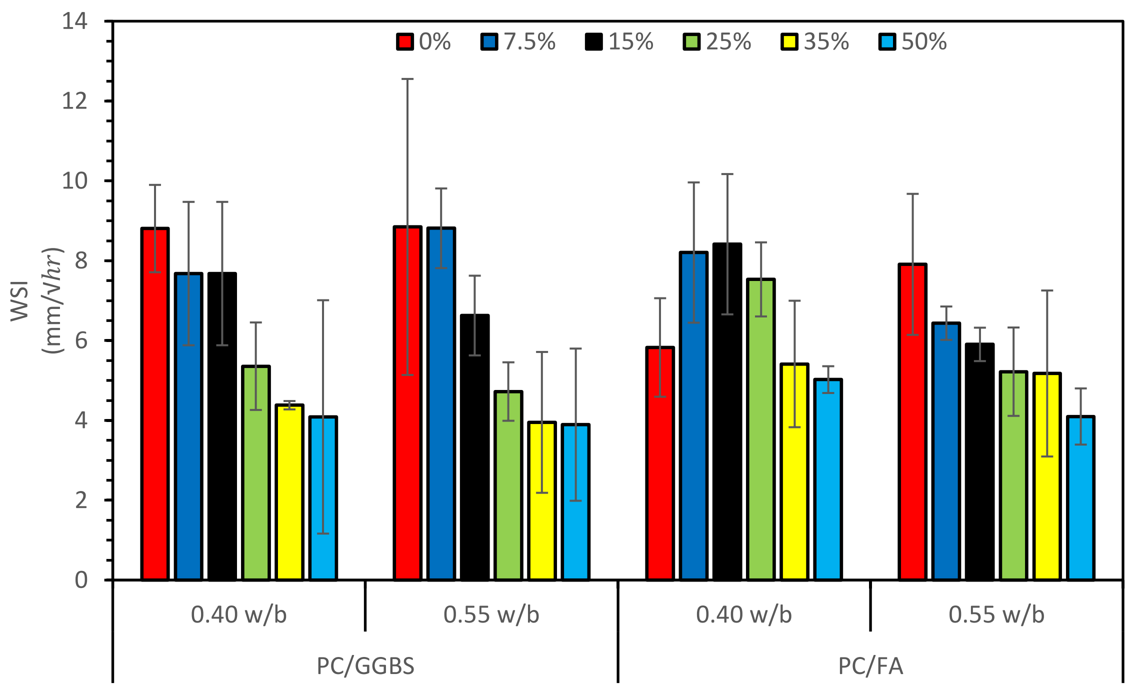


Figure 4. 25: Average WSI results at 56 d

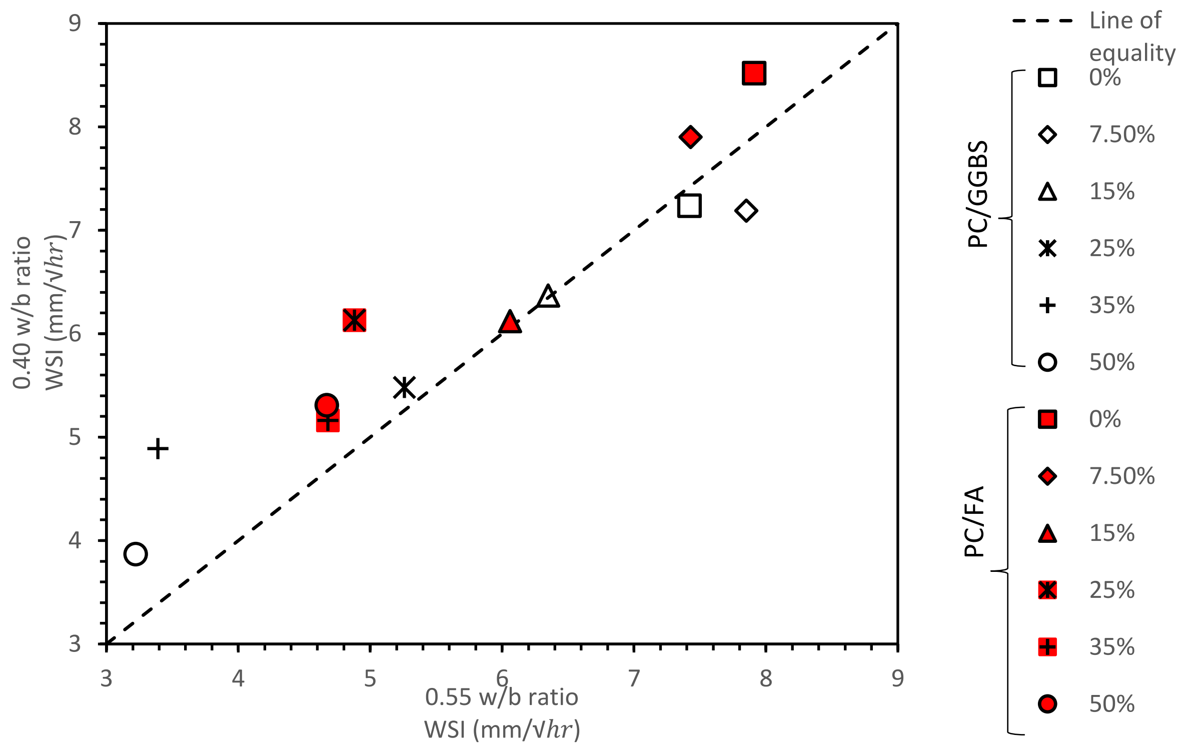


Figure 4. 26: Effect of w/b ratio on WSI at 28 day

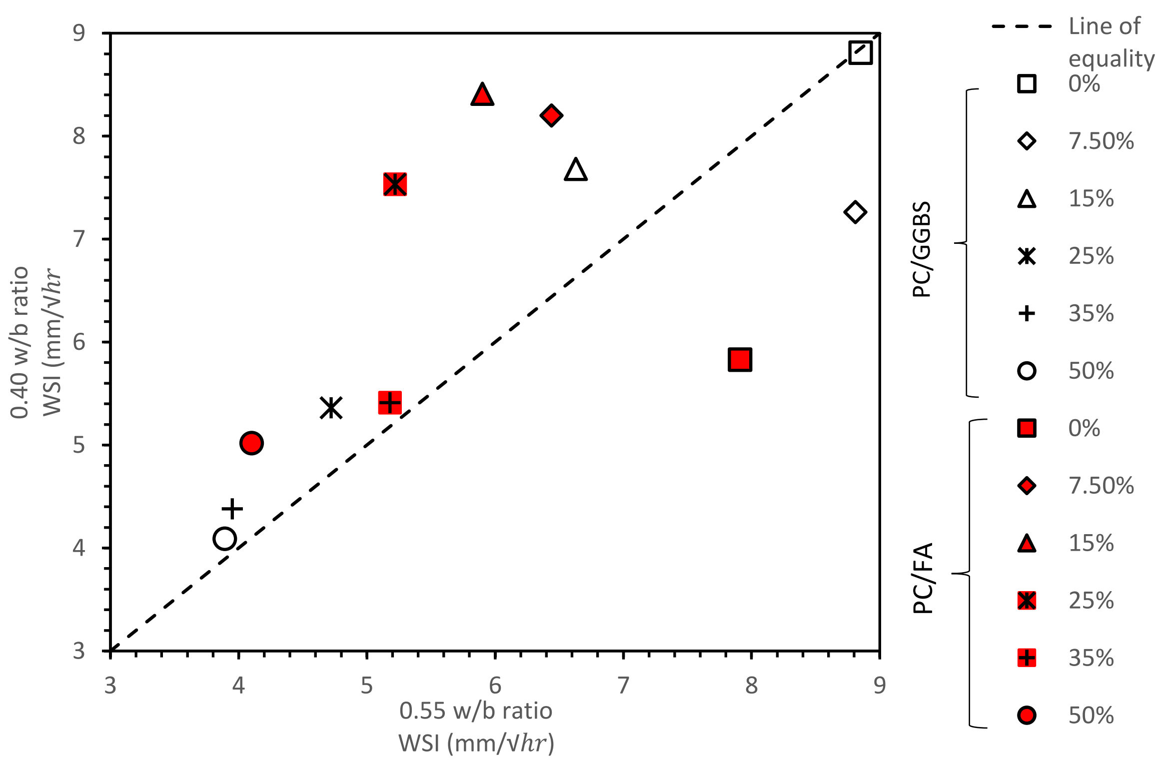


Figure 4. 27: Effect of w/b ratio on WSI at 56 d

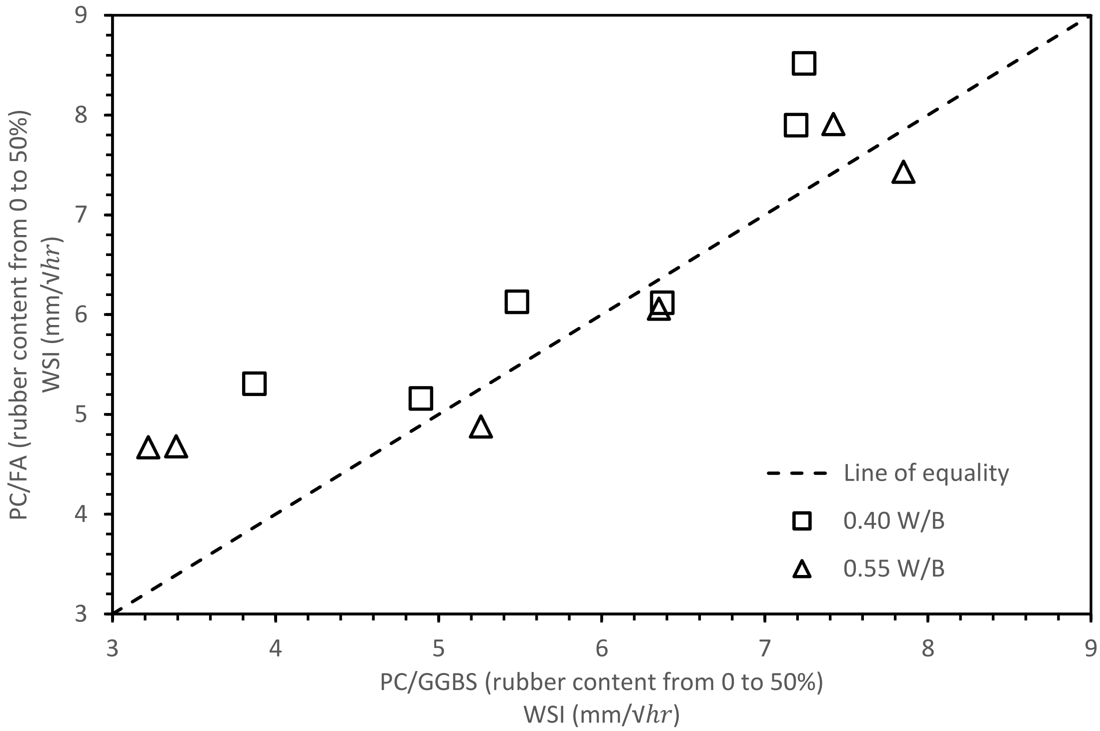


Figure 4. 28: Effect of binder type on WSI at 28 d

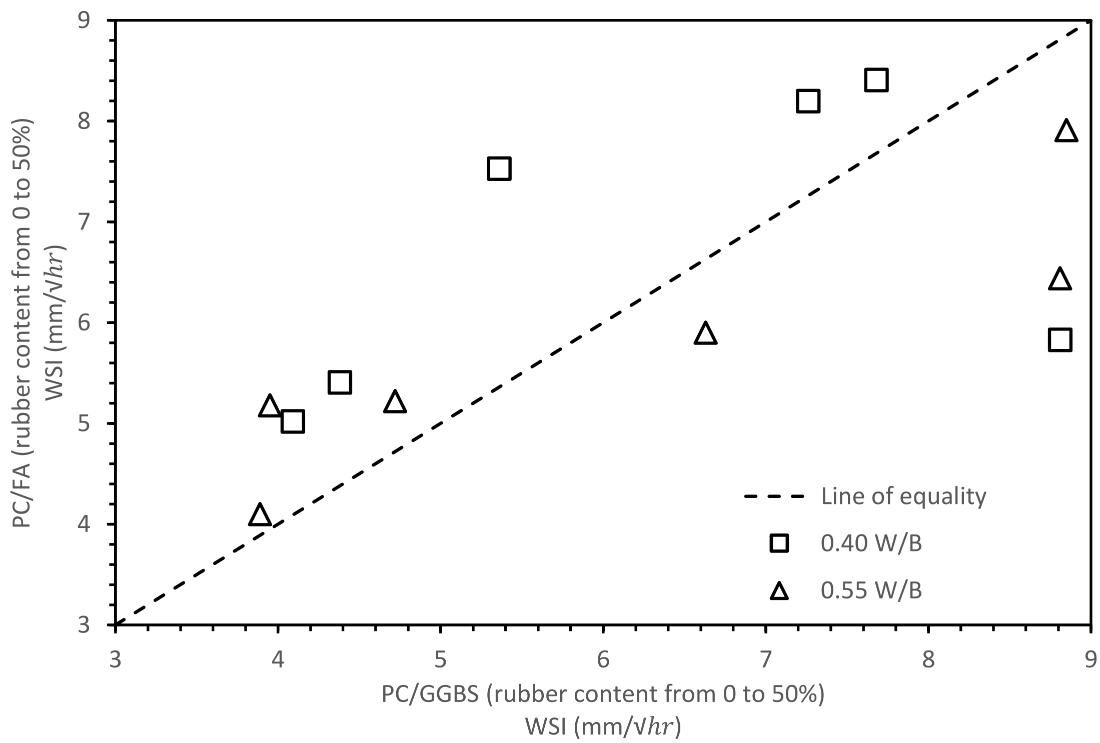


Figure 4. 29: Effect of binder type on WSI at 56 d

4.4.3.2 Effect of rubber content on WSI

Figure 4.30 and 4.31 presents the effect of rubber content on WSI of concrete at 28 and 56 day, for two binder types GGBS and FA at two different w/b ratios of 0.40 and 0.55 respectively.

It can be observed that on increasing the percentage of tyre rubber in concrete mix showed better sorptivity values. Non-polar nature of rubber particles results in entrapping air in closed pores and repelling water causing reduction in capillary absorption. Hence, increase in rubber content causes decrease in water absorption (Bjegović *et al.*, 2011). GGBS binder type rubberized concrete showed better sorptivity values than control mix from 7.5 to 50% tyre rubber content preplacement. GGBS rubberized concrete had lesser voids, due to the low Blaine fineness of GGBS (Sugapriya P and Ramkrishnan R, 2018). FA binder type rubberized concrete also showed similar trend, i.e. better sorptivity than control mix with increasing rubber content.

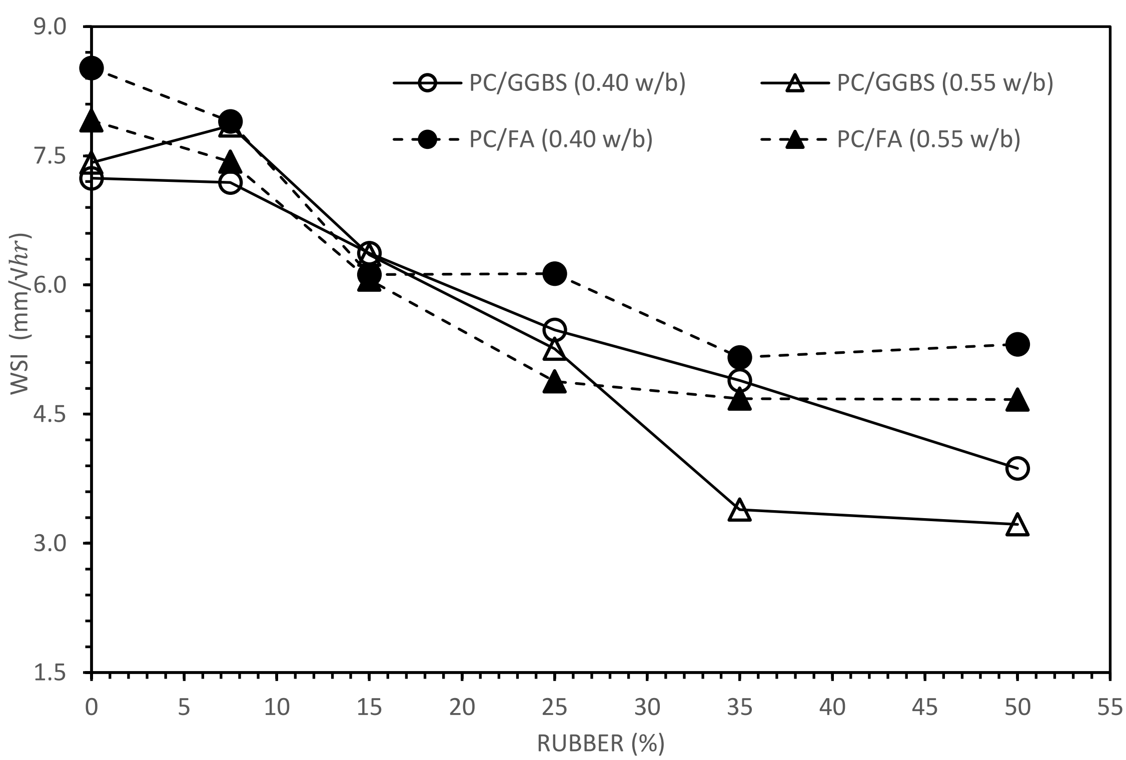


Figure 4. 30: Effect of rubber content on WSI at 28 day

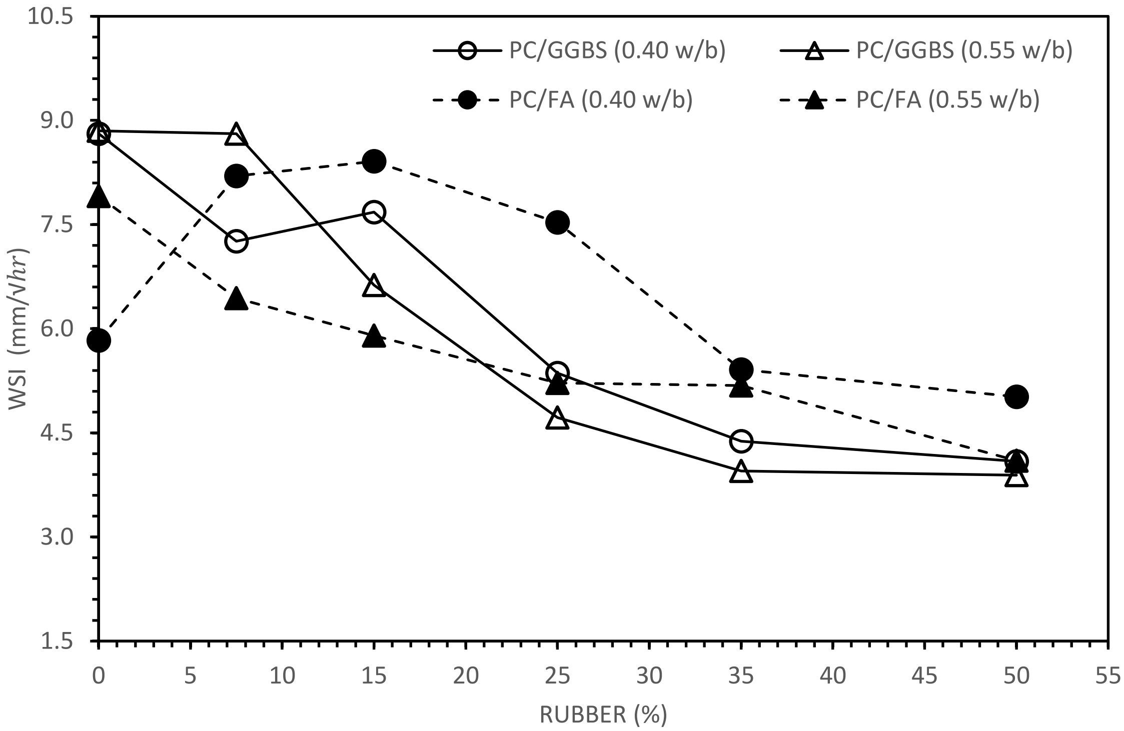


Figure 4. 31: Effect of rubber content on WSI at 56 day

Replacement of natural aggregate by rubber particles increases water permeability depth in concrete mixtures. Under high pressure, water penetrates concrete, which neutralizes the initial water repulsion by rubber. The inert rubber particles fill the voids of concrete reducing the permeability and increasing its resistance against acid attacks.

With increase in rubber content, FA binder type rubberized mix showed a better sorptivity value when compared to control mix. Sorptivity value shows a reduction in the range 7 to 40%, as rubber content increases from 7.5% to 50%, when compared to control mix with 0% rubber content. This shows a better quality of rubberized concrete when compared to conventional concrete. This supports the findings of (Noor, 2014), Eldin and Senouci (1994)

4.4.4 Chloride Conductivity Index

CCI measures the resistance of material to the ingress of chlorides by diffusion. A low value of CCI indicates the increased resistance to ionic ingress of a concrete specimen and high values of CCI indicates, less potential durability of concrete. Concrete containing fly ash and slag have been shown to have exceptional chloride binding characteristics and produce material of high chloride resistance (Mackechnie, 2001).

4.4.4.1 Effect of w/b ratio and binder type on CCI

Figures 4.32 and 4.33 outlines the average CCI results for the rubberized concrete specimens. The results show that CCI increases with increasing w/b ratio. It was observed that GGBS binder type concrete mix provides lower CCI values as compared to FA binder type concrete, irrespective of age of curing. Curing has a positive impact on the CCI. 56-day CCI results are lower than the 28-day results. This is because of the continued hydration process as a result of prolonged curing (Preez, 2019).

Higher w/b ratio resulted in higher CCI values irrespective of binder type. This is the result of capillary porosity of the concrete, with increase in w/b ratio (Malhotra, 1994).

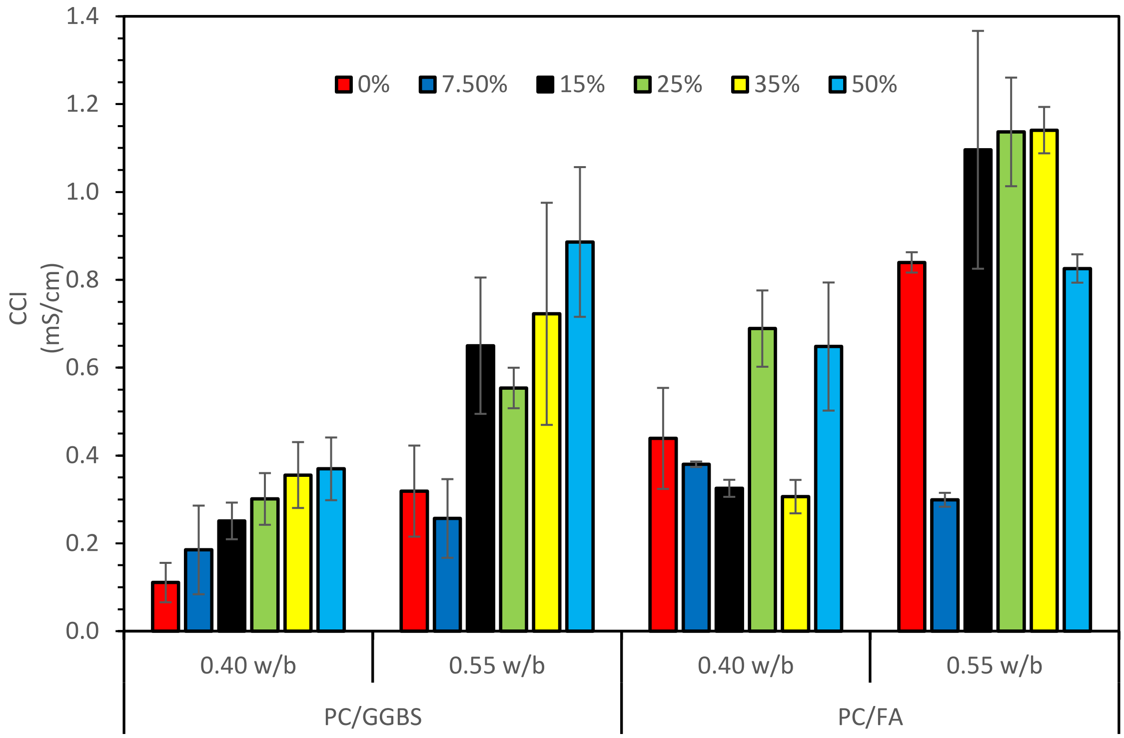


Figure 4. 32: Average CCI results at 28 day

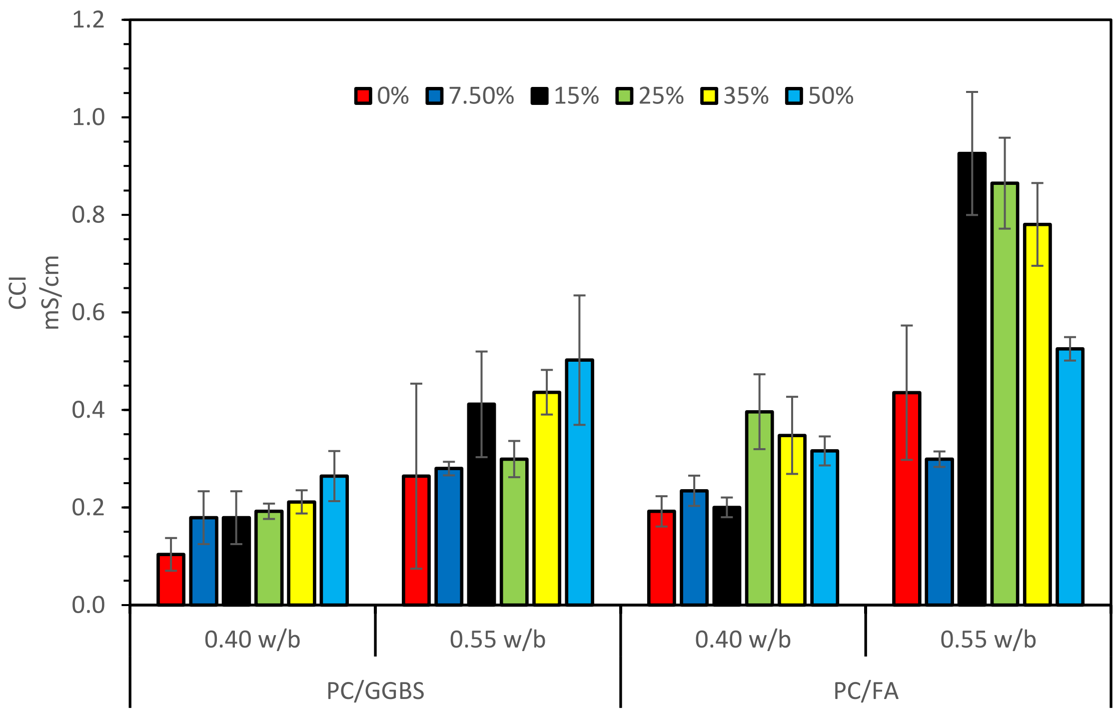


Figure 4. 33: Average CCI results at 56d ay

Figures 4.34 and 4.35 illustrates the effect of w/b ratio on CCI at 28 and 56-day, for GGBS and FA binder type concrete at varying rubber percentage from 0% to 50%.

Figures 4.36 and 4.37 presents the effect of two binder type GGBS and FA rubberized concrete on CCI value for two different w/b ratio of 0.40 and 0.55.

As stated earlier, GGBS binder type concrete yields lower conductivity results, compared to FA binder type. At 56-day curing, chloride conductivity decreased as compared to 28-day, irrespective of binder type. It was observed that irrespective of w/b ratio and age of curing, GGBS in rubberized concrete provided improved CCI values as against FA binder. The low CCI values in GGBS concretes is attributed to the chloride binding capabilities of the binder (Preez, 2019)(Neville, 2008a). According to Hadjsadok (2012), GGBS refines the pore structure of the hardened concrete matrix due to the formation of the secondary calcium silicate hydrate gel from the pozzolanic reaction of slag (Hadjsadok *et al.*, 2012). FA also had beneficial effect on the CCI values, however, this was less notable than in GGBS concretes. This is related to the behavior of fly ash towards the concrete pore refinement effect, which is lowering the rate of diffusion of substances into the concrete (Preez, 2019).

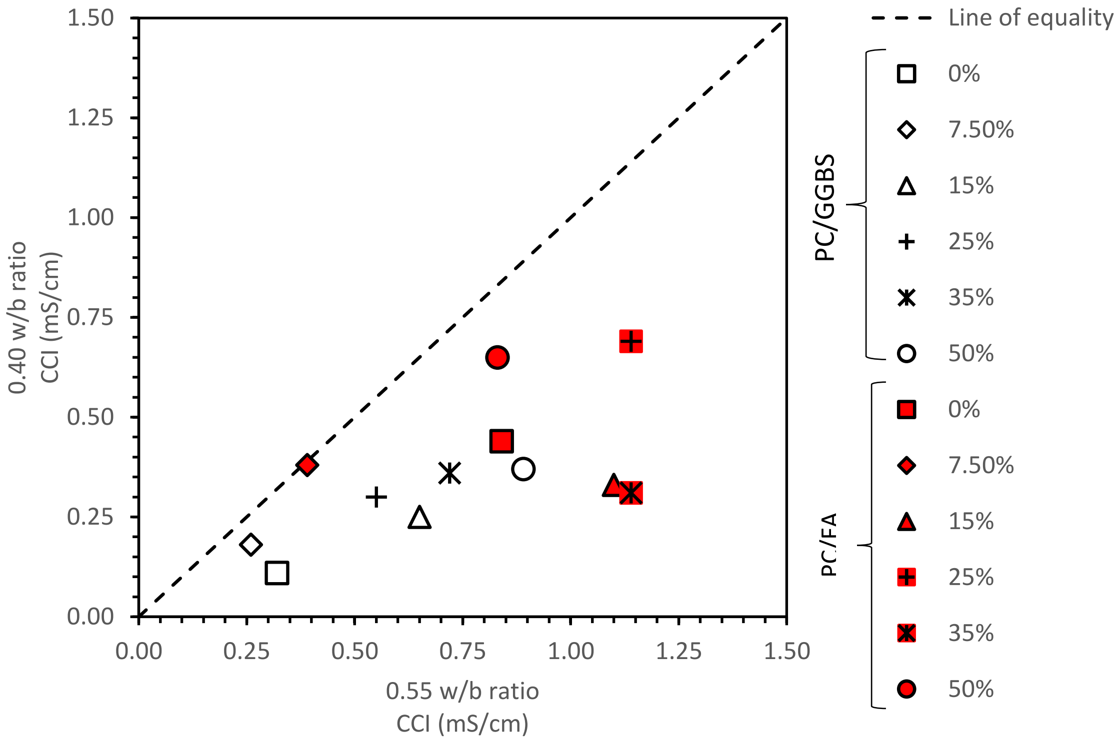


Figure 4. 34: Effect of w/b ratio on CCI at 28 d

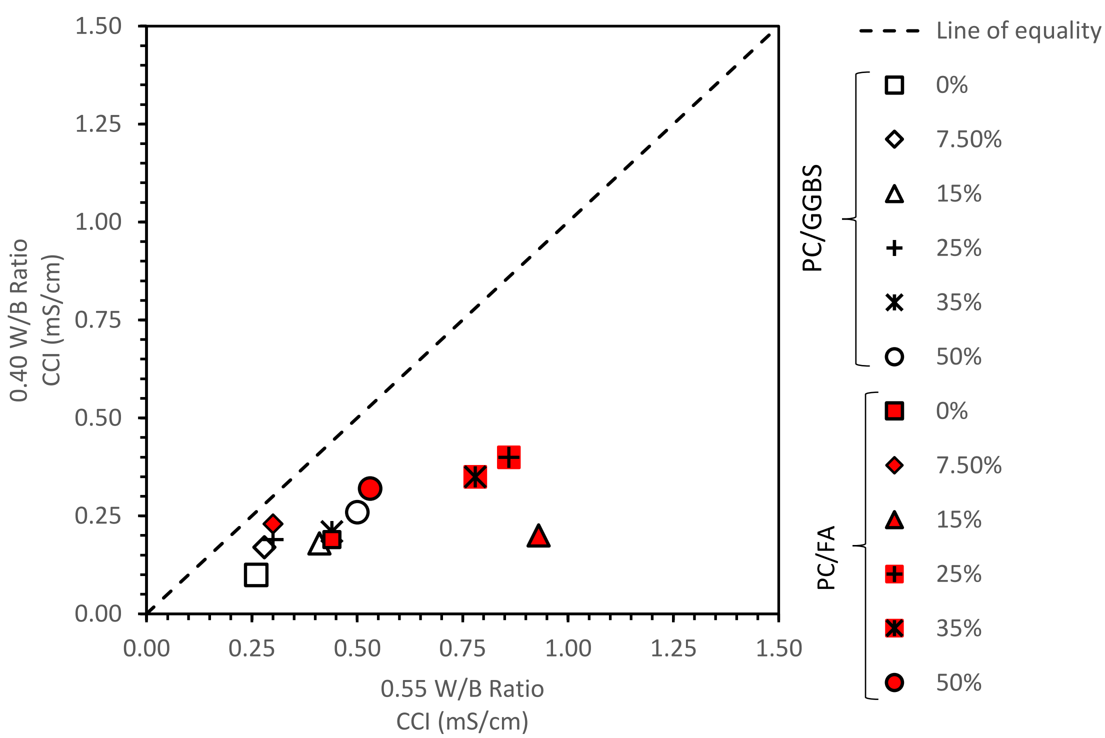


Figure 4. 35: Effect of w/b ratio on CCI at 56 d

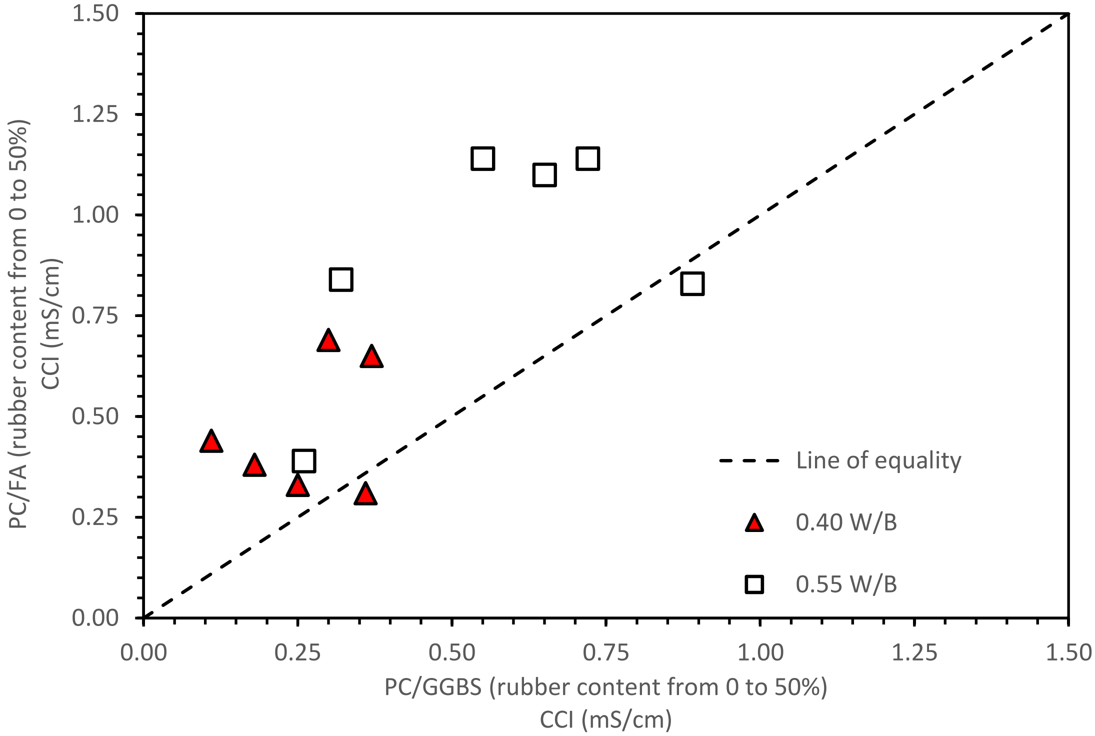


Figure 4. 36: Effect of binder type on CCI of rubberized concrete at 28 day

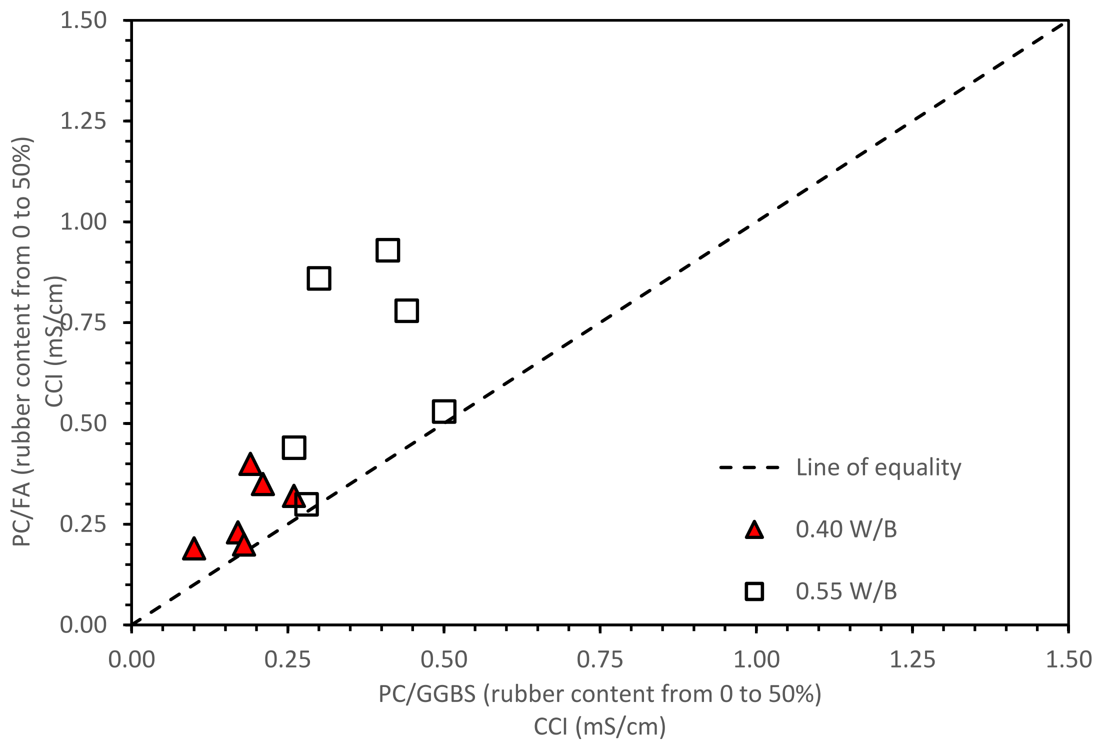


Figure 4. 37: Effect of binder type on CCI of rubberized concrete at 56 day

4.4.4.2 Effect of rubber content on CCI

Figure 4.38 and 4.39 illustrates the effect of rubber content on CCI values at 28 and 56-day, for binder types GGBS and FA, at two different w/b ratios for varying rubber percent replacing fine aggregates in concrete.

It is observed that, increasing rubber content increases the values of chloride conductivity in GGBS, however, these values decrease with prolonged curing. However, at higher w/b ratio, it can be observed that CCI value showed a decrease at 7.5% rubber content w.r.t control mix, and further started increasing with increase in rubber content. FA binder concrete also shows a similar trend, CCI shows decrease with 7.5% rubber content, and further increases. The CCI values shows a decrease with prolonged curing at all percentage of rubber content. These observations of showing better resistance to ingress of chloride ions for smaller rubber content and thereafter decreasing resistance is observed by Li et.al (2015). They concluded from their study that for both treated and untreated rubber aggregate mixes there was a decrease in chloride penetration up to a limiting replacement level, 5% and 15% for untreated and treated rubber mixes respectively (Li *et al.*, 2015). They reported poor durability performance at higher replacement levels with rubber content. Other authors like (Guneyisi *et al.*, 2014) have reported contrary results, i.e. continuous decrease in chloride penetration resistance with increasing rubber content.

Although there have been many studies related with the use of waste tire rubber in concrete, the knowledge regarding durability properties of rubberized concrete is still insufficient (Guneyisi *et al.*, 2014).

In GGBS binder type control mix the chloride conductivity increased from 0.11mS/cm to 0.37mS/cm at lower w/b ratio for 28-day curing, and from 0.1mS/cm to 0.26mS/cm 56-day curing. At higher w/b ratio, there is a slight drop in conductivity value at 7.5% and 25% rubber content as seen in figure 4.37.

In FA binder type control mix, the trend of chloride conductivity is unclear. At lower w/b ratio of 0.40, it shows a sharp increase in chloride conductivity value at 25% rubber content, below which they show better resistance to chloride ingress as compared to control mix. Beyond 25%, CCI values started decreasing at prolonged curing. At higher w/b ratio of 0.55 and prolonged curing of 56-days, it increased sharply at 15% rubber content, after a drop at 7.5%. Beyond 15% rubber content at prolonged curing, CCI showed a decreasing trend, as plotted in figure 4.38.

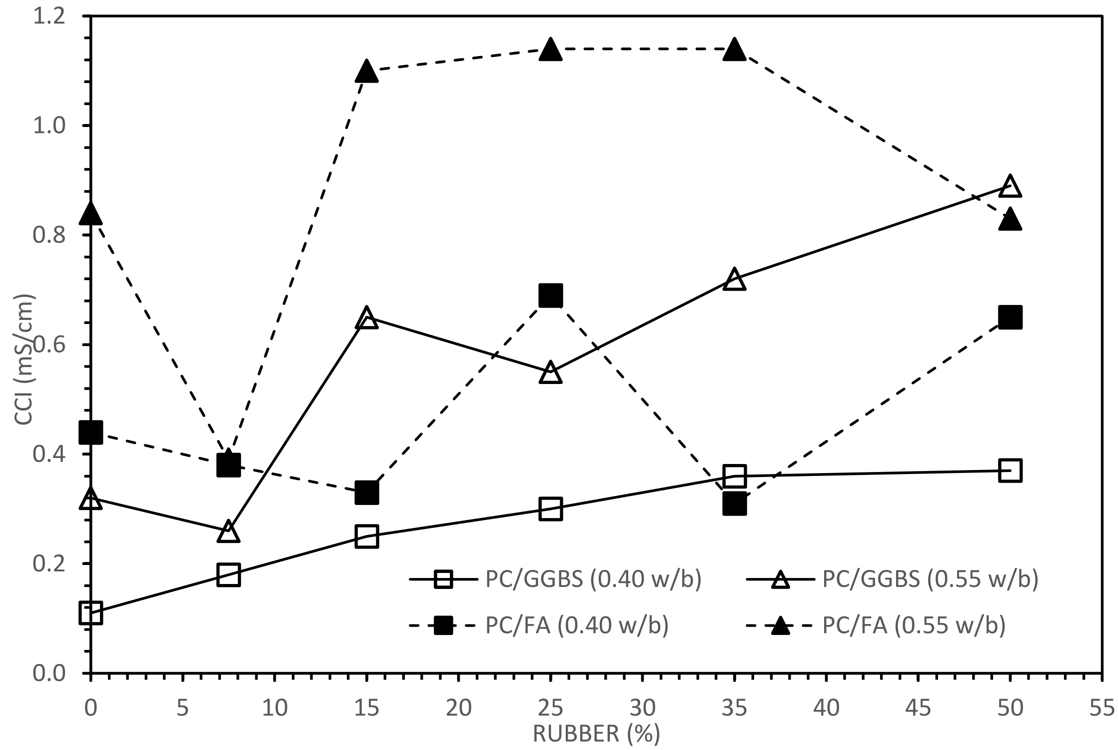


Figure 4. 38: Effect of rubber content on CCI at 28 day

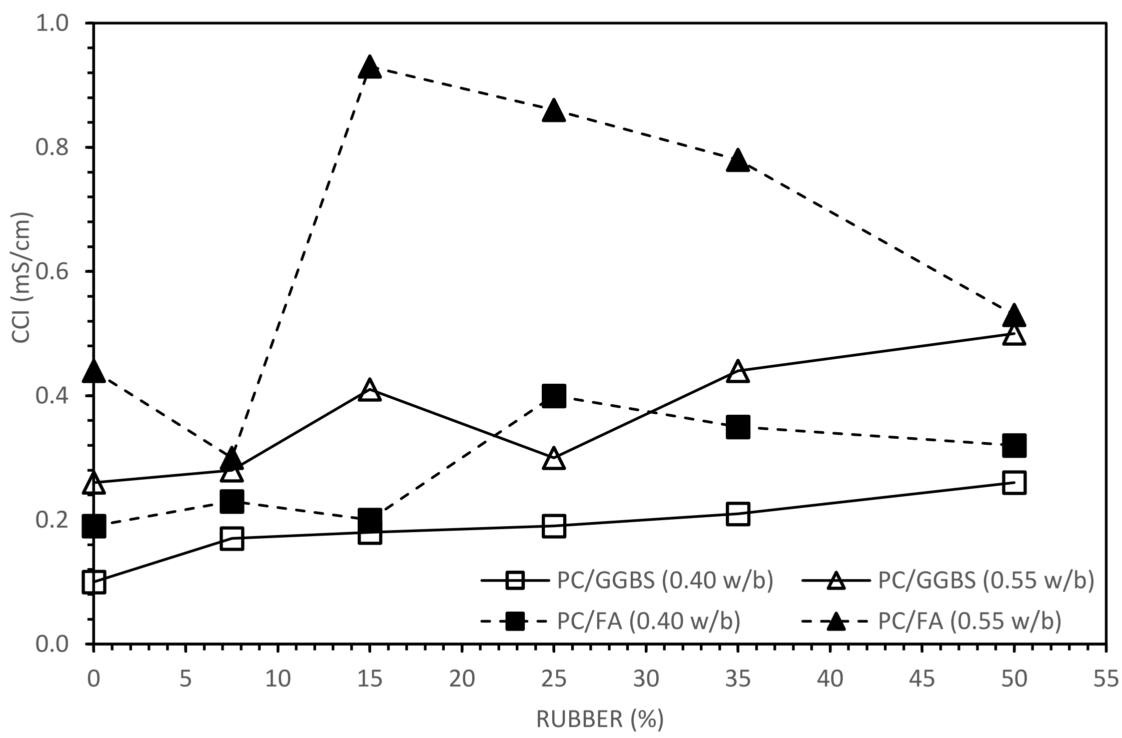


Figure 4. 39: Effect of rubber content on CCI at 56 day

4.4.5 Practical implications for rubberized concrete on the durability properties of non-structural concrete

All concrete specimens exhibit a minimum of 9.5 OPI, which can be classified as good durability class in accordance with performance-based specification. It is noted that all FA concrete irrespective of rubber content up to 50% replacing fine aggregate, exhibits above 10 OPI, excellent durability class requirement at lower w/b ratio of 0.40. At higher w/b ratio, specimens with up to 35% rubber content exhibited excellent durability and at 50%, specimens exhibited good durability. Therefore, GGBS and FA rubberized concrete can be used in non-structural construction works according to performance-based specification.

Local authorities in South Africa like SANRAL, compares the durability parameter OPI acceptance value to the limiting index value of minimum 9.8 for lab cured concrete specimens. Irrespective of low and high w/b ratio of 0.40 and 0.55 and two binder type GGBS and FA considered for varying percentage replacement of fine aggregate by volume from 7.5% to 50%, all rubberized concrete mixes exhibited an OPI(log scale) value well above 8.75, the minimum required for rejection of the concrete mix. At lower w/b ratio of 0.40, FA binder concrete gives permeability values above 9.8. However, at higher w/b ratio, up to 35% rubber content provides above 9.8 OPI. On other hand, in GGBS binder concrete, at lower w/b ratio, up to 35% rubber content mixes showed above 9.8 OPI. For higher w/b ratio, control mix shows an OPI of 9.61 at 56-day curing. It can be noted that specimens with rubber content exhibited very close values as to the control mix. The least OPI of 9.33 exhibited is at higher w/b ratio of GGBS binder type.at 28-d curing. At prolonged curing, these rubberized specimens recorded an OPI of 9.56. With FA binder type, rubberized concretes exhibited high OPI values at lower w/b ratio. The extent to which change in w/b ratio affects OPI is expected to decrease with age due to the improved interaction of the hydration products, for higher w/b ratio, at later ages. FA concretes were most affected by changes in w/b ratio. Water absorption increases with rubber content. Therefore, rubberized concrete is more susceptible to water, chloride and chemical attacks

(M.Pham *et al.*, 2019). However, it is noted that up to 50% replacement of fine aggregate by volume with tyre rubber produces OPI value to satisfy good durability requirement according to performance-based specification. Thus, showing good OPI durability properties for rubberized concrete.

GGBS concrete exhibit better water sorptivity values compared to FA concrete. WSI values decreases with increasing rubber content. It is observed that all specimens irrespective of w/b ratio, binder type and rubber content, exhibited WSI below 10 mm/$\sqrt{hr}$, satisfying the criteria for good durability class. It is noted that WSI exhibited below 6 mm/$\sqrt{hr}$, excellent durability class requirement, for mixes containing rubber content from 25% in GGBS concrete and for rubber content from 35% in FA concrete. The sorptivity values obtained falls below the maximum value of 12 mm/$\sqrt{hr}$ sorptivity requirement of SANRAL project. With the introduction of rubber particles, WSI values are observed to decrease with increasing percentage of rubber, especially in GGBS binder type, irrespective of w/b ratio and curing age. It can be concluded that concrete specimens with about 50% rubber content, show good durability property.

All concrete specimens exhibited chloride conductivity index below 1.5 mS/cm, requirement for good durability class according to performance-based specification. It is noted that GGBS rubberized concrete, irrespective of w/b ratio exhibited excellent durability CCI requirement of below 0.75 mS/cm. FA rubberized concrete exhibited below 0.75 mS/cm only at lower w/b ratio of 0.40. However, CCI values indicate good durability with GGBS and FA rubberized concrete to use in non-structural construction works. In GGBS concrete, irrespective of w/b ratio and rubber content, chloride conductivity values satisfy the requirements for all exposure classes as per SANRAL requirement given in Table 4.3. The maximum chloride conductivity noted in GGBS binder type concrete mix is 0.89mS/cm at 28-day curing, in higher w/b ratio for concrete specimens containing 50% rubber content. At lower w/b ratio, in GGBS binder type concrete mix, irrespective of curing age, increasing the rubber content increased the CCI values, thus giving poorer quality of concrete with respect to control mix. However, all GGBS rubberized concrete mixes satisfied the maximum CCI value suggested. In FA binder type concrete mix, XS 3b exposure class suggests a maximum of 1.10 mS/cm. For 28-day curing, at higher w/b ratio of 0.55, FA rubberized concrete specimens containing 25% and 35% rubber content, both exhibited a CCI of 1.14mS/cm, which does not satisfy the maximum criteria for use in exposure condition of XS 3b. At prolonged curing, these values reduced to 0.86 and 0.78mS/cm respectively. In FA binder type, at 7.5% rubber content, at higher and lower w/b ratio, irrespective of curing age, exhibited better durability property as compared to control mix. At lower w/b ratio, up to 25% showed better quality. However, it can be concluded that rubberized concrete replacing fine aggregate by rubber particles by volume with up to 50% rubber contents show good durability property with respect to maximum CCI values suggested.

4.5 General discussion on the use of rubber as aggregate in non-structural concrete

This study recommends the use of waste tyre rubber, by partially replacing fine aggregates of concrete up to 50%. With increase in rubber content, compressive strength decreases. Hence, use of rubberized concrete in non-structural concrete is recommended. From the study results, it can be concluded that the compressive strength for lower w/b ratio GGBS binder type concrete containing up to 50% replacement of fine aggregate with rubber particles satisfies the acceptance criteria of at least 2 MPa above minimum characteristic strength requirement of 15 MPa and 20 MPa for non-structural works in plain concrete and for non-structural concrete works with embedded metal. However, with increase in w/b ratio, GGBS binder type concrete satisfies the target strength requirement of non-structural plain concrete with only up to 35% rubber content replacement at 28-day curing and the requirement for non-structural work with embedded metal, with only 25% rubber content replacing fine aggregate by volume. On other hand, with FA binder type, concrete mix satisfied the acceptance criteria of at least 2 MPa above minimum characteristic strength requirement of 15 MPa for non-structural works in plain concrete, with up to 50% replacement of fine aggregate with rubber content at both 0.40 and 0.55 w/b ratio. For non-structural works including reinforcement, the criteria of at least 2 MPa above characteristic strength of 20 MPa is met with 50% rubber content replacing fine aggregate at lower w/b ratio of 0.40, however at higher w/b ratio only 35% rubber content concrete meets the requirement with prolonged curing. Durability properties exhibited by rubberized concrete can be categorized as good from the results analyzed.

According to Kumaran, Mushule and Lakshmipathy, the inclusion of rubber into concrete results in dynamic performance of rubber concrete because of its high resilient nature; durability and elasticity. In constructions that are subject to impact effects the use of rubberized concrete will be beneficial due to the altered state of conventional concrete with the inclusion of rubber particles (Kumaran, Mushule and Lakshmipat, 2008).

Schimizze concluded from his study that the elastic modulus of concrete mix containing fine rubber granular, showed a reduction of elastic modulus by 47% of that of control mixture. The reduction in elastic modulus indicates higher flexibility. This supports the use of rubberized concrete mixtures as stabilized base layers in flexible pavements (Kumaran, Mushule and Lakshmipat, 2008). From this study, it can be observed that elastic modulus reduces by 40% and 48% in GGBS rubberized concrete at 50% rubber content w.r.t control mix, at lower and higher w/b ratio of 0.40 and 0.55. In FA rubberized concrete, elastic modulus shows a reduction of 31% and 47% at lower and higher w/b ratio with 50% rubber content.

Crumbed rubber can be used for non-structural construction works, such as floor rips, partitions, support stone structures, concrete board siding, concrete blocks, and other non-structural uses. Currently, the waste tyre modified concrete is used in precast sidewalk panel, non-load bearing walls in buildings and precast roof for green buildings. Rubberized concrete can also be used in non-load bearing members such as lightweight concrete walls, building facades, or other light architectural units, thus the waste tyre modified concrete mixes could give a viable alternative to the normal weight concrete (Kumaran, Mushule and Lakshmipat, 2008).

4.6 Chapter summary

The experimental outcome from rubberized GGBS and FA concretes are summarized in this section. The results are analyzed to understand the impact of varying w/b ratio and binder type in rubberized concrete on strength and durability properties. This is to focus on the recycling of waste tyre rubber, to promote its use in non-structural concrete construction. This will thereby help in the reduction of tyre waste directed to landfill sites and thus help conserve the natural resources required for conventional concrete.

Results showed slump values below 15mm. According to BS EN 12350-2, slump test is sensitive to changes in the consistence of concrete which corresponds to slumps between 10mm and 210mm. Beyond these extremes slump is considered unsuitable. It was observed that in GGBS binder type mixes, slump showed value greater than 10mm, only in concrete mix containing 25% rubber content at lower w/b ratio of 0.40 and at 25% and 35% rubber content at higher w/b ratio of 0.55. Similarly, in FA binder type mixes, mixes containing 35% rubber content at lower w/b ratio and mixes containing both 35% and 50% rubber content at higher w/b ratio showed slump values greater than 10mm. For non-structural concrete, compaction of concrete is not as difficult as in the reinforced concrete constructions with high percentage of steel reinforcement, where compaction is difficult to be carried out with vibrators or other equipment. Therefore, a reduction of workability with addition of shredded tyre rubber particles does not hinder its application in non-structural concrete.

Compressive strength of mix concrete reduced significantly with the increase in rubber content. However, the addition of fly ash in rubberized concrete increases compressive strength compared to GGBS binder mix. At lower w/b ratio of 0.40, concrete specimens containing up to 50% replacement of fine aggregates with rubber particles, attained acceptance criteria of at least 2 MPa above minimum characteristic strength of

15 MPa and 20 MPa for plain and reinforced non-structural concrete works, with both GGBS and FA binder. At higher w/b ratio of 0.55, the acceptance strength requirement for non-structural plain concrete works was attained with the maximum of 50% in FA concrete, and with concrete containing only up to 35% rubber particles in GGBS concrete. However, for reinforced non-structural works, acceptance requirement was attained with only up to 25% fine aggregate replaced with rubber content in GGBS concrete and with 35% rubber content in FA concrete with prolonged curing.

The extent to which change in w/b ratio and binder type affects the durability properties of rubberized concrete, varies depending on the percentage of rubber particles incorporated in the mix design. Irrespective of the variations with w/b ratio and binder type, rubberized concrete satisfied the performance-based specifications of good durability class of oxygen permeability, water sorptivity and chloride conductivity; i.e., above 9.5 (log scale) for OPI, less than 10 mm/$\sqrt{hr}$ WSI and less than 1.5 mS/cm for CCI. FA rubberized concrete exhibited better OPI values compared to GGBS rubberized concrete at all percentage replacement of fine aggregate with rubber content. WSI values decreased with increasing rubber content. Therefore, concrete specimens with about 50% rubber content, show good water sorptivity property. GGBS concrete exhibit better water sorptivity values compared to FA concrete. Increasing the rubber content increased the CCI values, thus giving poorer quality of concrete with respect to control mix. However, even with decreasing resistance to chloride penetration with increasing rubber content, both GGBS and FA rubberized concrete with up to 50% rubber content mixes irrespective of w/b ratio, exhibited values below 1.5 mS/cm. GGBS rubberized concrete gave better resistance to chloride penetration as compared to FA rubberized concrete.

Results obtained was compared to the acceptance criteria of local authorities like SANRAL. Irrespective of low and high w/b ratio of 0.40 and 0.55 and two binder type GGBS and FA considered for varying percentage replacement of fine aggregate by volume from 7.5% to 50%, all rubberized concrete mixes exhibited an OPI(log scale) value well above 8.75, the minimum required for rejection of the concrete mix.

Sorptivity values obtained for all mixes falls below the specified maximum value of 12 mm/$\sqrt{hr}$. Chloride conductivity value satisfy the maximum requirements for all exposure classes to chloride induced corrosion as per binder type specified by SANRAL. Thus, it can be concluded that introducing up to 50% tyre rubber content to replace fine aggregates in GGBS and FA binder concrete mix, satisfies the durability requirements of concrete.

The next chapter outlines the results and discussions of the investigation and concludes the feasibility of the study results for use of rubber particles in non-structural concrete works in construction industry. It also includes recommendations for future research.

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CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Introduction

The objective of the study was to determine the feasibility of using rubberized concrete, with maximum percentage of aggregates replaced by rubber particles, to be used in non-structural applications. It is achieved by investigating the effect of water-to-binder ratio, binder type and varying replacement percentage of fine aggregate by volume with shredded tyre rubber particles, on compressive strength and durability properties of concrete. The possibility of utilizing maximum percentage of waste material rubber in concrete, can be fulfilled by investigating the strength and durability properties from the viewpoint of utilizing these concretes for non-structural purposes such as lightweight concrete walls, building facades, architectural concrete units, stabilized base layers in flexible pavements, filling concrete and blinding concrete.

5.2 General conclusions

5.2.1 Workability

The results obtained showed that slump did not show significant change in all mixes prepared with GGBS and FA concrete for two water-to-binder ratios of 0.40 and 0.55, with increasing rubber content of 7.5, 15, 25, 35 and 50% replacing fine aggregate by volume. Slump showed values below 15mm. According to BS EN 12350-2, slump test exhibiting slump values below 10mm and over 210mm is considered unsuitable. Hence other methods of determining consistence should be considered. It was observed that in GGBS binder type mixes, slump showed value greater than 10mm, only in concrete mix containing 25% rubber content at lower w/b ratio of 0.40 and at 25% and 35% rubber content at higher w/b ratio of 0.55. Similarly, in FA binder type mixes, mixes containing 35% rubber content at lower w/b ratio and mixes containing both 35% and

50% rubber content at higher w/b ratio showed slump values greater than 10mm. Hence, with increasing rubber content, concrete exhibited better workability.

For non-structural concrete, compaction of concrete is not as difficult as in the reinforced concrete constructions with high percentage of steel reinforcement, where compaction is difficult to be carried out with vibrators or other equipment. Therefore, a reduction of workability with addition of shredded tyre rubber particles does not hinder its application in non-structural concrete.

5.2.2 Compressive Strength

It can be concluded from the results that strength of concrete decreases as the percentage of tyre rubber particles increases in concrete. It is observed that with increasing w/b ratio, strength further decreases for same percentage of rubber particles replacing fine aggregates. FA concrete yield higher strength compared to GGBS rubberized concretes. Strength improvements are observed with respect to prolonged curing.

It is observed that at lower w/b ratio of 0.40, concrete specimens containing up to 50% replacement of fine aggregates with rubber particles, attained acceptance criteria of at least 2 MPa above minimum characteristic strength of 15 MPa and 20 MPa for plain and reinforced non-structural concrete works, with both GGBS and FA binder. At higher w/b ratio of 0.55, the criteria for acceptance strength requirement for non-structural plain concrete works was attained with the maximum of 50% in FA concrete, and with concrete containing only up to 35% rubber particles in GGBS concrete. However, for reinforced non-structural works, acceptance strength requirement was attained with only up to 25% fine aggregate replaced with rubber content in GGBS concrete and with 35% rubber content in FA concrete with prolonged curing.

From a practical point, it is therefore very important for selecting and using a correct w/b ratio and curing period, as increasing percentage of fine aggregate is removed and replaced with rubber particles by volume. It is particularly important to mix containing

GGBS binder, as the compressive strength decreases potentially on increasing the percentage of rubber, as compared to FA containing mix. The act of adding water on site to improve workability needs to be addressed and the practice put to an end by initiating awareness.

5.2.3 Durability

Determination of durability indices are relevant to both structural and non-structural concrete construction works, as these tests are sensitive to material, environmental and constructional factors which influences concrete durability. It is applied in quality control of site concrete, optimization of mix proportion of concrete, performance-based specifications and long-term performance predictions.

From the overview of OPI test results, the DI performance-based approach can be implemented in practice. Various issues arising in construction industry can be identified and addressed. The different materials and their proportions used in the mixes will influence physical characteristics of the fresh and hardened concrete. The type of aggregates used may influence the water sorptivity. If concrete is to be designed for freeze/thaw exposure class, the freeze/thaw resistance will be high, if the capillary action is low, but resistance will be low if the pores in the concrete are saturated. Hence, knowing sorptivity index for non-structural concrete works is important. Chloride conductivity test is relevant as it is used in engineering practice for construction quality control, as it is sensitive to quality-related parameters such as water/binder ratio, binder type, curing and compaction.

The extent to which change in w/b ratio and binder type affects the durability properties of rubberized concrete, varies depending on the percentage of rubber particles incorporated in the mix design. Irrespective of the variations with w/b ratio and binder type, rubberized concrete satisfied the performance-based specifications of good durability class of oxygen permeability, water sorptivity and chloride conductivity; i.e., above 9.5 (log scale) for OPI, less than 10 mm/$\sqrt{hr}$ for WSI and less than 1.5 mS/cm for CCI. FA rubberized concrete exhibited better OPI values compared to GGBS rubberized concrete at all percentage replacement of fine aggregate with rubber content. WSI values decreased with increasing rubber content. Therefore, concrete specimens with about 50% rubber content, show good water sorptivity property. GGBS concrete exhibit better water sorptivity values compared to FA concrete. Increasing the rubber content increased the CCI values, thus giving poorer quality of concrete with respect to control mix. However, even with decreasing resistance to chloride penetration with increasing rubber content, both GGBS and FA rubberized concrete with up to 50% rubber content mixes irrespective of w/b ratio, exhibited values below 1.5 mS/cm. GGBS rubberized concrete gave better resistance to chloride penetration as compared to FA rubberized concrete.

FA rubberized concrete and GGBS rubberized concrete with about 50% rubber content, irrespective of w/b ratios of 0.40 and 0.55 used, exhibited good durability property, indicating its feasibility to be used for non-structural construction work.

It can be summarized that compressive strength and durability requirements are satisfied by all mixes with FA binder rubberized concrete to be used in non-structural construction works irrespective of w/b ratio used. However, at higher w/b ratio of 0.55, GGBS rubberized concretes with only up to 35% rubber particles replacing fine aggregate by volume satisfies strength requirements to be used in non-structural work, even though it exhibits good durability requirements up to 50% replacement with rubber particles.

5.2.4 Use of rubber in non-structural concrete

This study recommends the possibility of utilizing waste tyre rubber, by partially replacing fine aggregates of concrete up to 50% by volume, provided w/b ratio, binder type and curing period is understood clearly as requirements for nonstructural plain and reinforced concrete construction works differ.

Rubberized GGBS and FA concrete containing up to 50% rubber content, at lower w/b ratio of 0.40 tested can be used in both plain and reinforced non-structural construction works. However, at higher w/b ratio 0.55, FA concrete containing up to 35% and GGBS concrete containing up to 25% rubber content by replacing fine aggregate can be used for reinforced non-structural works. It can be used in non-structural construction work such as in blinding layer. Irrespective of structural or non-structural construction work, concrete should maintain its original form and serviceability when it is exposed to adverse environment. They influence each other. For example, failure of blinding can cause issues with the structure above. Hence, good durability requirement is essential for non-structural concrete work.

As percentage of rubber particles replacing fine aggregate by volume increases, compressive strength reduces drastically. However, the study showed GGBS and FA rubberized concrete with up to 50% rubber content can be used in non-structural construction works, non-load bearing members such as lightweight concrete walls, building facades, or other light architectural units.

From this study, it can be observed that elastic modulus reduces by 40% and 48% in GGBS rubberized concrete at 50% rubber content w.r.t control mix, at lower and higher w/b ratio of 0.40 and 0.55. In FA rubberized concrete, elastic modulus shows a reduction of 31% and 47% at lower and higher w/b ratio with 50% rubber content. The elastic modulus of concrete mix containing fine rubber granular shows a reduction of elastic modulus as compared to conventional concrete. The reduction in elastic modulus indicates higher flexibility, which supports the use of rubberized concrete mixtures as stabilized base layers in flexible pavements.

Good durability properties of FA and GGGBS rubberized concrete obtained from the study supports its application in non-structural construction like ditches, sidewalks and other similar civil works in marine environment, freeze-thaw exposure environment and where moderate humidity is 60-80% and external concrete is sheltered from rain and when concretes are exposed to cyclic wet and dry conditions.

It can thus be concluded that waste tyre modified concrete could give a viable alternative to the conventional concrete in non-structural construction works with FA and GGBS binder and up to 50% of fine aggregate replaced by volume with rubber particles.

5.3 Recommendations for future research

Based on this study, the following recommendations are put forward for further research:

a) This study can be extended by increasing the experimental variable ranges (w/b ratio, binder types, rubber percentage and curing period). The focus should be to help further understand the correlation between these variables and the effect on resulting concrete strength and durability properties.
b) In this study, the rubber sample utilized was not subjected to any pretreatment, to enhance the properties of resulting concrete. It would be interesting to subject rubber samples to different methods of pretreatment and investigate the impact on resulting concrete properties.
c) The influence of replacing different proportion of coarse aggregates, partial replacement of coarse and fine aggregates with rubber particles, on the strength and durability properties of concrete were not investigated in this study. It would be of interest to expand the investigation to include rubber content to replace coarse aggregates and partial replacement of coarse and fine aggregates, to further the understanding of the influence of rubber content as aggregates on the resulting concrete properties.
d) The study concentrated on PC/GGBS at 50/50 and PC/FA at 70/30 proportion. It would be fascinating to alter the binder proportions to understand the best combination for rubber content replacing aggregates.
e) Further studies should attempt to understand the fire hazards on the addition of rubber content to concrete.

APPENDIX A

MIX DESIGN CALCULATION

Concrete mix design is the process of finding the proportions of concrete mix in terms of binder, fine and coarse aggregates. Mix proportions are calculated using volumetric method, derived from ACI Standard 211.1-91, most used in South Africa and around the world. Actions in the mix design process are given below. Table A.1 gives the particle relative densities of cementitious materials.

Table A. 1:Particle relative densities of cementitious materials (Addis and Goodman, 2009)

Material	Particle relative density
CEM I 52.5N	3.14
Fly ash (FA)	2.30
Ground granulated blastfurnace slag (GGBS)	2.90

Step 1: Specify concrete properties and characteristics

28-day target strength = 25 MPa

Concrete to be placed by vibration, hence

Slump = 0-25 mm

K-value = 1.05

Aggregates are andesite from Afrisam Eikenhof

Fine aggregate are andesite crusher sand

Maximum stone size = 13.2 mm

Water/binder (w/b) ratio selected 0.40 and 0.55

Water demand = 195 l/m^3

Correction for stone size 13.2mm = +10 l/m^3

Therefore, total water demand, M_w = 195 + 10

=205 l/m^3

Step 2: Select, characterize and proportion materials

Binder type: OPC/GGBS and OPC/FA

Dry compacted bulk density (CBD) = 1605 kg/m^3

Relative density of andesite = 2.92

Fineness modulus of sand = 3.1

Step 3: Calculate cement content, M_c

$$M_c = \frac{M_w}{w/b} \qquad \text{Equation A.1}$$

Step 4: Calculate stone content, M_a

$$M_a = CBD * (K-0.1*FM) \qquad \text{Equation A.2}$$

Step 5: Calculate sand content, M_s

Calculate effective relative densities of blends of different materials as

$$D = \frac{100}{\frac{\%X}{D_x} + \frac{\%Y}{D_y + \cdots\ldots\ldots}} \qquad \text{Equation A.3}$$

Where,

D is the effective particle relative density of the blend;

%X and %Y are percentages of mass of materials X and Y respectively in the blend;

D_x and D_y are particle relative densities of materials X and Y respectively

For, (i) 70/30 OPC/FA

$$D_c = \frac{100}{\frac{70}{3.14} + \frac{30}{2.30}} = 2.83$$

(ii) 50/50 OPC/GGBS

$$Dc = \frac{100}{\frac{50}{3.14} + \frac{50}{2.9}} = 3.02$$

The sand content is calculated, assuming full compaction of the concrete, from the fact volume of concrete is equal to the sum of absolute volumes of cement, sand, stone and water.

i.e. Volume of concrete =1 m^3

$$1 = \frac{M_c}{D_c*1000} + \frac{M_s}{D_s*1000} + \frac{M_a}{D_a*1000} + \frac{M_w}{1*1000} \qquad \text{Equation A.4}$$

Where,

M and D respectively to mass, kg and particle relative density;

Subscripts c, s, a and w refer respectively to cementitious material, sand, stone and water.

Therefore,

$$M_s = D_s * 1000 \left[1 - \frac{M_c}{D_c*1000} - \frac{M_a}{D_a*1000} - \frac{M_w}{1*1000}\right] \qquad \text{Equation A.5}$$

Reference

Addis, B., Goodman, J., 2009. Chapter 11: Concrete Mix Design, in: Owens, G. (Ed.), Fulton's Concrete Technology. 9th edn. Cement & Concrete Institute, South Africa, pp. 219–228.

APPENDIX B

STATISTICAL ANALYSIS: t – DISTRIBUTION

William Gosset developed t-distribution for statistical tests in 1908. It represents a continuous statistical distribution. The T-distribution also called Student's T-distribution, is used when the sample size is small. It tends to be heavier tailed than the normal distribution (Underhill and Bradfield, 2013). Confidence interval gives an acceptable range for the sample. Confidence interval is defined by the following equation:

$$(\bar{X} - t_{n-1} * s/\sqrt{n} \ , \ \bar{X} + t_{n-1} * s/\sqrt{n})$$

Equation B.1

where:

$\bar{X}$ = sample mean

t = t-value obtained from the t-tables. For 95% confidence intervals, use the column headed 0.025

s = sample standard deviation

n = sample size

The standard sample deviation is calculated as follows:

$$s = \sqrt{\frac{1}{n} * \sum_{i=1}^{n}(Xi - \bar{X})^2}$$

where:

X_i = individual test results

Reference

Underhill, L., Bradfield, D., 2013. Chapter 9: The t-distribution and F-distributions, in: Introstat. Department of Statistical Sciences University of Cape Town, pp. 199–226.

APPENDIX C

COMPRESSIVE STRENGTH RESULTS

Table C. 1: Compressive strength values for GGBS concrete

Designation	Age	Average [1] Strength (MPa)	Std. Deviation	Deviation[2]	Range	
					Min (MPa)	Max (MPa)
GB-0%-0.40	7	40.2	1.7	4.28	35.9	44.5
	28	60.3	0.9	2.13	58.1	62.4
	56	67.7	1.6	4.01	63.7	71.7
GB-0%-0.55	7	24.9	0.3	0.72	24.1	25.6
	28	44.4	0.8	1.92	42.5	46.3
	56	50.2	3.0	7.37	42.8	57.6
GB-7.5%-0.40	7	33.7	0.5	1.27	32.4	34.9
	28	50.7	1.7	4.24	46.4	54.9
	56	57.9	0.8	1.96	56.0	59.9
GB-7.5%-0.55	7	23.5	0.7	1.75	21.8	25.3
	28	35.7	1.7	4.19	31.6	39.9
	56	41.9	2.1	5.32	36.6	47.2
GB-15%-0.40	7	31.0	0.6	1.42	29.6	32.4
	28	45.0	0.7	1.65	43.4	46.7
	56	52.8	1.2	2.93	49.9	55.7
GB-15%-0.55	7	16.6	0.3	0.84	15.7	17.4
	28	27.0	0.4	1.04	26.0	28.1
	56	31.5	1.0	2.41	29.1	33.9
GB-25%-0.40	7	23.3	0.4	1.04	22.3	24.3
	28	34.8	1.2	3.02	31.8	37.8
	56	41.0	1.2	2.94	38.0	43.9
GB-25%-0.55	7	13.7	0.5	1.15	12.6	14.9
	28	23.9	0.3	0.68	23.2	24.6
	56	27.2	0.9	2.28	24.9	29.5
GB-35%-0.40	7	20.7	0.7	1.67	19.1	22.4
	28	30.2	1.0	2.39	27.8	32.6
	56	36.3	0.9	2.18	34.1	38.5
GB-35%-0.55	7	9.8	0.2	0.48	9.3	10.3
	28	17.8	0.8	2.02	15.8	19.9
	56	21.2	0.3	0.63	20.5	21.8
GB-50%-0.40	7	13.9	0.7	1.70	12.2	15.6
	28	21.9	0.4	0.97	20.9	22.9
	56	24.9	0.2	0.60	24.3	25.5
GB-50%-0.55	7	6.7	0.5	1.27	5.4	7.9
	28	12.2	0.2	0.59	11.6	12.8
	56	14.4	0.8	2.09	12.3	16.5

Notes (refer to table superscript references) : 1. Average of 3 readings; 2. For a 95% confidence level, t-value = 4.303; number of specimens tested = 3.

Table C. 2: Compressive strength values for FA concrete

Designation	Age	Average [1] OPI	Std. Deviation	Deviation[2]	Range	
					Min.	Max.
FA-0%-0.40	7	46.9	0.8	2.02	44.9	48.9
	28	61.6	3.0	7.48	54.2	69.1
	56	73.8	2.1	5.13	68.7	78.9
FA-0%-0.55	7	44.6	0.8	2.11	42.5	46.7
	28	60.9	2.9	7.12	53.7	68.0
	56	68.7	1.2	2.89	65.8	71.6
FA-7.5%-0.40	7	49.2	1.2	3.08	46.1	52.3
	28	63.5	3.6	8.86	54.7	72.4
	56	70.7	3.1	7.60	63.1	78.3
FA-7.5%-0.55	7	36.5	1.2	3.08	33.4	39.6
	28	48.8	2.4	5.87	42.9	54.6
	56	58.9	2.1	5.18	53.8	64.1
FA-15%-0.40	7	39.9	1.8	4.55	35.3	44.4
	28	55.0	2.4	5.86	49.1	60.8
	56	61.6	0.7	1.81	59.8	63.4
FA-15%-0.55	7	21.7	0.7	1.76	19.9	23.5
	28	31.1	1.8	4.49	26.6	35.6
	56	37.6	2.3	5.62	31.9	43.2
FA-25%-0.40	7	31.8	0.8	2.04	29.8	33.9
	28	42.9	1.4	3.39	39.5	46.3
	56	50.8	0.4	0.99	49.8	51.8
FA-25%-0.55	7	17.1	1.2	2.92	14.2	20.0
	28	23.6	0.4	0.91	22.7	24.5
	56	28.3	0.5	1.23	27.1	29.6
FA-35%-0.40	7	30.0	0.3	0.86	29.1	30.9
	28	39.3	1.4	3.49	35.8	42.8
	56	44.2	1.8	4.47	39.8	48.7
FA-35%-0.55	7	14.4	0.6	1.48	12.9	15.9
	28	19.6	0.5	1.17	18.4	20.8
	56	23.8	0.7	1.66	22.2	25.5
FA-50%-0.40	7	21.6	1.8	4.59	17.0	26.2
	28	29.5	0.6	1.40	28.1	30.9
	56	34.9	0.6	1.59	33.3	36.5
FA-50%-0.55	7	12.8	0.3	0.78	12.0	13.5
	28	17.1	0.9	2.33	14.7	19.4
	56	19.1	1.9	4.84	14.3	24.0

Notes (refer to table superscript references) : 1. Average of 3 readings; 2. For a 95% confidence level, t-value = 4.303; number of specimens tested = 3.

APPENDIX D

DURABILITY INDEX RESULTS

D1. OPI TEST RESULTS

Table D. 1: OPI test results for GGBS concrete

Designation	Age	Average [1] (m/s3)	Std. Deviation	Deviation (2)	Range	
					Min (MPa)	Max (MPa)
GB-0%-0.40	28	10.3	0.3	0.47	9.8	10.8
	56	10.3	0.2	0.34	10.0	10.6
GB-0%-0.55	28	9.4	0.1	0.09	9.3	9.5
	56	9.6	0.1	0.19	9.4	9.8
GB-7.5%-0.40	28	9.7	0.2	0.32	9.3	10.0
	56	10.3	0.3	0.47	9.9	10.8
GB-7.5%-0.55	28	9.6	0.2	0.39	9.2	10.0
	56	9.5	0.1	0.22	9.3	9.7
GB-15%-0.40	28	10.1	0.2	0.36	9.7	10.5
	56	10.2	0.1	0.18	10.1	10.4
GB-15%-0.55	28	9.3	0.1	0.16	9.2	9.5
	56	9.5	0.2	0.26	9.2	9.7
GB-25%-0.40	28	9.9	0.3	0.45	9.4	10.3
	56	9.8	0.2	0.34	9.4	10.1
GB-25%-0.55	28	9.5	0.1	0.22	9.3	9.7
	56	9.6	0.2	0.33	9.3	9.9
GB-35%-0.40	28	10.0	0.3	0.51	9.5	10.5
	56	9.8	0.1	0.11	9.7	9.9
GB-35%-0.55	28	9.5	0.2	0.24	9.3	9.8
	56	9.4	0.1	0.14	9.3	9.5
GB-50%-0.40	28	9.7	0.2	0.34	9.4	10.1
	56	9.8	0.2	0.37	9.4	10.2
GB-50%-0.55	28	9.3	0.0	0.06	9.3	9.4
	56	9.6	0.0	0.03	9.5	9.6

Notes (Refer to table superscript references) :

1. Average of four readings
2. For a 95% confidence level, t-value = 3.182; number of specimens tested = 4.

Table D. 2: OPI test results for FA concrete

Designation	Age	Average (1) OPI	Std. Deviation	Deviation (2)	Range	
					Min (MPa)	Max (MPa)
FA-0%-0.40	28	10.0	0.2	0.37	9.6	10.3
	56	10.2	0.2	0.29	9.9	10.5
FA-0%-0.55	28	9.7	0.2	0.37	9.3	10.1
	56	9.9	0.2	0.33	9.6	10.2
FA-7.5%-0.40	28	10.3	0.1	0.23	10.0	10.5
	56	10.3	0.2	0.27	10.0	10.5
FA-7.5%-0.55	28	10.3	0.2	0.27	10.1	10.6
	56	10.3	0.1	0.23	10.0	10.5
FA-15%-0.40	28	10.1	0.1	0.15	10.0	10.3
	56	10.4	0.2	0.33	10.0	10.7
FA-15%-0.55	28	9.8	0.4	0.58	9.3	10.4
	56	9.8	0.2	0.37	9.4	10.1
FA-25%-0.40	28	9.8	0.2	0.37	9.4	10.2
	56	10.3	0.2	0.31	10.0	10.6
FA-25%-0.55	28	9.7	0.3	0.48	9.2	10.2
	56	10.1	0.2	0.33	9.8	10.5
FA-35%-0.40	28	10.2	0.1	0.21	10.0	10.4
	56	10.2	0.2	0.26	10.0	10.5
FA-35%-0.55	28	10.0	0.2	0.29	9.7	10.3
	56	10.1	0.1	0.23	9.9	10.3
FA-50%-0.40	28	10.2	0.1	0.09	10.1	10.3
	56	10.2	0.1	0.14	10.1	10.4
FA-50%-0.55	28	9.7	0.2	0.31	9.4	10.0
	56	9.7	0.3	0.41	9.3	10.1

Notes (Refer to table superscript references) :

1. Average of four readings
2. For a 95% confidence level, t-value = 3.182; number of specimens tested = 4.

D.2. WSI TEST RESULTS

Table D.3: WSI test results for GGBS concrete

Designation	Age	Average[1] WSI	Std. Deviation	Deviation [2]	Range	
					Min.	Max.
GB-0%-0.40	28	7.2	1.3	2.14	5.1	9.4
	56	8.8	0.7	1.09	7.7	9.9
GB-0%-0.55	28	7.4	0.3	0.48	6.9	7.9
	56	8.8	2.3	3.71	5.1	12.6
GB-7.5%-0.40	28	7.2	0.5	0.85	6.3	8.0
	56	7.7	1.1	1.80	5.9	9.5
GB-7.5%-0.55	28	7.9	0.7	1.16	6.7	9.0
	56	8.8	0.6	0.95	7.9	9.8
GB-15%-0.40	28	6.4	0.6	0.95	5.4	7.3
	56	7.7	1.1	1.80	5.9	9.5
GB-15%-0.55	28	6.3	0.4	0.59	5.8	6.9
	56	6.6	0.6	1.00	5.6	7.6
GB-25%-0.40	28	5.5	0.4	0.56	4.9	6.0
	56	5.4	0.7	1.10	4.3	6.5
GB-25%-0.55	28	5.3	0.6	1.01	4.3	6.3
	56	4.7	0.5	0.73	4.0	5.5
GB-35%-0.40	28	4.9	0.3	0.45	4.4	5.3
	56	4.4	0.1	0.10	4.3	4.5
GB-35%-0.55	28	3.4	0.3	0.41	3.0	3.8
	56	4.0	1.1	1.76	2.2	5.7
GB-50%-0.40	28	3.9	0.4	0.63	3.2	4.5
	56	4.1	1.8	2.92	1.2	7.0
GB-50%-0.55	28	3.2	0.1	0.20	3.0	3.4
	56	3.9	1.2	1.91	2.0	5.8

Notes (Refer to table superscript references) :

1. Average of four readings
2. For a 95% confidence level, t-value = 3.182; number of specimens tested = 4.

Table D. 3: WSI test results for FA concrete

Designation	Age	Average[1] WSI	Std. Deviation	Deviation [2]	Range	
					Min.	Max.
FA-0%-0.40	28	8.5	0.4	0.62	7.9	9.1
	56	5.8	0.8	1.23	4.6	7.1
FA-0%-0.55	28	7.9	0.4	0.57	7.3	8.5
	56	7.9	1.1	1.77	6.1	9.7
FA-7.5%-0.40	28	7.9	0.8	1.27	6.6	9.2
	56	8.2	0.5	0.84	7.4	9.0
FA-7.5%-0.55	28	7.4	0.7	1.07	6.4	8.5
	56	6.4	1.3	2.10	4.3	8.5
FA-15%-0.40	28	6.1	0.7	1.05	5.1	7.2
	56	8.4	1.1	1.76	6.7	10.2
FA-15%-0.55	28	6.1	0.4	0.64	5.4	6.7
	56	5.9	0.3	0.42	5.5	6.3
FA-25%-0.40	28	6.1	0.3	0.53	5.6	6.7
	56	7.5	0.6	0.93	6.6	8.5
FA-25%-0.55	28	4.9	1.0	1.64	3.2	6.5
	56	5.2	0.7	1.11	4.1	6.3
FA-35%-0.40	28	5.2	0.2	0.38	4.8	5.5
	56	5.4	1.0	1.58	3.8	7.0
FA-35%-0.55	28	4.7	0.9	1.38	3.3	6.1
	56	5.2	1.3	2.08	3.1	7.3
FA-50%-0.40	28	5.3	0.2	0.29	5.0	5.6
	56	5.0	0.2	0.34	4.7	5.4
FA-50%-0.55	28	4.7	0.5	0.83	3.8	5.5
	56	4.1	0.4	0.70	3.4	4.8

Notes (Refer to table superscript references) :

1. Average of four readings
2. For a 95% confidence level, t-value = 3.182; number of specimens tested = 4.

D.3. CCI TEST RESULTS

Table D. 4: CCI test results for GGBS concrete

Designation	Age	Average[1] CCI	Std. Deviation	Deviation (2)	Range	
					Min (MPa)	Max (MPa)
GB-0%-0.40	28	0.11	0.03	0.04	0.07	0.16
	56	0.10	0.02	0.03	0.07	0.14
GB-0%-0.55	28	0.32	0.07	0.10	0.22	0.42
	56	0.26	0.12	0.19	0.07	0.45
GB-7.5%-0.40	28	0.18	0.06	0.10	0.08	0.29
	56	0.18	0.03	0.05	0.12	0.23
GB-7.5%-0.55	28	0.26	0.06	0.09	0.17	0.35
	56	0.28	0.01	0.01	0.27	0.29
GB-15%-0.40	28	0.25	0.03	0.04	0.21	0.29
	56	0.18	0.03	0.05	0.12	0.23
GB-15%-0.55	28	0.65	0.10	0.16	0.49	0.81
	56	0.41	0.07	0.11	0.30	0.52
GB-25%-0.40	28	0.30	0.04	0.06	0.24	0.36
	56	0.19	0.01	0.02	0.18	0.21
GB-25%-0.55	28	0.55	0.03	0.05	0.51	0.60
	56	0.30	0.02	0.04	0.26	0.34
GB-35%-0.40	28	0.36	0.05	0.08	0.28	0.43
	56	0.21	0.01	0.02	0.19	0.24
GB-35%-0.55	28	0.72	0.16	0.25	0.47	0.98
	56	0.44	0.03	0.05	0.39	0.48
GB-50%-0.40	28	0.37	0.04	0.07	0.30	0.44
	56	0.26	0.03	0.05	0.21	0.32
GB-50%-0.55	28	0.89	0.11	0.17	0.72	1.06
	56	0.50	0.08	0.13	0.37	0.63

Notes (Refer to table superscript references) :

1. Average of four readings
2. For a 95% confidence level, t-value = 3.182; number of specimens tested = 4.

Table D. 5: CCI test results for FA concrete

Designation	Age	Average [(1)] CCI	Std. Deviation	Deviation [(2)]	Range	
					Min (MPa)	Max (MPa)
FA-0%-0.40	28	0.44	0.07	0.11	0.32	0.55
	56	0.19	0.02	0.03	0.16	0.22
FA-0%-0.55	28	0.84	0.01	0.02	0.82	0.86
	56	0.44	0.09	0.14	0.30	0.57
FA-7.5%-0.40	28	0.38	0.00	0.01	0.37	0.39
	56	0.23	0.02	0.03	0.20	0.27
FA-7.5%-0.55	28	0.30	0.01	0.02	0.28	0.31
	56	0.30	0.01	0.02	0.28	0.31
FA-15%-0.40	28	0.33	0.01	0.02	0.31	0.34
	56	0.20	0.01	0.02	0.18	0.22
FA-15%-0.55	28	1.10	0.17	0.27	0.83	1.37
	56	0.93	0.08	0.13	0.80	1.05
FA-25%-0.40	28	0.69	0.05	0.09	0.60	0.78
	56	0.40	0.05	0.08	0.32	0.47
FA-25%-0.55	28	1.14	0.08	0.12	1.01	1.26
	56	0.86	0.06	0.09	0.77	0.96
FA-35%-0.40	28	0.31	0.02	0.04	0.27	0.34
	56	0.35	0.05	0.08	0.27	0.43
FA-35%-0.55	28	1.14	0.03	0.05	1.09	1.19
	56	0.78	0.05	0.08	0.70	0.87
FA-50%-0.40	28	0.65	0.09	0.15	0.50	0.79
	56	0.32	0.02	0.03	0.29	0.35
FA-50%-0.55	28	0.83	0.02	0.03	0.79	0.86
	56	0.53	0.02	0.02	0.50	0.55

Notes (Refer to table superscript references) :

1. Average of four readings
2. For a 95% confidence level, t-value = 3.182; number of specimens tested = 4.