Green concrete
–
An Investigation into the use of waste materials for concrete applications in the South African construction sector

By

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Abstract

Concrete is one of the most common construction material in the world, which contributes approximately 5% of man-made CO_2 globally (C&CI, 2011). An increase in consumer resource consumption and waste production creates an opportunity for research into concrete which uses less virgin minerals, waste materials and has a lower cement content. Such prior research into alternative aggregates has not only delivered innovative high-end concrete adaptations for high performance and specialist applications, but also economical greener alternative concrete solutions which provide a means for developing nations to improve their standard of living by incorporating materials which are cheaper or more environmentally friendly and still yield adequate performance. This study is aimed at the latter, by investigating the potential viability of waste concrete mixes that involved the partial substitution of cement and aggregates with locally sourced waste products. Cement was substituted with coal bottom-ash (BA), stone aggregate with sugarcane bagasse fibre (SCBF) and recycled high density polyethylene (HPDE) plastic pellets.

In this study, an investigation into the potential viability of waste modified concrete in a local context was carried out by comparing strength (compressive, flexural, and splitting), durability (Oxygen Permeability Index, Chloride Conductivity, and Water Sorptivity), workability and economics (R/m^3 concrete cost & scenario analyses) with conventional concrete. The critical volume substitution for each waste material was found after evaluating the compressive strength of each waste and this was used to develop mixed waste specimens to investigate combinations of waste in concrete. To further investigate the behaviour of waste concrete mixes and fulfil knowledge gaps identified in literature, this study also investigated the effect of varying volume substitutions (2.5%, 5%, 10%, 20%, 40%) of waste materials on strength, workability, density and specific heat as well as properties such as elastic modulus, SEM analysis, and the effect of moisture state, using the optimum/critical volume substitutions identified.

The effect of varying substitutions of waste was shown to have a noticeable effect on concrete properties and the optimum/critical volume percentages were found to be 2,5% for HDPE, 5% for BA and 10% for SCBF. Waste mixes also performed best when used on their own as the mixed combinations generally were inferior in performance. The SCBF and BA mixes at 10% and 5% substitution respectively, did have potential viability for structural applications when using dry waste materials as workability was acceptable, mixes were above the target strength and were relatively durable based on this study. HDPE (2,5%) however, only had the potential for non-structural applications that support passive heating and cooling measures. However, with further research into developing a waste concrete mix design specification to achieve a pre-determined compressive strength, HDPE and the natural moisture state mixes that yielded compressive strengths above 30 MPa (general structural use), still had the future potential to be viable conventional concrete alternatives.

Economically, if the project was profit-driven, none of the waste mixes were viable in terms of maximising profit as they cost more than the conventional mix for both the small scale precast and larger scale cast in-situ scenarios. The waste mixes also did not offer any significant enhanced concrete properties to validate the higher costs. However, if corporate social responsibility (CSR) or government environmental incentives were factors, then by reducing virgin mineral consumption and utilizing waste materials, this promotes environmental preservation. Hence, the reduction in profits may therefore be the warranted, thus making the waste mixes economically viable in the context of CSR projects or with the potential for government environmental incentives.

Keywords: Green Concrete, Waste materials, Fibre Aggregate, Polymer Aggregate, Bottom-Ash

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Glossary

Absolute volume - The space occupied by a material in the solid state without any voids (Engineering-Dictionary, 2008).

Anthropogenic - Resulting from the influence of humans (Merriam-Webster, 2013).

Aggregate – Loose particulate materials such as sand, gravel or pebbles which are added to a cementing agent to make concrete, plaster etc. (Dictionary.com, 2014).

Bottom ash – The material that falls to the bottom of the boiler after burning coal (ALS Environmental, 2013).

C&CI- Cement and concrete institute

Cellulose -An inert carbohydrate $(C_6H_{10}O_5)_n$ which is a large constituent of plant cell walls (Farlex, 2013).

CC - Chloride Conductivity

Composite material – A combination of two or more distinct materials that do not merge but still impart their properties to the product resulting from the combination (The Business Dictionary, 2014) .

Concentration gradient – A difference between concentrations in a space (Biology Corner, 2014) .

Corporate Social Responsibility (CSR) – The voluntary action undertaken by companies to integrate social and environmental concerns in their business actions (United Nations Industrial Developement Organization, 2014).

Deleterious solids- Damaging or harmful (Merriam-Webster, 2013).

Diametric – Completely opposed (Merriam-Webster, 2014)

Extrinsic – Not part of something: coming from the outside of something (Merriam-Webster, 2014),

Heat sink – An environment or medium that absorbs excess heat (Farlex, 2013)

HPDE-High density Polyethylene- Dense hydrocarbon thermoplastic (Business dictionary, 2013).

HCP –Hardened Cement Paste.

Hemi cellulosic- Groups of complex carbohydrates that surround cellulose fibres of plant cells (Encylcopedia Britanica, 2013).

Hydration – The chemical reaction between water molecules and another compound (Encyclo, 2014).

Hydrophilic –Compounds that have an affinity to water (Hughes, 2004).

Hydrophobic – Compounds that are repelled by water (Hughes, 2004).

Hygroscopic – Readily taking up and retaining water (Merriam-Webster, 2014) .

HVAC – Heating, ventilation and air conditioning.

Interfacial zone – The region of a concrete matrix formed at aggregate cement interface with a higher W/C ratio and porosity.

Inert - Not readily chemically reactive with other materials, forming little if not any chemical compounds (Farlex, 2013).

Intrinsic - Occurring as a natural part of something (Merriam-Webster, 2014).

Lignin – Lignin is an amorphous polymer, oxygen-containing, organic compound and is the second most abundant organic material on Earth. It binds the cell walls of plants (Merriam-Webster, 2013).

Ligno cellulosic- Substance consisting of lignin that constitutes part of the woody cell walls of plants (Merriam-Webster, 2013).

OPI –Oxygen Permeability Index.

PPE –Personal Protective Equipment

Quicklime –The chemical compound calcium oxide (CaO) (Dictionary of Construction.com, 2014).

Resonance – When an object vibrates at the same natural frequency as a second object and causes vibrational motion of the second object (The Physics Classroom, 2014).

SCBF - Sugarcane Bagasse Fibre-The fibre waste after the sugar juice extraction process (Encyclopedia Britanica, 2013).

SEM - Scanning Electron Microscope

Slaked lime - The chemical compound calcium hydroxide [$Ca(OH)_2$] which is formed when adding water to CaO (The Free Dictionary by Farlex, 2003).

Specific heat - A measurable quantity that is the amount of heat energy (joules) required to raise the temperature of a unit mass (g) of material by 1°C (Àbout.com, 2013).

Specifications – Descriptions of aspects of work to be done (Addis, 1986).

Thermal mass – A term that describes a materials ability to store heat (The Concrete Centre, 2013).

Thermoplastic- A plastic that can be repeatedly softened and hardened through heating and cooling (Farlex, 2013).

van der Waal's forces – Electrostatic forces that attract water in liquid form together (Nave.R, 1999).

W/C – Water cement ratio

WS- Water Sorptivity.

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Chapter 1 **Introduction**

1.1. Introduction

“Viability is the ability to work as intended” (Cambridge University, 2014). Sustainability is long term viability which does not deplete or degrade the resources” (Russell, 2010).

The primary aim of this research is to investigate the potential viability of the waste materials (SCBF, HDPE and Bottom Ash) in concrete in comparison to conventional concrete. By using waste materials there is already an inherent reduction in virgin resource depletion. In this study the viability of using waste materials in concrete will be evaluated in terms of investigating if waste concrete mixes meet minimum strength requirements and are durable, workable and economical (Mishra, 2014) (Bjegović & Oslaković, 2013). A comprehensive statement on sustainability can only be proposed with a full life cycle analysis and this is beyond the scope of this study.

This chapter provides an overview of the study, describes the research background and indicates the research objectives as well as how they were achieved in order to accomplish the research aim. A summary of the thesis structure is shown by figure 1.1.

1.2. Research background and Rationale

The increasing global population brings about an increase in resource consumption and waste production, thus rendering current practice unsustainable in the long-term if changes in consumer behaviour or waste reduction measures are not employed. It is therefore necessary for each facet of society to conserve the environment in one way or another. The construction industry has played an active role in this regard producing sustainability assessment tools such as Leadership in Energy and Environmental Design (LEED) (U.S Green Building council, 2014) and Green Star S.A (Green Buidling Council of South Africa, 2014), incorporating energy efficiency into buildings through codes such as SANS 204 (SABS, 2011) and promoting the use of materials which are more sustainable and environmentally friendly. Concrete, being one of the worlds most used construction materials, is an ideal medium to implement sustainable measures due to its wide ranging market and adaptability. Adaptability in a sense that concrete is fundamentally a composite material comprised of a binder (cement) which holds a matrix (coarse & fine aggregates) together. Therefore, individual constituents can be substituted or modified with materials that exhibit similar or enhanced properties, to form varying composite materials, as long as the required concrete properties are maintained. Such constituent substitution can either result in high performance concrete variants aimed at specialist markets, or provide a variation of concrete which appeals to developing nations, as it may not provide enhanced performance but it might meet basic material property requirements at a reduced cost. An example of high performance concrete is the fly-ash concrete used for dam construction and river tunnels, researched by Thomas (2007), which exhibited a reduction in permeability, improved long-term strength and chloride resistance. An example of low-cost concrete for developing nations is the use of rice husk cement in Cuban low-cost housing panels researched by Swamy (1987), which has lasted over 18 years.

This research will aim to expand on knowledge of aggregate and cement substitutes and provide a local context by investigating the viability of three locally available waste materials to form an adaptation of conventional concrete referred to in this study as "*Green concrete*". Reducing virgin mineral consumption and utilizing waste, which by definition, is any *"unwanted or unusable substances or bi-products"* (Oxford Dictionary, 2013), supports environmental preservation.

The waste materials investigated in this research were: high density polyethylene (HDPE) recycled plastic, sugarcane bagasse fibres (SCBF), and coal bottom ash (BA). The latter two were sourced from the Illovo Sugar Mill in Sezela and HDPE was obtained from the RE-SA plastic recycling facilities in Prospecton.

SCBF was used to partially replace stone aggregate and was selected as it is an abundant local renewable fibre in KwaZulu-Natal, which has the potential to improve flexural and splitting strength (Sivarja, et al., 2010). HDPE was also used to partially substitute stone aggregate and was selected because South Africa has a high plastic recycling percentage (18%) and this implies relative availability of material (McKenzie, 2012). Even though HDPE has been documented to not have a noticeable influence on strength, it is a lighter material than the stone it replaces and hence this may result in lighter concrete mixes. If strength is not compromised this can be of benefit in terms of reducing the permanent load on a structure. Coal bottom ash will be used to partially substitute cement binder and was selected because even though it is not as reactive as fly-ash it still has the potential to improve long-term strength as per research by Kurama & Kaya (2007), possibly reduce alkalinity in the mix and hence protect fibres (Jorillo & Shimizu, 1992) and reduce pollution caused by ash dumps by making use of the material. Literature research into the use of all three waste aggregates in concrete was limited and no research was found on combining these waste aggregates in a concrete mix. This study was subsequently carried out to fulfil such knowledge gaps and expand knowledge on the topic.

In order for the waste concrete alternatives to be viable, they must be economically feasible whilst meeting the basic requirements of the conventional concrete, or offer enhanced properties at a premium. This study first compared the influence of the respective wastes on properties such as compressive strength, flexural strength, splitting strength, workability, density, Scanning Electron Microscope (SEM) analysis and specific heat. The critical volume was identified from the compressive cube strengths as per the research methodology of Patel, *et al* (1989). The critical volume was the volume which yielded the best performance and after which a noticeable decline in performance was noticed.

In order to narrow down the possible applications of the waste materials only the critical volume mixes and combinations of critical volume mixes were tested further for elastic modulus, moisture effects using dry waste aggregates and durability tests (Oxygen permeability index, Chloride Conductivity, and Water Sorptivity) to complete the material property evaluation. Lastly, the economic viability of these mixes was evaluated by calculating a simple R/m^3 waste concrete rate using current market material rates and applying them to a scenario analysis.

The research seeks to provide insight on the potential viability of the waste concrete mixes in terms of workability, strength, durability and cost which are main factors regarding material selection as per Mishra (2014) and Bjegović & Oslaković, (2013). It also fulfils knowledge gaps noticed in literature studies by comparing the additional tested material properties such as specific heat, density, elastic modulus, SEM analysis and the effects of moisture states for a range of waste proportions (2.5%, 5%, 10%, 20%, 40%) and mixtures of different wastes together.

1.3. Research question

Is the use of SCBF and recycled HDPE for use as partial aggregate substitutes and coal bottom ash as a partial binder substitute in concrete potentially viable in the South African context in terms of strength, workability, durability and cost?

1.4. Research Aim

To investigate the potential viability of waste materials (SCBF, HDPE, and BA) for concrete use in the South Africa construction sector in terms of strength, durability, workability and cost. In addition to this, provide further insight on waste concrete behaviour by investigating varying waste substitution, mixtures of waste materials and properties such as elastic modulus, density, SEM analysis, specific heat and the moisture effects of the selected waste materials (SCBF, HDPE, and BA).

1.5. Research Objectives

1. Investigate the effect of varying volumetric proportions (2.5%, 5%, 10%, 20%, and 40%) of waste materials (SCBF, HDPE, and BA) at natural moisture state, in terms of compressive cube strength (7 & 28 day), flexural strength (7 & 28 day), splitting strength (28 day), density, workability and specific heat (28 day).
2. Establish the potential critical volumetric substitution within the range of 2,5% - 40% substitution, to be used for further investigation, from the compressive cube strength results.
3. Further investigate the concrete mixes for each waste at critical volumes (based on compressive strength) and in combinations of waste at critical volume (BA + SCBF, HDPE + BA, SCBF + BA + HDPE), in terms of elastic modulus (28 day) and durability (28 day) (OPI, CC, and WS).
4. Identify whether the chosen waste materials perform better independently or in conjunction with each other by comparing individual waste mixes with mixed waste results.
5. Investigate the effect of moisture absorption on compressive cube strength and workability for each waste material at critical volumes, by testing mixes with wastes at dry moisture state and comparing the results to the natural state mix results.
6. Evaluate the material cost of each mix by creating and comparing the R/m^3 value using current market rates.
7. Evaluate construction cost implications in terms of a scenario analysis.
8. Comment on the viability of the waste concrete mixes in terms of strength, workability, durability and cost, and recommend future work to complement knowledge based on the findings obtained from this study.

1.6. Research Methodology

In order to achieve the objectives of the study the following approaches were adopted:

- A review was done on the literature and previous studies to create a precedent, contextualise the research, identify potential knowledge gaps and provide a theoretical background for testing methods and materials used.
- Concrete mixes were designed based on the Cement and Concrete Institute (C&CI) method with cement and aggregate quantities adjusted, based on the respective proportions of volumetric waste content. It is acknowledged that different aggregates do have varying water demands based on material properties such as absorption and porosity. However, water content was kept constant to quantify the effect of moisture absorption and moisture content on the respective concrete properties tested.
- Each individual waste mix, using wastes at natural moisture state and volumetric waste material percentages of 2.5%, 5%, 10%, 20% and 40%, were tested for compressive cube strength (7 & 28 day), flexural strength (7 & 28 day), splitting strength (28 day), density, SEM analysis, workability and specific heat (28 day) and compared with a control sample of no waste material content. This research assessed the effect on the concrete mix due to the effect of v**arying proportions of waste material** and **mixtures of waste materials** keeping all other variable such as Water/Cement (W/C) ratio constant. The concept of keeping W/C ratios constant and investigating the resulting effect on concrete properties was also evident in internationally published researched work by Yaragal & Prabhu (2013). It is acknowledged that W/C does have an impact on material properties. For example the higher the W/C ratio the lower the strength, but this effect can be a topic of future research aimed at developing specifications for waste concrete, using the outcomes from this study such as the potential critical volumetric substitution between 2,5 - 40% substitution.
- Compressive strength, workability and durability are the common material specifications that designers use for concrete. Compressive strength and durability are closely related. However, the tests to carry out durability are costly for a wide range of mixes. Therefore, it was deemed adequate to determine the critical volumes based on the compressive cube strength tests carried out for each waste between

2,5% and 40% volumetric substitutions. These critical volumetric substitutions were used for the mixed waste combinations and further testing.

- Only the critical volume waste mixes (based on compressive strength) and the mixed waste combinations were tested for elastic modulus and durability (OPI, CC, and WS) to narrow down the study and arrive at the most viable variant for each waste.
- The material test results of the mixed combinations and individual waste mixes were compared to identify if the material is best used independently or in a combination.
- To evaluate the moisture absorption effect, the mixes at critical volume were tested for compressive cube strength and workability, using dry waste materials. This was compared to the respective results with waste materials at natural state. The moisture absorption and moisture contents of each waste material were also calculated to enhance the discussion and explanation of results.
- For the economic analysis, rates were obtained from relevant suppliers and each mix was priced to arrive at a R/m^3 rate.
- The R/m^3 rate was applied to costing scenarios to identify the cost implications of using the mixed waste and optimum/critical volume individual waste mixes. Both small and larger works applications were investigated to identify the impact of costing for smaller items compared to larger items.
- The test carried on elastic modulus, SEM analysis, specific heat and density were used to fulfil potential knowledge gaps found in past research and enrich knowledge on the behaviour of the waste material concrete mixes.
- Based on the results of the workability, strength, durability and cost analysis, conclusions were drawn on the potential viability of the waste materials as partial materials substitutes in concrete.
- Recommendations for future research were stated in order to refine and expand on knowledge gained from this study.

1.7. Significance of research

Concrete is the one of most widely used modern day construction materials on Earth by weight (Cement Industry Federation, 2009). Past research into modifying the standard concrete mix has lead the development of high-performance mixes for specialist applications as well as low-cost or environmentally friendly alternatives to conventional concrete that uplift developing nations and preserve the environment.

In terms of the significance of this study:

- Locally available waste materials (SCBF, HDPE, and BA) were investigated, evaluating a range of volumetric partial substitutions as well as mixes of the selected waste materials which have not been attempted in prior research. The potential critical volumes for compressive strength were found for each waste between 2,5% - 40% volumetric substitution. This coupled with the effect of moisture states on tested concrete properties could serve as precursors for the development of specifications for the selected waste materials, as well as complement current data available, thus expanding knowledge on the topic.
- Knowledge gaps identified in literature regarding the tested properties were addressed, and insight into the behaviour of waste concrete was provided in terms of: compressive cube strength (7 & 28 day), flexural strength (7 & 28 day), splitting strength (28 day), density, SEM analysis, workability, specific heat (28 day), moisture effects, elastic modulus and durability.
- Relationships between strength, durability, workability, elastic modulus, etc., were investigated and could be useful for the purpose of material property predictions from known results.
- The costing analysis not only evaluated the economic viability of the waste concrete mixes, but also indicated whether there was a more significant impact on small precast works or larger cast in-situ works.

In terms of the significance of the primary research aim, if the materials were found to be potentially viable structurally and economically, this would aid in advocating their use locally and encourage further research into waste concrete standards and specifications being developed. This would ultimately reduce waste pollution and virgin mineral consumption by creating *"greener"* concrete alternatives.

1.8. Thesis structure

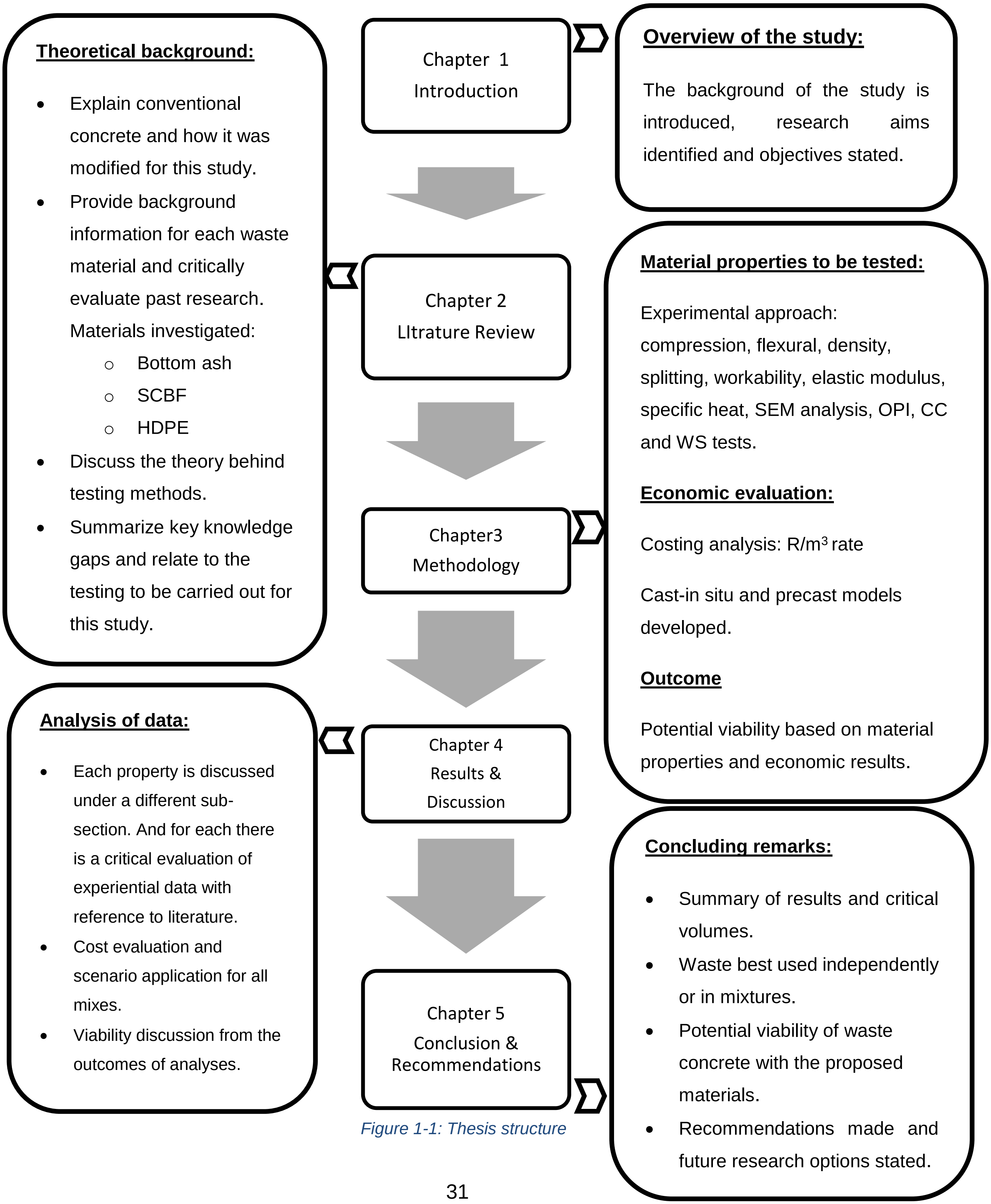


Figure 1-1: Thesis structure

1.9. Chapter Summary

This chapter introduced the reader to the study, provided a brief background for the research and indicated the research question, research aims, research objectives and the research methodologies that was implemented to accomplish the research objectives.

The next chapter will provide the theoretical foundation for the study and evaluate past research on the topic.

Chapter 2 **Literature review**

2.1. Introduction

As defined in the introductory chapter, the primary objective of this research was to investigate if the use of the selected waste materials (SCBF, HDPE, and BA) were potentially viable in concrete for the local context, in terms of workability, strength, durability and cost. In addition to this, investigations into the effects of varying waste substitutions, mixtures of waste materials and properties such as elastic modulus, density, SEM analysis, specific heat and the moisture effects, provided further insight into the behaviour of the selected waste materials (SCBF,HDPE,BA) in concrete.

This chapter provided a theoretical foundation for the research and related past research to this study. To do this the review was divided into sub-sections which:

- Gave a brief overview of conventional concrete and indicated what aspects of conventional concrete were modified in the context of this study.
- Explained the waste materials that were tested in this study (sugarcane bagasse fibre, high density polyethylene pellet and coal ash).
- Provided the theory and research background behind the material properties that were tested (workability, density, compressive strength, flexural strength, splitting strength, SEM analysis, specific heat, static elastic modulus, oxygen permeability, chloride conductivity, water sorptivity and moisture effects)
- Provided the theoretical reasoning for the economic analysis used.
- Summarised findings from the literature review

The overview of concrete in section 2.2.1, gave a general background about concrete and its constituents and indicated to the reader what aspects of concrete were changed for this study, compared to conventional concrete. For sections 2.2.2 to 2.2.3, based on the waste materials to be tested, a background for each material (SCBF, HDPE, and BA) was provided, followed by the methods used for its processing, literature properties, past research and the possible benefits and limitations based on literature.

In terms of tested concrete properties, the study involved tests which investigated certain properties associated with both fresh and hardened concrete states. Section 2.3, based on the tested material property literature, explained to the reader the theoretical aspects of the tested properties such as formulae used and the significance of the tests.

For the economic analysis, assumptions made and the reasoning adopted for the study were indicated in section 2.4.

Lastly, concluding remarks on literature were made in section 2.5 which highlighted the potential knowledge gaps identified from the literature review, to improve research on the viability of the investigated waste materials in concrete.

The summary in section 2.5 also served as a link to the methodology chapter of the thesis which described the evaluation procedures used in order to fill the identified knowledge gaps and meet the research aim.

2.2. Literature on materials investigated in this study

2.2.1. Conventional concrete & concrete modification

Concrete is widely regarded as the world's most used construction material, roughly twice as much as the total of other building materials combined (Chemistry world, 2008) with around 25 billion tonnes being produced globally every year (CSI, 2009). Figure 2-1 shows the annual production of materials globally between 2002 to 2004 and it can be seen that during this period more concrete was produced than oil and coal.

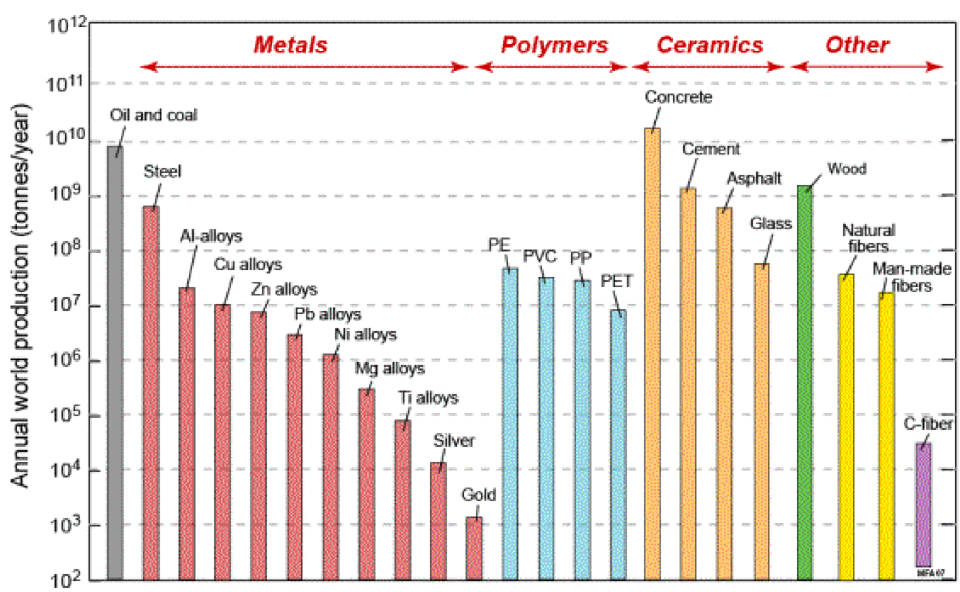


Figure 2-1: Annual world production of materials 2002-2004 (Granta-Material Intelligence, 2004)

Concrete is popular because once hardened, it is strong in compression, has the advantage of being easily moulded into virtually any shape, is long-lasting and requires little maintenance. Concrete is however a brittle material which is weak in tension, because it contains cracks and flaws which can quickly propagate when stretched under tension load and hence this must be remedied with reinforcing if critical (Jackson, 1980).

Concrete has both a fresh state and a hardened state. The fresh state in general lasts few hours and it is during this time that the required workability is essential to allow for concrete to be handled, transported, placed and compacted to the prescribed specification. Once hardened, properties such as strength and durability become critical as these are the performance criteria specified for the application. Concrete is traditionally a combination of water, cement, coarse aggregate (stone) and fine aggregate (sand), this mix is what is referred to as conventional concrete and will also be referred to as the control mix in this study.

Apart from concrete being a composite material comprised of the constituents mentioned, it can also be viewed as a three-phase material comprising of a transition zone, hardened cement phase and aggregate.

The transition zone is the thin shell (10-50 micron thick) between the aggregate and the cement paste (see Figure 2-2). Adsorption properties of aggregates affect the W/C ratio in the transition zone and the degree of water held on the surface of aggregates dictates the effect on properties related to W/C ratio such as strength and porosity. The transition zone is generally weaker than both the aggregate and hardened cement paste and therefore, even though the volume of the concrete matrix occupied by the transition zone is relatively small, the impact on concrete properties is significant. The transition zone is relevant to this study as the varying bond characteristics as well as the adsorption and absorption properties of the waste aggregates used may have an influence on the transition zone and hence the properties of the concrete matrix (Addis & Owens, 2001). For example, a hydrophobic material like plastic repels water which limits hydration in the transition zone and hence decreases bond strength.

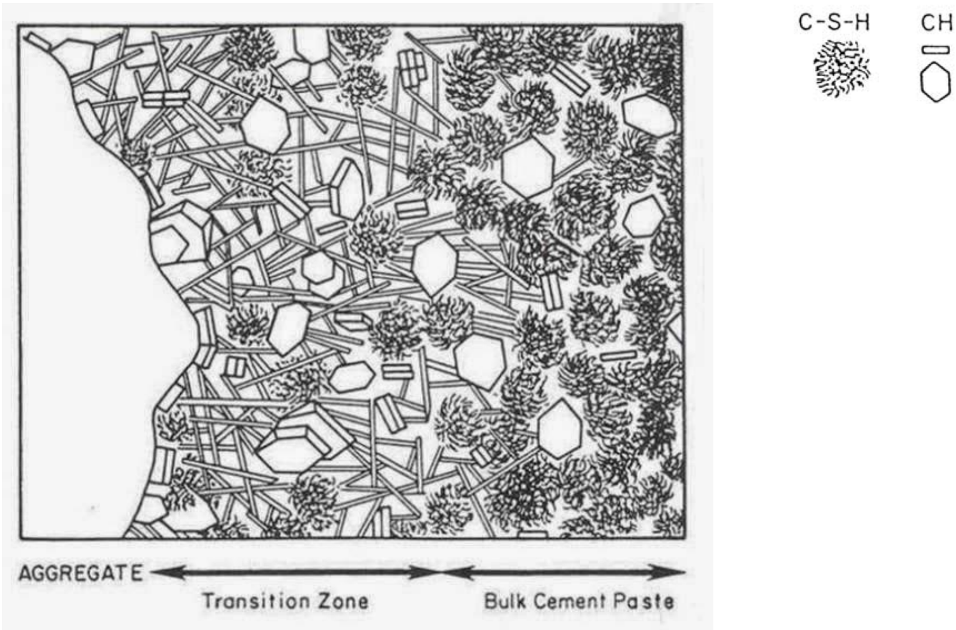


Figure 2-2: Diagram of the Interfacial zone (Caldarone, 2009)

The other two components of the three phase theory of concrete, namely cement paste and aggregates, were discussed further in general, within subsections 2.2.1.1 to 2.2.1.3, as the study involved the substitution of cement with coal bottom ash and coarse stone aggregate with recycled HDPE pellets and SCBF fibres.

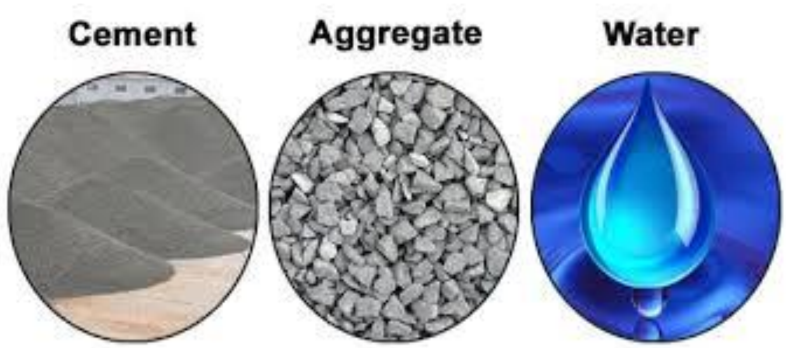


Figure 2-3 Concrete Composition (Mastour, 2013)

2.2.1.1. **Cement**

Concrete gains its strength from the hydration of cement which is the binding agent that holds the other constituents together. The earliest binders date as far back as the 3000 B.C. where the Egyptians used lime based mortars in the construction of the pyramids. Hydraulic cement using limestone was investigated by John Smeaton in 1756 (Bellis, 2013) but the "*Portland cement*" we know today was developed and named as such by Joseph Aspdin in 1824 after the colour of limestone found in Portland (Addis, 2008). A summary of some important dates related to the history of cement and concrete is shown by Figure 2-4.

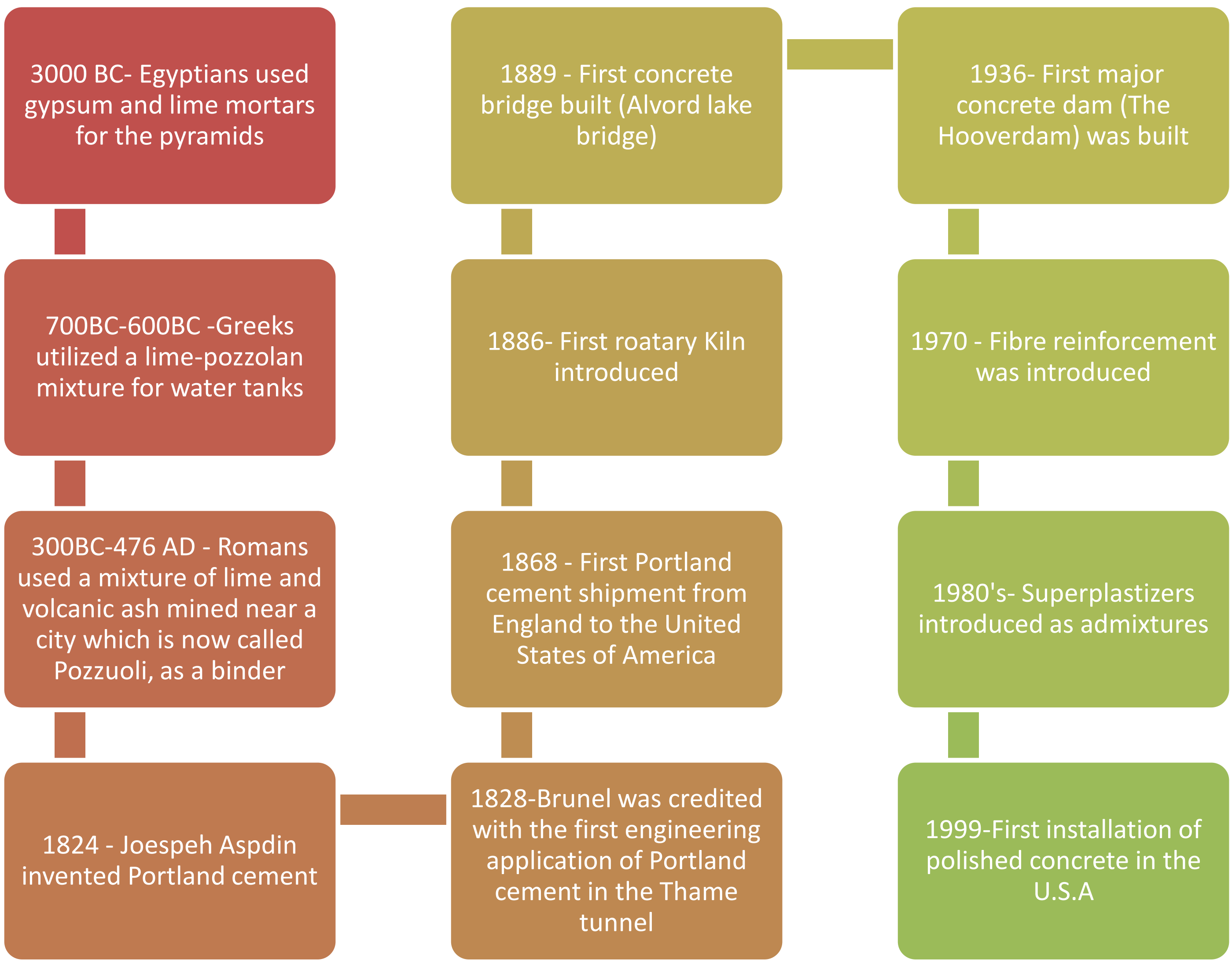


Figure 2-4: Summarised timeline of cement/binder history (Auburn, 2014)

The cement used today, being a hydraulic binder, hardens when mixed with water through the hydration reaction. The principal cement compounds that react with water during the hydration reaction are shown in Table 2-1.

Table 2-1 Cement Chemical compounds

Compound			**Content (%)**
Name	Formula	Abbreviation	
Tricalcium silicate	$3CaO.SiO_2$	C_3S	35-55
Dicalcium silicate	$2CaO.SiO_2$	C_2S	20-40
Tricalcium aluminate	$3CaO.Al_2O_3$	C_3F	5-12
Tetracalcium aluminoferrite	$4CaO.Al_2O_3.Fe_2O_3$	C_4AF	4-7

From the calcium silicates, aluminates and other the compounds that make up cement, tricalcium silicate (C_3S) and dicalcium silicate (C_2S) are the most stable. As shown in Table 2-1, they form a large portion of the cement chemical make-up and hence have the most significant influence on the concrete's strength (Jackson, 1980). Tricalcium silicate rapidly hydrates and is responsible for early-age strength with the initial set typically being reached after 4 hours, after which the concrete will not be workable (Neuwald, 2010). In contrast to Tricalcium silicate, dicalcium silicate hydrates slowly and generally only starts to develop strength after 7 days. The relationship between hydration time and compressive strength for the main compounds that make up cement are shown by Figure 2-5, extracted from the book "*Fundamentals of concrete"* by Brain Addis (Addis, 2008).

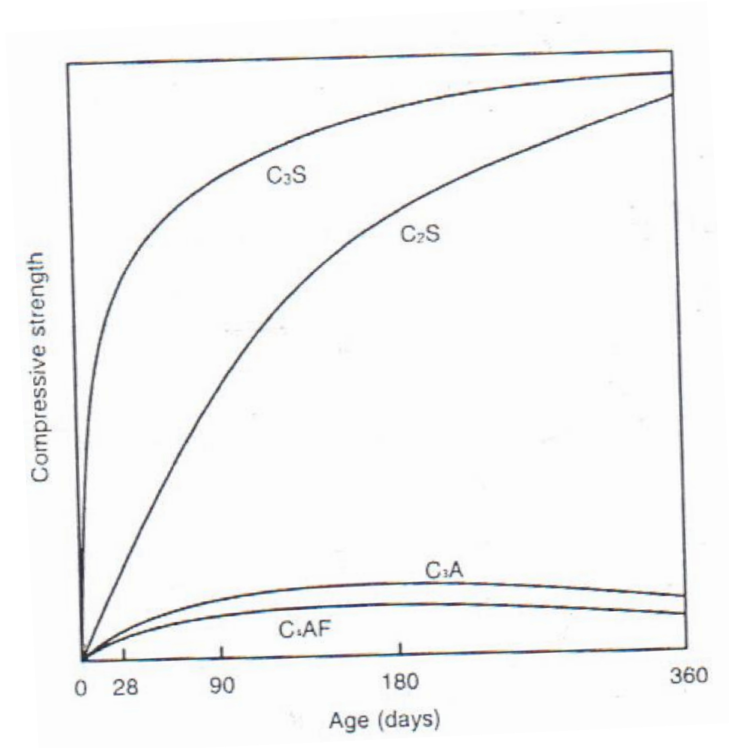


Figure 2-5: Strength Development (Addis, 2008)

The exothermic hydration reactions that take place when water (H_2O) reacts with C_3S and C_2S, are shown by the approximate chemical equations below:

$2Ca_3SiO_5 + 6H_2O \rightarrow Ca_3Si_2O_7.3H_2O + 3\ Ca(OH)_2$

$2Ca_2SiO_4 + 4H_2O \rightarrow Ca_3Si_2O_7.\ 3H_2O + Ca(OH)_2$

Alternatively in cement nomenclature:

$2C_3S + 6H \rightarrow C_3S_2H_3 + 3\ CH$

$2C_2S + 4H \rightarrow C_3S_2H_3 + CH$

The products of the hydration reaction are hydroxide ions and calcium silicate hydrate ($C_3S_2H_3$) which is sometimes shown as C-S-H. Calcium silicate hydrate forms a gel of very fine needles and plates which provide most of the hardened cement paste strength. Calcium hydroxide ($Ca(OH)_2$) forms crystals embedded in C-S-H gel and increases the pH of the pore water but does not affect the strength of the cement paste. The calcium hydroxide and

C-S-H act as seeds (nucleation points) from which more C-S-H can form which eventually interlock and hold the mix together (Amsterdam, 2000). C-S-H gel occupies more volume than the C_3S and C_2S it replaces, and because the overall volume of the cement paste does not change significantly after mixing, the capillary pore volume in the mix is decreased, permeability is reduced and hence compressive strength is improved. The hydration reaction does get slower over time but can continue for years, strengthening concrete as it ages, provided there is enough room for reaction products to form and moisture available for hydration (Thomas & Jennings, 2014). Figure 2-6 shows the basic structure of hardened cement extracted from *"Fundamentals of concrete"* (Addis, 2008) illustrating the gel of fine needles as described above.

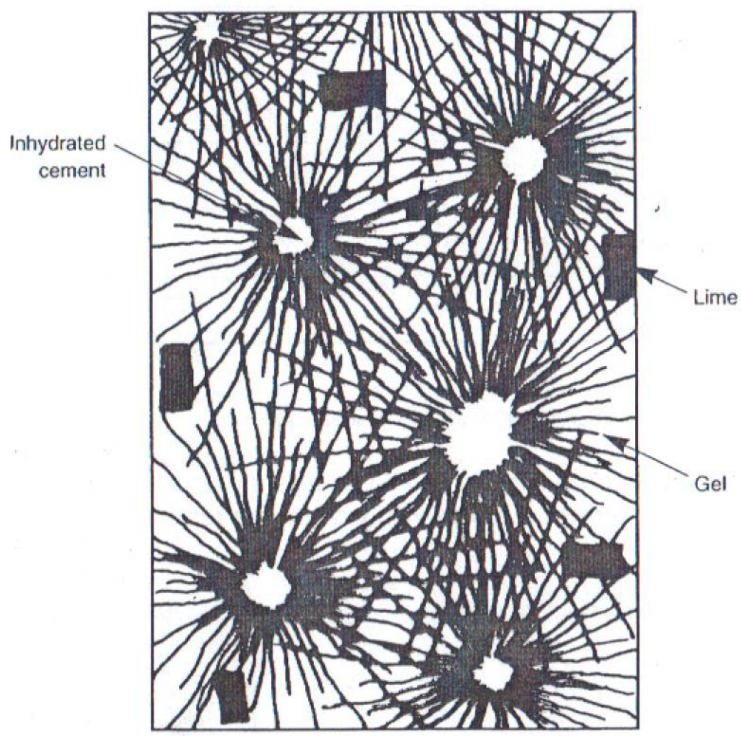


Figure 2-6: Cement matrix (Addis, 2008)

Cement does however have some negative aspects such as contributing significantly to the high CO_2 emissions related to concrete. Concrete contributes around 5% of global anthropogenic CO_2 emissions mainly due to the release of CO_2 from the manufacture of cement which is harmful to the environment (International Energy Agency, 2009). The energy intensive process of manufacturing cement generates approximately one ton of CO_2

from the burning of fuel and a further ton of CO_2 from the reaction that changes raw material to clinker, for every ton of cement produced (Wilson & Ding, 2007).

The cement manufacturing process shown by Figure 2-7, starts with the quarrying, crushing and prehomogenization of the raw materials, which are subsequently preheated through a series of vertical cyclones.

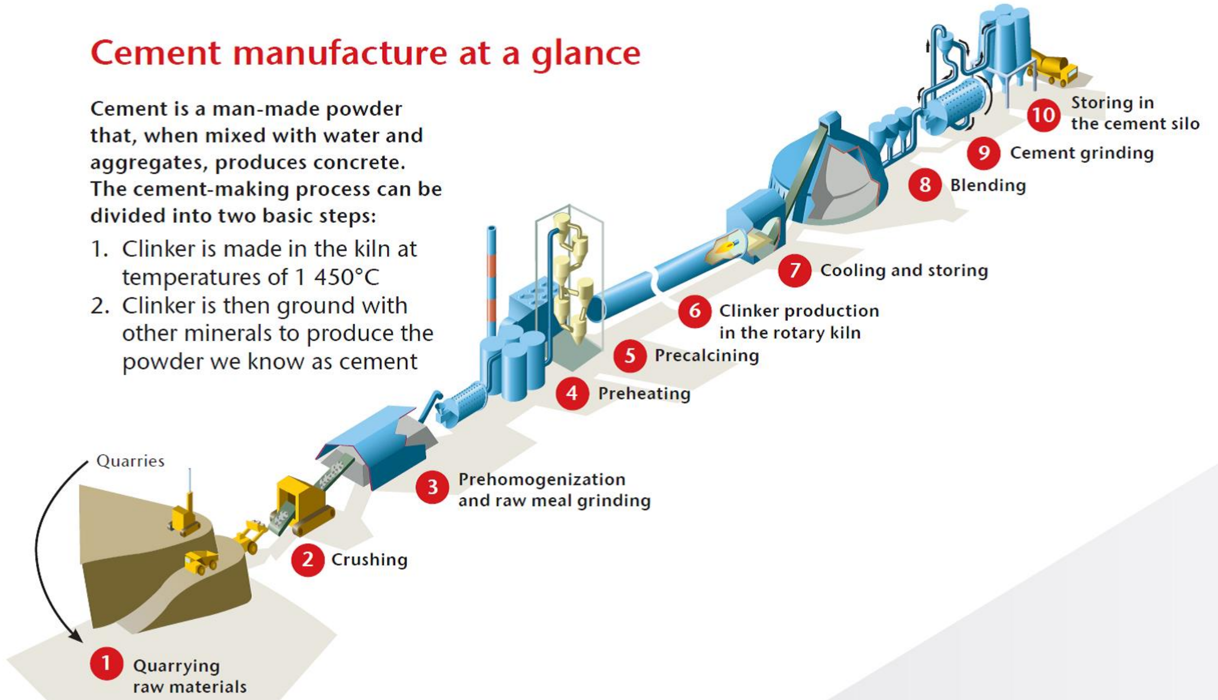


Figure 2-7: Cement manufacturing process (International Energy Agency, 2009)

Limestone ($CaCO_3$) is then decomposed to form quicklime (CaO) from the calcination process shown below.

$CaCO_3 \rightarrow CaO + CO_2$

The meal consisting of calcium oxide, silica, alumina and iron oxides (see Table 2-2) is then heated in a rotary kiln from temperatures of 1400 to 1450°C, to form clinker. The four main compounds that make up clinker are: tricalcium silicate (C_3S), dicalcium silicate (C_2S), tricalcium aluminate (C_3A) and tetracalcium aluminoferrite (C_4AF) (Addis, 2008).

Table 2-2 Oxides used in the manufacture of Portland cement (Addis, 2008)

Oxide	Chemical formula	Symbol	Derived from	Possible source	Content of clinker, %	Main function
Calcium	CaO	C	⇨ Calcareous material	Limestone, calcrete, chalk	63-68	Produce strength-giving compounds
Silicon	SiO_2	S	⇨ Argillaceous material*	Clay, shale	19-24	Produce strength-giving compounds
Aluminium	Al_2O_3	A	⇨ Argillaceous material*	Clay, shale	4-7	⇨ Fluxing agent in kiln
Iron	Fe_2O_3	F	Argillaceous material as above, or separately	Clay, shale, or natural iron oxide	1-4	⇨ Fluxing agent in kiln

* Some S and A will also be derived from ash of the coal used to fire the kiln.

After being left to cool, the clinker is then ground into a powder with gypsum which retards the setting of cement when mixed with water. The addition of gypsum is necessary to allow time for transport and placing of concrete. The cement is then ground with extenders such as fly-ash or silica fume based on the specification required and the final product is stored in silos or bagged (International Energy Agency, 2009). The bagged product is known to vary slightly in composition from bag to bag but the impact is almost negligible due to quality control measures implemented by the concrete plant (Ultratech concrete, 2014).

For the experimental tests conducted in this study, the bagged cement product was partially substituted with Coal bottom ash in volumetric substitutions ranging from 2.5- 40%. The use of bottom ash has been utilized in the past as a structural fill material where economically viable to transport to site. By exploring the use of coal bottom ash as a cement substitute in concrete, this will possibly lower the net CO_2 footprint of the concrete (Venter, 2014), decrease the amount of bottom ash diverted to ash-dumps and potentially reduce the cost of concrete within a feasible region. The feasible region will be based on transportation constraints to be investigated in this study. Bottom ash has also been shown to exhibit pozzolanic properties which could benefit concrete albeit being less reactive than fly-ash.

Section 2.2.1.2 will explain pozzolans further to build an understating of how they affect the conventional concrete mix.

2.2.1.2. Cement modification – Pozzolans

A pozzolan is a siliceous or aluminous material that does not possess any cementitious properties on its own but reacts with water and calcium hydroxide (slaked lime) to produce cementitious C-S-H compounds (U.S Department of Transportation Federal Highway Administration, 2011). Pozzolans can either be natural (volcanic ash, pumicite) or artificial/man-made (fly-ash, granulated blast furnace slag).

Figure 2-8: Left to Right :Class C fly ash, Metakaolin, Silica Fume, Class F fly ash , Slag, Calcined Shale (Girard, 2011)

The use of pozzolanic material is not a new technology and far predates the invention of Portland cement as evidence shows the use of lime-pozzolan mixtures by the Greeks to construct water storage tanks between 700BC-600BC (Wilson & Ding, 2007). The technique of utilizing pozzolans was also utilized later by the Romans who produced hydraulic binders by mixing lime with volcanic ash in the construction of the Pantheon (see Figure 2-9). The name pozzolan in fact comes from a town called Pozzuoli in Italy.

Figure 2-9 The Pantheon in Rome made using natural pozzolans (Rome on Segway, 2013)

In terms of contemporary pozzolan use, after the invention of Portland cement, pozzolans have been used as a partial cement binder substitute to reduce costly cement content and improve concrete properties such as durability, compressive strength and permeability. The pozzolans used today are in most cases man-made waste materials such as fly-ash and silica fume sourced from industry, because they are more readily available than natural pozzolans (Wilson & Ding, 2007). This is however country dependent, for example, South Africa utilizes coal fired power plants for it national grid and hence produces large volumes of ash. Commercial pozzolan usage generally ranges from 6-50 % cement substitution (Afrisam, 2009).

The basis for the pozzolanic reaction is an acid base reaction between calcium hydroxide ($Ca(OH)_2$) generated from the hydration of cement and silica (SiO_2). The product of the pozzolanic reaction is calcium silicate hydrate (C-S-H), which is the main compound responsible for concrete strength development (Juma, et al., 2010).

$$Ca(OH)_2 + SiO_2 + H_2O \rightarrow \text{C-S-H}$$

Slaked lime ($Ca(OH)_2$) used up in the pozzolanic reaction forms approximately 25% of hydrated Portland cement, however, it does not contribute to the strength or durability of the concrete. Therefore, by consuming the slaked lime to form calcium-silicate-hydrate, the concrete mix is strengthened, the packing of cementitious particle improved, workability increased, density is increased and porosity is decreased (Girard, 2011). When the pozzolan products fill pores and lower permeability, this also decreases the potential ingress of harmful ions such as chlorine and carbonate thus improving the durability of the mix (Chappex & Scrivener, 2012) (Kurama & Kaya, 2007).

Despite the benefits, the use of pozzolans also has its shortcomings. Strength gain is generally slow initially and benefits are only realised in excess of 28 days as this is when there is sufficient hydroxide available to react with from the products of the hydration reaction. Strength is also decreased and the cement is more permeable at the early age because of the reduction in clinker present in the mix due to cement volume substitution with the pozzolan (Bijen, et al., 1991).

Not all pozzolans behave in the same manner even if they are the same type of pozzolan, as properties can change based on where and how the pozzolan was obtained. The effect of pozzolans on the mix also varies with particle size, chemical composition and dosage of the pozzolanic material (Magistri, et al., 2011). Generally the finer the pozzolan particles the more reactive they are because there is a greater area of contact between cement minerals and water and hence a more rapid rate of reaction (Addis, 1986).

Bottom ash (BA) was selected as the pozzolan to be investigated for this study (*refer to sub-section 2.2.4*). Based on literature, by substituting cement with a pozzolan such as bottom ash there can be possible strength gains and reductions in cost as well as carbon footprint. Based on the general behaviour of pozzolans however, strength improvement over conventional concrete is usually realised after 28 days. The bottom ash used in this study was passed through a 1400μm aperture sieve to reduce the particle size and hopefully improve its reaction rate in accordance to comments made by Addis (2008) on the effects of pozzolan particle size. The BA will also be used in conjunction with HDPE and SCBF waste aggregates to study the resulting effects on selected concrete properties (strength, durability, workability etc.), which have not been investigated in prior research for the local context.

The cementitious binder as discussed, gives concrete its strength, however 60% to 75% of the concrete volume is made up of inert granular materials called aggregates that provide bulk and improve the dimensional stability of the mix. Section 2.2.1.3 will discuss aggregates and section 2.2.1.4, the modification of the aggregate component of the mix.

2.2.1.3. **Aggregates**

Aggregates are divided into coarse (Figure 2-10) and fine aggregates (Figure 2-11). Conventional coarse aggregates are obtained from crushed quarried rocks, boulders and cobbles. Fine aggregates (generally less than 4.75mm) are sourced from pits, rivers, lakes and the seabed (PCA , 2014). Use of virgin minerals as aggregates not only depletes natural resources but also causes a disturbance to the environment due to such mining activities (Econet, 2014).

Coarse aggregates form the basic structural component of concrete and fine aggregates (sand) aid in filling voids between the cement and coarser aggregates, thus reducing the porosity of the mix and locking the coarse aggregates together (Amsterdam, 2000).

Figure 2-10: Coarse stone aggregate (Ottawa Stone aggregates, 2014)

Figure 2-11 Fine sand aggregate (A&B Kerns Trucking and Stone, 2014)

Aggregates can influence both the fresh and hardened state in terms of concrete properties (Addis, 2008). The degree of influence on concrete properties such as durability, strength and weight of concrete were shown by Mamlouk and Zanieswki (1999), to depend on the properties of aggregates such as the absorption, size, surface texture, porosity, shape, unit weight and reactivity to chemicals.

The moisture state and water absorption properties of an aggregate have an impact on the W/C ratio which in turn effects porosity and strength of the concrete mix (Popovics, 1998). Usually a correction factor is used to adjust the mix for the moisture content of the constituents, but for the purpose of the study no water content correction was made in order to quantify the effect of varying moisture states and aggregate absorbency properties in terms of the tested concrete properties.

In terms of the compressive strength of aggregates, conventional concrete stone aggregates have compressive strengths of about 70 to 400 MPa. However, the compressive strength of the concrete is usually less than this, which shows that the strength of the concrete is largely controlled by the bond strength and characteristics of the cement aggregate interface rather than the strength of the aggregate itself (PCA , 2014). The elastic modulus and flexural strength of the concrete is also dependent on the properties of the aggregates. If all factors are kept constant, an aggregate with a higher elastic modulus should result in a mix with a higher flexural strength. This is because the higher modulus means a stiffer bulk matrix with increased rigidity (PCA , 2014). An aggregate with a high elastic modulus can also be a problem however with regards to shrinkage cracks, because when the concrete dries, the stiffer aggregate will offer restraint, building stresses in the matrix, thus causing cracks to occur.

The shape, size and surface texture of an aggregate are examples of properties that have an influence on both the fresh and hardened properties of concrete. In fact, if an aggregate has a rough texture, is elongated or angular, it will have better bond characteristics and concrete strength, but the mix can become more harsh and less workable (PCA , 2014). Considering the size of aggregates, smaller sized aggregates tend to require more water and cement due to increased surface area. This makes the mix less economical, but there can be improvements in strength due to larger surface areas for cement to bond with, thus lowering bond stress in the interfacial zone and increasing bond capacity (Caldarone, 2009).

It can be seen that the properties of an aggregate do have a significant influence on the properties of the concrete. Therefore, attempts have been made to not only enhance these properties, but create variations of concrete that are more cost-effective and consume less virgin minerals by modifying the types of aggregate used in the mix. Section 2.2.1.4 will describe what aspects of the conventional concrete mix will be modified in terms of aggregate substitution investigated for this study.

2.2.1.4. **Aggregate modification – Fibre / pellet substitution**

As mentioned, concrete is a brittle material that is weak in tension, which explains why it is susceptible to cracking (C&CI, 2013). To alleviate the lack of tensile strength and enhance other concrete properties, aggregates can be modified/ *"fibre reinforced"*. The fundamental concept of fibre reinforcing is that the fibres and the matrix are bonded together so they stretch equally under load, and stress is divided between the fibres and matrix. Fibres are intended to bridge internal cracks and transfer shear stresses until failure in either pull-out or breakage, therefore enhancing the tensile strength of the concrete. Aggregates can also be modified to cater for specific application demands such as adding plastics for thermal benefits (Elzafraney, et al., 2005). The modification of aggregates must not be assumed however as being strictly for performance enhancement only, because it is also a means in which virgin minerals can be reduced whilst creating an opportunity to make mixes of sufficient strength, more affordable or accessible to the populace depending on the type of aggregate substitution implemented. Some examples of the types of materials that have been used for concrete aggregate substitution are shown in Figure 2-12. The material types used in this study are highlighted in green and the specific materials used are highlighted in red.

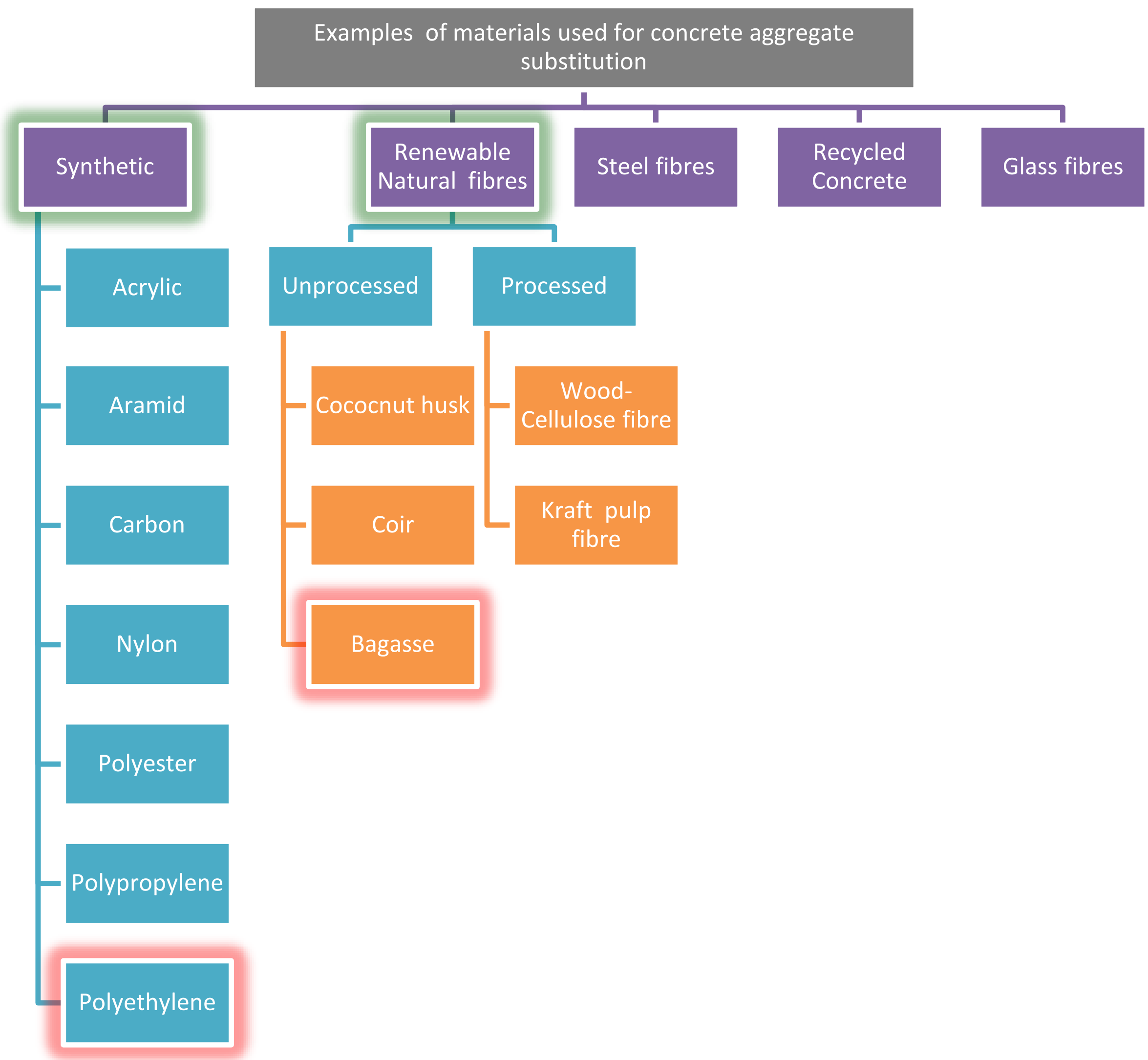


Figure 2-12: Examples of alternative aggregates used in concrete (C&CI, 2013)

As shown by figure 2.12 there is a variety of material types that can be considered when reference is made to fibre reinforcing or aggregate substitution. These range from low-modulus, malleable materials such as polyethylene and nylon, which do not improve strength but help control cracking, to high strength, high modulus fibres such as steel and glass that impart strength and stiffness to the mix. The concept of reinforcing a brittle matrix is not a contemporary revelation as the use of fibres in composite materials has been implemented over many centuries, for example, horsehair in plaster and straw in sun-baked bricks (Addis, 1986).

For the purpose of this study, interest in using recycled or renewable materials in construction to reduce virgin mineral consumption and waste has led to the investigation being based on a synthetic aggregate in the form of recycled high-density polyethylene pellets and a renewable natural fibre in the form of sugarcane bagasse. These types of materials (natural & synthetic aggregates) in general will be discussed further, with a more in-depth investigation of the materials used specifically for this study in sections 2.2.2 and 2.2.3 respectively.

Synthetic aggregates are man-made aggregates originated from research and development in the textile and petrochemical industry (C&CI, 2013). Synthetic aggregates can either be micro-fibres (<0.3mm), macro (>0.3mm) fibres, shredded or pelletized. Micro-synthetic fibres are used in cement based applications for plastic shrinkage control, impact protection and anti-spalling, but are strictly non-structural fibres and should not be used to replace wire mesh or any structural elements (FRCA, 2007). Macro-synthetic fibres such as polypropylene and polymer blends have been used as an alternative to steel fibres where post-cracking flexural strength was required. The use of pelletized synthetics in general, has mainly been as a light-weight filler material as opposed to a means to limit crack propagation and shrinkage as with fibres (FRCA, 2007). The use of pelletized plastic synthetics are however not as well documented as the fibrous variants for concrete use, hence its selection for this study to expand knowledge on the material.

Figure 2-13 : Synthetic plastic fibre

In pelletized form the aggregate properties that are assessed are generally particle size, density and surface characteristics (Saikia & de Brito, 2012) which are similar to those considered for conventional aggregates *(refer to section 2.2.1.3)*. Unlike conventional aggregates, however, variations in moisture states do not have a significant effect on the resulting concrete properties when plastic synthetic aggregates are used, due to the hydrophobic nature of the material (Saikia & de Brito, 2012). Even though variations in plastic aggregate moisture state may not result in changes to concrete properties, as stated by Saikia & de Brito (2012), the hydrophobic nature mentioned could be a problem in terms of bonding with the cement matrix due to poor hydration activity in the interfacial zone. The use of synthetic aggregates has therefore been limited to low volumetric substitutions generally between 0.4%-0.8%, so the strength of the concrete remains unaffected and costs are minimized. This study investigated a range of substitutions, from low to high, to expand on the knowledge covered in past research and establish if there was a critical volume where performance peaked over the range of 2,5%-40% volumetric substitutions.

Figure 2-14: Recycled HDPE pellets

The other modification to the conventional mix will be the use of renewable natural fibre substitution.

Natural fibres are defined as substances that are produced by animals or plants like sisal, coconut husk or bagasse (Bcomp, 2014) (Fiber Reinforced Concrete Association, 2013). Natural fibres can either be processed like wood-cellulose composites and kraft pulp fibres or unprocessed like bagasse, sisal and coir. Unprocessed fibres are a renewable resource and have the benefit of being less costly and not requiring sophisticated machinery as required by processed fibres. Unprocessed Sugarcane bagasse fibre, which is abundant in certain areas locally, was used for this study and will be discussed further in section 2.2.2.

As mentioned, fibres are used mostly to mitigate crack propagation, therefore when fibres in general are used for concrete it was observed by Aziz, *et al*. (1984), that they should:

- have a relatively higher tensile strength than the matrix;
- have a reasonable bond interface with the matrix;
- have a high elastic modulus which is elastically compatible with the low–modulus matrix

The characteristics mentioned are important because the strength of the binder-aggregate bond and transfer of stress between the matrix and fibres have a significant influence on the performance of the concrete (The Constructor, 2014). Also, elastic compatibility is significant because if there is a large variance in elastic moduli between the matrix and the aggregate, cracking can occur from built up stresses exerted on the matrix because of the increased resistance offered by the aggregate (C&CI, 2013). On the other hand, if the elastic modulus and strength of the aggregate are low, like nylon or the bagasse used in this study, then they are not likely to impart any vast strength improvement on the concrete mix, but may still reduce the effect of cracking albeit to a lesser degree than a stronger, stiffer material such as steel fibre (The Constructor, 2014).

In terms of natural fibres, the most significant properties that affect the concrete mix are, fibre length, volume fraction and absorption.

Fibres can be classified as either long or short in terms of length. Short fibres, which are discontinuous, have an aspect ratio (length/diameter) of 20-60. Long fibres which are referred to as continuous, have an aspect ratio of 200-500. The longer the fibre, the better the stress transfer from the matrix to the fibre. However, longer fibres have been shown to decrease concrete workability due to increased surface area and interlocking (Kalpakjian & Schmid, 2001). Long fibres also have the tendency to clump together, which leads to a decrease in workability and results in a non-uniform distribution of fibres. A uniform distribution of fibres is essential to distribute stresses effectively, thus improving the resistance to cracking, impact and shock loading (Aziz, et al., 1984).

Both long and short fibres are intended to limit crack propagation. Short fibres are aimed at bridging micro-cracks during initial tensile loading. Long fibres are aimed at bridging macro-cracks after micro-cracks propagate and merge, as shown in Figure 2-15.

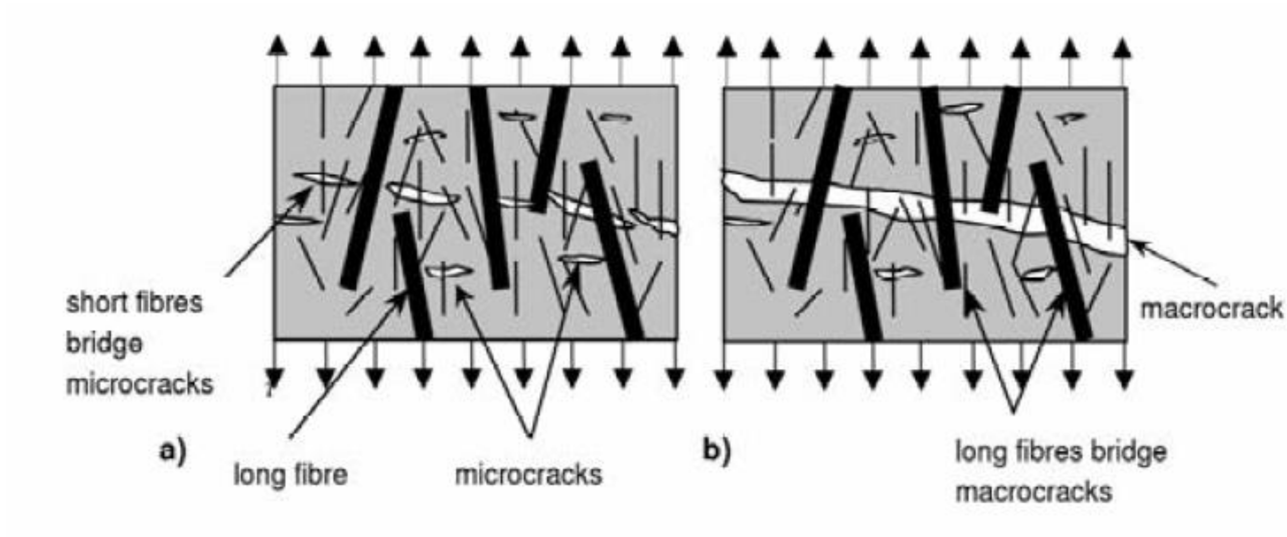


Figure 2-15: How short & long fibres limit crack propagation (Vandewalle, 2006)

In terms of volume fractions, unprocessed natural fibres have shown improvements to concrete properties with volumetric substitutions above 3%, however durability issues have been noticed in the long-term. For natural fibres, the durability issues faced over time are caused by fibre deterioration and softening due to exposure to alkaline pore-water (Daniel, 2002). One way to remedy this is by adding a pozzolan to react with lime and reduce the alkalinity of the pore water.

Absorption of natural fibres is generally higher than the conventional aggregates they substitute, ranging from 50% to as much as 180% (C&CI, 2013). Depending on the degree of absorption concrete strength can be potentially reduced if moisture is absorbed to the extent that there is insufficient water to adequately hydrate the cement (National Research Council, 2000). Alternatively, the decrease in W/C ratio may possibly increase the strength of the concrete (Gardiner & MacDonald, 2013), however the high absorption of natural fibres would also decrease slump of the mix, which can make the concrete less workable. For the purpose of this study, the mix water content remained constant to quantify such effects when using the waste materials selected, in terms of the changes to the tested concrete properties (strength, workability, durability etc.).

With the overview of conventional concrete and how it was modified discussed, the next section (2.2.2) will begin the in-depth investigation into each of the waste materials to be used for this study by providing an informative background for each material. These backgrounds critically evaluated past research and incorporated information from the explanation on conventional concrete, by relating material properties to those mentioned in subsections 2.2.1.2 and 2.2.1.4.

2.2.2. Sugarcane Bagasse (Organic waste/agro waste)

2.2.2.1. Background

Fibre reinforced concrete is a concrete mix that contains dispersed, randomly orientated, unconnected fibres of metallic, mineral or organic materials (The Free Dictionary by Farlex, 2003). *(Refer to section 2.2.1.4 for a general background on fibre reinforcing)*. Sugarcane bagasse fibre (SCBF) was the renewable natural fibre selected for this study.

SCBF is a natural organic solid (Mamlouk & Zaniewski, 1999) sourced from sugarcane, a carbon neutral (*emissions equal to energy generated*) member of the grass family, that is cultivated mainly in tropical and sub-tropical regions (Aziz, et al., 1984). SCBF is the residual lignocellulosic fibre waste bi-product of the sugar cane juice extraction process (*refer to section 2.2.2.2)*. Elementally, by weight percentage, bagasse is largely comprised of long chains of carbon and hydrogen molecules and botanically it is made up of mostly cellulose (C6 H10 O5) n *(*refer to Table 2-3*)* which is “n” repetitions of linked D-glucose units (C6 H10 O5) (My Organic Chemistry, 2014) *(see figure 2-16)*.

Table 2-3 :Bagasse composition (Bilba, et al., 2003)

Element Composition	Weight %	Botanical composition	Weight %
Carbon (C)	45.5	**Cellulose**	41.8
Hydrogen (H)	5.6	**Hemicellulose**	28
Oxygen (O)	45.2	**Lignin**	21.8
Nitrogen (N)	0.3		

Figure 2-16 shows the chemical structure of cellulose, hemicellulose, lignin and pectin. Cellulose is a semi crystalline polysaccharide that is the load-bearing component of the plant matrix and is responsible for the hydrophilic nature of bagasse. Cellulose is bound together by lignin, which is comprised of mainly aromatics, has little effect on moisture absorption and is analogous to an epoxy resin that gives the plant matrix rigidity and stiffness (Lignoworks, 2014). Hemicellulose binds pectin and cellulose, is amorphous in nature, and is partially soluble in alkaline solutions (Westmann, et al., 2010). The susceptibility of lignin and hemicellulose to alkali attack can be a problem in concrete due to the alkaline nature of pore water, which could weaken the bagasse fibre structure and hence the concrete mix. The addition of pozzolans have been shown in past research to reduce the alkalinity of the concrete mix and possibly reduce the chance of alkali attack and thus preserve strength properties (Onésippe, et al., 2010) (Gram, 1986). The impact on concrete strength when mixing a pozzolan (Bottom Ash) in a mix with SCBF was investigated in this study. However, alkali attack and the effects on pH as a result of possible alkali reductions with a pozzolan were not tested in this study. This is recommended for future research, to be investigated in conjunction with tests on reinforced concrete that identify changes to the passivation layer formed around re-bar due to pH variations.

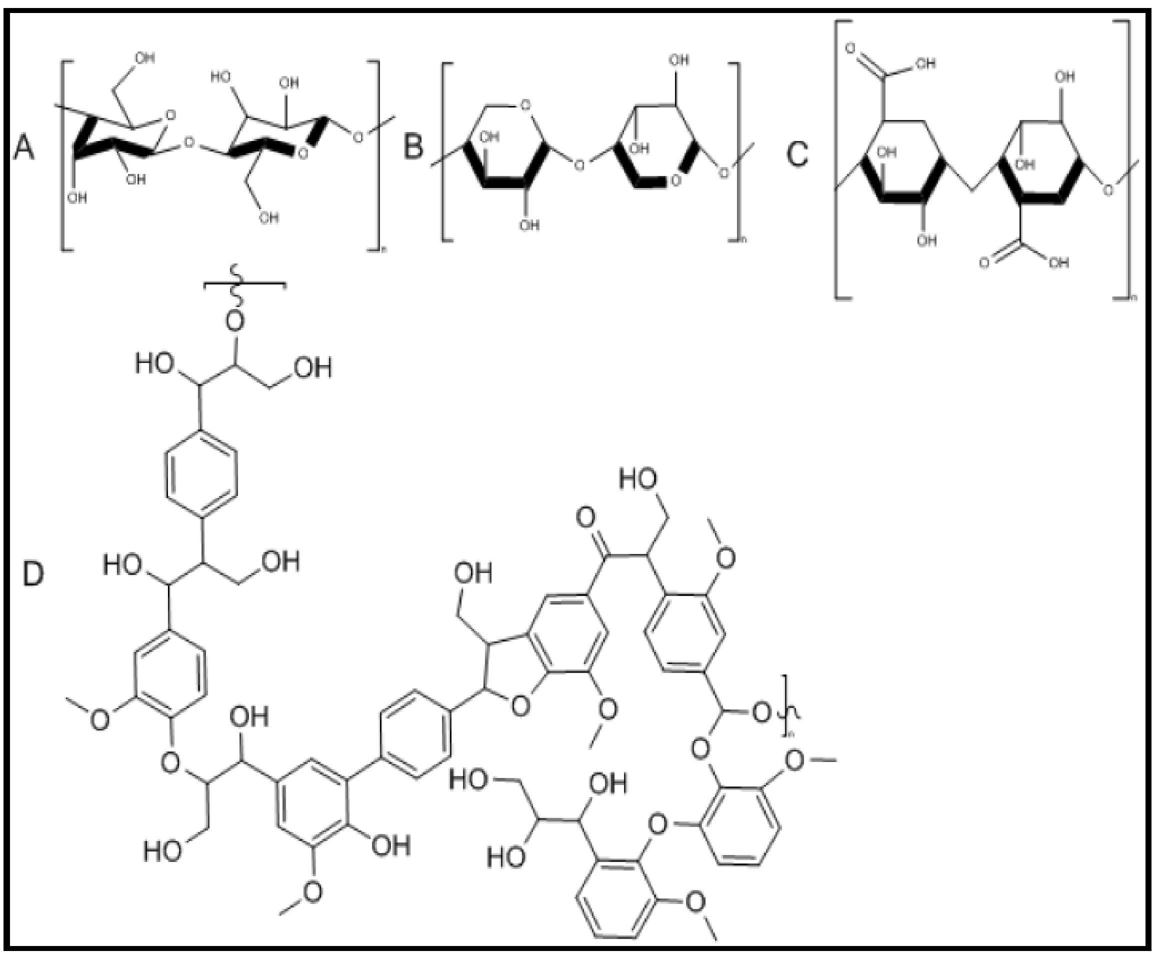


Figure 2-16: (A) cellulose, (B) hemicellulose, (C) pectin, (D) lignin (Westmann, et al., 2010)

R.N Swamy (1990) has also shown that fibres with a high cellulose content generally have a low modulus of elasticity, high moisture absorption and are susceptible to biological attack. Based on literature, the comments on moisture and elastic modulus made by RN Swamy

(1990), have been shown to apply to SBCF as shown by Table 2-4 on page 57. Refer to section 2.2.2.3 for the critical review on material properties which states how these properties may potentially influence the concrete mix.

Sugarcane bagasse is widely available in the province of KwaZulu-Natal and most of the deleterious solids are removed in the refining process by the mills, hence no pre-treatment would be required (Stephens, 1994). In 2011 South Africa produced approximately 6.5 million metric tons of bagasse fibre (UN Data, 2013). This abundance of SCBF in certain areas locally (refer to Figure 2-17*)*, coupled with the benefits of reducing virgin mineral consumption with a material that is renewable, emphasises the concept of using bagasse fibre as an aggregate substitute in areas within feasible proximity to mills.

Figure 2-17 provided by the Sugar Mill Research Institute (SMRI), shows the locations of the 14 mills in South Africa. It can be seen that the mill locations are only on the eastern half of the country so this may limit the viable market in terms of the restrictions imposed by transport costs. The scenario analysis was based in KwaZulu-Natal, so if transportation costs were in excess for the scenario then it would obviously be unfeasible in areas exceeding the scenarios distance from the mill (refer to section 4.12 for economic analysis).

Figure 2-17 Sugar mills of South Africa (SMRI, 2011)

2.2.2.2. **Method of processing**

Sugarcane bagasse is a bi-product of the sugar making process. It is the fibrous material that is left behind after the juice is extracted from the sugar cane. The process of obtaining bagasse starts with sugarcane stalks being harvested from the fields and transported to the sugar mill. The stalks are then shredded before the juice is extracted through repeated crushing and washing (milling) or repeated washing with a final squeeze to dry the fibre (diffusion). The Sezela mill, where the samples were taken from for this study, utilized the diffusion process. Bagasse fibre from the diffusion process has to be de-watered in the mill before being sent away to the boilers or for other bi-product purposes. Figure 2-18 shows a simple illustration of the process described (Huletts, 2013).

Figure 2-18: Process diagram of sugar milling

2.2.2.3. Critical review on the literature about Sugarcane bagasse fibre material properties

Material property values obtained for SCBF are shown in Table 2-4.

Table 2-4: SCBF property values summarised from literature

Property	Value	Reference
Relative Density	1.2.- 1.3 (unit less)	(C&CI, 2013)
Specific heat capacity	460 J/kg °C	(Shrivastav & Hussain, 2013)
Calorific value	2200 kcal/kg	(India Solar, 2013)
Fibre length	1.7mm (pith)-300 mm (rind)	(Bentur & Mindess, 1990), (Chandra, 1998)
Fibre diameter	0.2-0.4 mm	(C&CI, 2013)
Ultimate tensile strength	180-290 MPa	(Addis & Owens, 2001)
Modulus of elasticity	15-19 GPa	(C&CI, 2013)
Water absorption percent (%)	70-186%	(C&CI, 2013) (Basu, 2010)

Bagasse is comprised of both long outer rind or stalk fibres and short internal pith fibres (Asagekar & Joshi, 2014). This is why there is a relatively large range of fibre length from 50-300mm. The length of fibres do have an impact on the properties of concrete. Short natural fibres used up to volumes of 16% have been shown to improve tensile strength and reduce micro-crack propagation (Wang, et al., 2000). Long natural fibres have been shown to improve tensile strength and reduce macro-crack propagation, however, long natural fibres have also been shown to decrease the workability of the concrete mix (Vandewalle, 2006). The tensile strength and workability test carried out in this study investigated the comments made by Wang, *et al*.(2000) on tensile strength and Vandewalle (2006) on workability in terms of unprocessed fibres (combination of long and short fibres).

Figure 2-19 : Rind (left), Pith (right) (Rabelo, et al., 2008)

Even though the tensile strength of the bagasse fibre is not as high (avg.160 MPa-*see* Table 2-4) as steel fibres (500-2000 MPa (C&CI, 2013)*)*, it is still much higher than the tensile strength of the hardened cement matrix (1.4-7 MPa (Cheremisinoff & Cheremisinoff, 1993)), therefore improvements may still be noticed depending on how well load is transferred from the matrix to the fibres through adhesion (Cheremisinoff & Cheremisinoff, 1993).

The relatively low modulus of elasticity of the SCBF means that SCBF will not offer as much resistance to deformation as the stone it partially replaces and hence this may possibly lower the elastic modulus of the mix (Lamond & Pielert, 2006). *Refer to section 4.8 for elastic modulus investigation.*

The high absorption of bagasse fibres is another notable property. Bagasse is hydroscopic because it comprises of hydroxyl ($-OH^-$), amino ($-NH_2$) and carboxyl ($-COOH$) groups which are hydrophilic (Shibata, et al., 2014). The high water absorption of SCBF means that if excessive water is absorbed then concrete strength may decrease due to inadequate cement hydration (National Research Council, 2000). Alternatively, the reduction in moisture may reduce the W/C ratio of the mix thus potentially reducing porosity and increasing strength (Gardiner & MacDonald, 2013). However, a decrease in slump would be expected. A fixed water content was therefore used in this study to quantify the effect of absorption on the compressive strength and slump of the respective concrete mixes. The high absorption would also have an effect on shrinkage; however, the analysis and testing of shrinkage properties along with, creep and cracking behaviour would constitute a separate MSc research topic such as that carried out by Petra Cornelia Gaylard (Gaylard, 2011) and were beyond the scope of this study.

In terms of specific heat, because 65-70% of concrete comprises of aggregates, the thermal properties of the concrete are to a degree dependent on the properties of the aggregates (Lamond & Pielert, 2006). The specific heat of coarse aggregate is 837 J/kg °C (Harder, 2008) and SCBF is 460 J/kg °C *(see* Table 2-4*),* therefore the specific heat of concrete with SCBF in theory could be less than that of conventional concrete. SCBF has however been shown to increase permeability (Omoniyi & Akinyemi, 2013) and if this is the case, more voids could be present in the concrete and this may increase specific heat of the hardened concrete because the specific heat of air at 20°C is 1005 J/kg °C, (Engineering toolbox, 2014). The significance of fibre volume on specific heat will be assessed in this study by testing varying proportions of SBCF in the mix. Refer to section 4.7 for the specific heat analysis.

2.2.2.4. **Past research and applications**

Sivarja *et al.* (2010) conducted research which compared a conventional concrete mix with concrete modified with sugarcane bagasse fibres. The researchers limited their study to a low-volume substitution of 1.5% of aggregate to reduce the effect of balling and the negative impacts of fibre length on the workability of the mix. However, a reduction in workability of 29% from 110 mm to 78 mm was still noticed. Considering the researchers limited the quantity of fibres in the mix to reduce the effect of fibre length causing particle interference, the reduction in workability was most likely attributed to the moisture absorption properties of the SCBF, as stated in natural fibre research by Jorillo & Shimizu (1992) and Silva & Suely (1984). The effect of moisture absorption was evaluated in this study by comparing concrete with SCBF at varying moisture states, keeping water content unmodified to indicate the extent of variation in tested concrete properties *(Refer to section 4.11 for the investigation into moisture states)*.

In terms of the mechanical concrete properties tested by Sivarja *et al.* (2010) after 28 days of curing, the specimens that contained SCBF improved in compressive strength by 4%, flexural strength by 18% and splitting strength by 37% when compared to the conventional concrete mix. The increased compressive strength, as with workability changes, could have been attributed to moisture being absorbed from the mix hence decreasing the W/C ratio. The variation in compressive strength was less than the improvements to tensile properties, indicating that fibres enhance certain qualities of concrete more than others and, as stated in section 2.2.1.4, are generally included in the mix to bridge internal cracks and enhance tensile properties.

The improvements to flexural and splitting strengths showed that SCBF at a low volume fraction (1,5%) performed well as crack arrestor by enhancing the tensile properties of the concrete. This study will assess not only low-volume substitutions (2.5%) but also higher volumes (5%, 10%, 20%, and 40%) to investigate if the beneficial effects on tensile properties are proportional to fibre quantity with regard to SCBF (*Refer to sections 4.6.2 and 4.6.3 for the investigation into tensile properties in this study)*. The elastic modulus of the SCBF mix at 1,5% volume substitution, was also increased by 15% when compared to the conventional mix. This meant that even though the elastic modulus of SCBF is relatively low compared to fibres such as steel *(refer to section 2.2.2.2)* it can still offer a certain degree of resistance to deformation under load and thereby stiffen the mix.

It was concluded by Sivarja, *et al*. (2010) that the introduction of SCBF was beneficial to the concrete matrix at low-volume substitutions, but no attempt was made at exploring low to higher volume substitutions to identify if a peak volumetric substitution value exists. The task of investigating the possibility of a peak volume was undertaken by the author of this study.

Ramirez-Coretti (1992) did a study on the effect of natural fibres (rice straw, sugarcane bagasse, banana racquis and coconut husk) on cement mixes looking at the tensile strength after 28 days. Ramirez-Coretti hypothesized that the use of discontinuous, natural fibres, randomly distributed in the cement matrix would be able to improve the tensile strength and inhibit crack propagation through the plain cement mix. It was concluded from the test results that volume substitutions of between 3% and 7% of aggregate would be the best proportions to use, however improvements in strength properties were minimal. The study by Ramirez-Coretti supports findings by Sivarja, *et al*. (2010) that at low volumes, the crack-arresting properties of natural fibres are beneficial, however, in their studies higher volume substitutions were not investigated.

Research by Racines & Pama (1978) was therefore reviewed because the effect of high volume natural fibre substitutions of 10%, 20% and 30% of aggregate in the concrete mix were investigated, instead of low-volume substitutions investigated by Ramirez-Coretti (1992) and Sivarja *et al.* (2010). Racines & Pama showed that compressive strength was more sensitive to increases in volume substitution than flexural strength. Compressive strength decreased by almost 50% for each 10% increase in SCBF volume but dropped around 25% in flexural strength for every 10% increase in SCBF volume when compared to a conventional mix. The variance in sensitivity could possibly be explained by the fact that the introduction of fibre caused substantial voids to form at high volumes thus reducing compressive strength, however, fibres still imparted some degree of crack resistance which aided tensile strength. Due to the fact that all high volume substitution mixes showed decreases in strength and considering research by Ramirez-Correti, (1992) and Sivarja, *et al*. (2010), it can be said that the best results are obtained from low-volume substitutions. The threshold/critical volume at which strength peaked was not investigated by any of the researchers mentioned and therefore establishing if a potential critical volume existed for the respective waste concrete mixes within the range of 2,5% - 40% substitution, formed part of this study.

Bentur & Mindess (1990) stated that natural cellulose fibres are generally sensitive to changes in moisture content which can have an effect on mechanical properties of the fibres. When natural cellulose fibres are wetted, they swell and the fibres lose stiffness. The swelling and shrinking of fibres under wetting and drying cycles may also lead to variations in bond strength between the fibres and the cement matrix. In relation to this study, the absorbency potential of the SCBF used is relatively high *(see section 2.2.2.3)* therefore the fibres could readily take in water which may result in a loss of stiffness as mentioned. With cognizance of the influence of moisture content from research by Bentur & Mindess (1990), this study involved the testing and comparison of specimens using dry fibres (maximum absorbency potential) with specimens using fibres at natural moisture state, in terms of changes in compressive strength.

In terms of the possible effect on the modulus of elasticity of the concrete mix due to a possible reduced stiffness, findings stated by Bentur & Mindess (1990) contrasted those of Sivarja et.al (2010), which showed an increase in elastic modulus with fibres introduced to the concrete mix. This may have been due to variations in properties of the natural fibres used in the respective studies, therefore, the elastic modulus of concrete with fibres introduced was compared to a mix with no fibres to draw conclusions on the effect of fibres on the modulus of elasticity for the local context *(See section 4.8 for modulus of elasticity analysis).*

Durability is a key factor when natural fibres are considered for use in concrete. Omoniyi & Akinyemi (2013) conducted research on the permeability characteristics of bagasse fibre reinforced concrete using volume substitutions of 1%, 2%, 3%, 4% and 5%. Permeability plays an important role in long-term durability as it dictates the rate at which water and other substances such as sulphates and chloride ions penetrate the concrete. This is relevant to the rate of corrosion under wetting and drying cycles for reinforced concrete (Hong, 1998). Omoniyi & Akinyemi (2013) established that by using lower-volume substitutions of 1-3 %, the durability of fibre reinforced concrete was increased because the permeability was decreased by a maximum of 8% at 3% volume substitution (Omoniyi & Akinyemi, 2013). However, beyond 3% substitution, permeability increased by as much as 556% at 5% volume substitution *(Refer to Figure 2-20).*

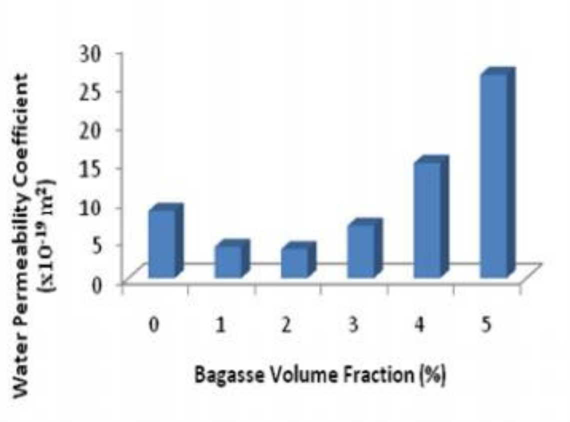


Figure 2-20: Comparison between Water Permeability and Bagasse Volume fraction (Omoniyi & Akinyemi, 2013)

The increase in permeability above 3% volume fraction could have been attributed to the balling of fibres at higher volumes, which clumped together leaving voids in the mix. This relates to conclusions by Omoniyi & Akinyemi (2013), that the permeability of a concrete mix is directly related to the amount of void space interconnections present. This study expanded on findings by Omoniyi & Akinyemi (2013), by testing concrete mixes for oxygen permeability, water sorptivity and chloride conductivity in the durability analysis (refer to section 4.9). Both oxygen and water are equivalent in terms of evaluating permeability, but on different scales, so comparisons were made between this study and findings by Omoniyi & Akinyemi (2013) in terms of the percentage change in permeability relative to a control mix. The oxygen permeability test was used in this study as it is more reliable than the water permeability test, because oxygen flow through a sample does not react with hydrates or alter the pore structure (Prajapati & Arora, 2011) (refer to section 4.9 for the durability index analysis in this study).

Research by Gram (1986) and the Swedish Cement and Concrete Research Institute also investigated the durability of concrete which incorporated natural fibres. Gram's results showed that durability was reduced because of the introduction of the fibres. The reasoning however for the reduction in durability was different to the reasoning provided by Omoniyi & Akinyemi (2013). Gram (1986) stated that the reduction in durability was caused by the fibres becoming brittle from exposure to the alkaline cement. The mechanism of degradation was the dissolution of lignin and hemicellulose in the alkaline pore water of the concrete. It was theorised that this degradation could be reduced by substituting cement with a pozzolanic material in order to reduce the alkalinity of the cement matrix. In addition to exploring the durability of concrete with natural fibres using the oxygen permeability, chloride conductivity and water sorptivity tests, this study compared mixes with and without a pozzolanic material (bottom ash), where changes in permeability indicated whether less reactive pozzolans such as bottom ash possibly improved durability properties of concrete with natural fibres, as researched by Gram (1986).

Bilba *et al.* (2003) examined the hardening properties of concrete with bagasse fibres added. An increase in setting times was noticed with the introduction of bagasse, which was possibly due to the raw bagasse containing sucrose, which is known in the construction industry to have a natural retarding effect on concrete. This retarding effect creates an extended period of workability for fresh concrete (Thomas & Jennings, 2014), however, this also means there is a delay in setting times and strength development. Formwork and material would subsequently stand for longer durations and this would increase costs. A Differential Scanning Calorimeter (DSC) was not available at the time of the study as used by Bilba *et al.* (2003) to carry out a meaningful investigation into hydration temperatures and concrete hardening. However, this study did investigate the 7 and 28 day compressive strengths to provide comments on strength development post-initial set, relative to a control mix with no fibres added.

2.2.2.5. **Summary of the possible benefits of using SCBF in concrete**

The main benefits of using SCBF in concrete, based on the findings from the available literature, are the following:

- SCBF is a renewable potential resource that is carbon neutral (Aziz, et al., 1984).
- SCBF is biodegradable making it an attractive option in terms of environmental protection (Eco Kloud, 2012).
- SCBF can potentially be used as a crack arrestor and hence improve the tensile strength of concrete based on past research by Sivarja, *et al*. (2010), which showed increases in flexural and splitting strengths as well as reducing permeability for low volumetric substitutions between 1-3%.

2.2.2.6. **Summary of SCBF limitations in concrete**

The main limitations of using SCBF in concrete, based on the findings from the available literature, are the following:

- The susceptibility of lignin and hemi-cellulose to degrade in alkaline environments such as the concrete matrix can potentially weaken the concrete mix.

- The supply of bagasse in South Africa is limited by the location of sugar mills which are predominantly situated on the eastern half of the country as shown by Figure 2-17.
- The supply of SCBF is also limited by sugarcane crop yields and the amount of bagasse regarded as surplus by the mill.
- Possible decrease in workability due to long fibre length (Vandewalle, 2006).
- Possible reduction in elastic modulus of the concrete because SCBF has a relatively low elastic modulus (Lamond & Pielert, 2006).
- The absorption properties of SCBF can negatively affect concrete in terms of reduced workability and increased permeability (Gram, 1986).
- Possible presence of residual sugar and lignin hydrolysis may cause retardation of hydration and hardening of concrete (Bilba, et al., 2003).
- At high volume substitutions (>10%), compressive and flexural strength are known to decrease (Racines & Pama, 1978) and permeability has been shown to increase (Omoniyi & Akinyemi, 2013).

2.2.2.7. **Concluding remarks on SCBF literature**

Sugarcane bagasse was reviewed as a potential renewable resource for concrete applications in the local context. It is a material that is mainly used for power generation, but this creates pollution (Eco Kloud, 2012). Therefore, investigating alterative markets such as the construction industry can lead to a potential waste reduction for the mills whilst providing a resource that is renewable (Eco Kloud, 2012).

Past research has shown that the volume percentage of natural fibres has a significant influence on the performance of the natural fibres in the concrete mix. Research utilizing low-volume substitutions (Sivarja, et al., 2010) have generally produced more favourable results than higher volume substitutions (>10%) (Racines & Pama, 1978). By testing low–high volume substitutions (2, 5%, 5%, 10%, 20% & 40%) this research attempted to establish an idea of the critical volumetric substitution at which performance of SCBF concrete peaked.

Past research has also shown that the high absorption and durability are issues with regards to the use of SCBF, which has a tendency to degrade in alkaline environments and introduce void into the mix. The effect of SCBF on permeability was of particular concern, as noted by Omoniyi & Akinyemi (2013), and hence the permeability of the critical volume and mixed waste concrete specimens were investigated using the durability index testing procedure in this study (see section 4.9). Some aspects from past research are debatable such as the effect of SCBF on elastic modulus. Bentur & Mindess (1990) stated the reduction in stiffness can potentially reduce elastic modulus of the concrete whereas Sivarja, *et al*. (2010) showed an increase in elastic modulus with fibres introduced to the concrete mix. Therefore, elastic modulus was investigated in this study to clarify the matter (see section 4.8).

The economics of SCBF was evaluated with a cost and scenario analysis in this study to identify how significant an impact on cost, if any, is made by using SCBF substitution. Due to SCBF being sourced from the mills, the cost to transport the material to site may be a significant factor and was considered in the economic analysis (see section 4.12).

From the onset, the amount of possible limitations did outnumber the amount of possible benefits to using SCBF in concrete. The significance of these limitations on the viability of SCBF were however assessed in this study due to its potential to be used as a renewable resource that can possibly improve the tensile strength of concrete.

2.2.3. HDPE Recycled plastic pellets

2.2.3.1. Background

The global population is increasing year by year and with it, the production of waste materials such as plastic (polymer). Plastics degrade very slowly, in excess of a human lifetime. This poses a problem for waste disposal if recycling is not utilized, as traditionally, plastic can either be buried in a landfill or burnt. Both of these options have negative environmental effects. Recycling is therefore a key component to material sustainability because it reduces the use of virgin minerals, associated mining activities and manufacturing processes.

The plastics industry is defined as a priority sector by the South African government, contributing 4% to overall manufacture in South Africa and 0.4% of the country's Gross Domestic Product (GDP) (Department of Trade and Industry, 2014). Figure 2.20 below extracted from the report compiled by Plastics S.A. (2012), titled "*The journey continues*", shows that South African consumption of virgin plastics is on the rise. This is possibly due to the increase in population. This can be seen by Figure 2-21, which shows the plastic consumption in 2000 being 900 000 metric tons compared to 2011 where it is 1 300 000 metric tons (44% increase). However, between 2007 and 2011 this trend has been stabilizing, indicating the possible impact of the recycling industry.

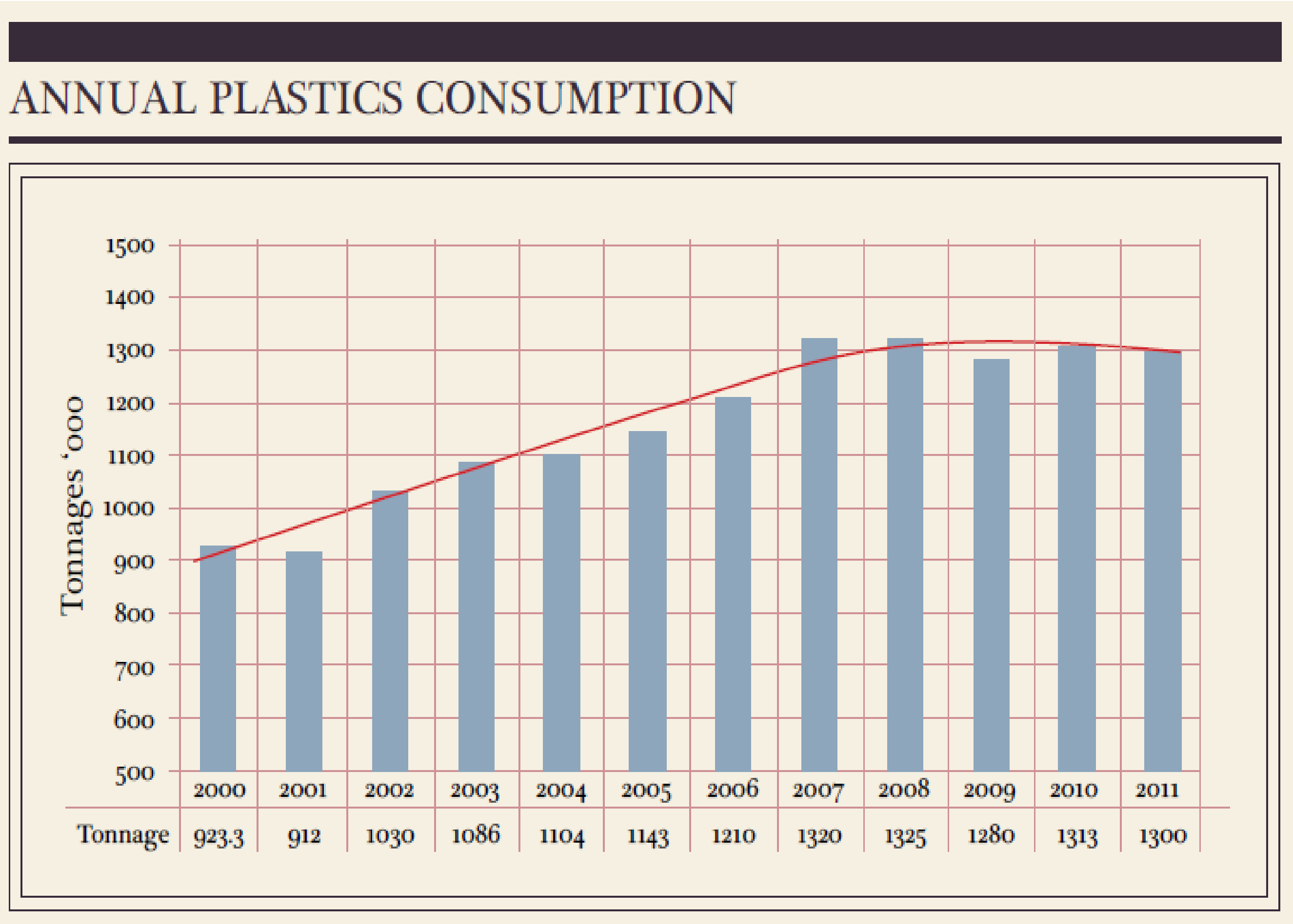


Figure 2-21: South African annual plastic consumption (Plastics S.A., 2012)

The increasing plastic consumption creates an opportunity for plastics to be used as a resource in the construction industry as a synthetic aggregate *(see section 2.2.1.4)* to aid in waste reduction. Plastics utilized for construction have benefits such as resistance to corrosion and chemical resistance because they are inert, but they also have poor physiochemical bonding with cement paste (Malhotra, et al., 1994). There are various type of plastic created which will now be discussed.

Plastics (Polymers) are long chained molecules formed by chemical processes between monomers. The main classifications of polymers are thermoplastics, thermosets, natural polymers (such as lignin) and elastomers (Hollaway, 1990).

- Thermoplastics are plastics that can be melted for recycling and consist of long chains of linear polymerized molecules. The linear molecules that form thermoplastics are held together by strong covalent bonds but these linear chains are not inter-connected. Thermoplastics can be classified as having either an amorphous random structure like polymethylmethacrylate or a crystalline ordered structure. Polypropylene is a non-orientated thermoplastic which can be drawn into an elongated form to increase strength and stiffness (Hollaway, 1990). Other examples of thermoplastics are polyethylene and polyamide.
- Thermosets are produced when molecular chains cross-link to form solid material (Hollaway, 1990). The cross-linked molecules of thermosets mean they cannot be melted to liquid phase by heating. Thermosets such as polyester, are formed when long polymerized molecules are cross-linked at room temperature or under heat and pressure. Examples of thermosets are melamine, silicone and polyurethane (Hollaway, 1990).
- Elastomers are a type of polymer and comprise of long-chained molecules that are coiled and twisted randomly and held together by chemical bonds. Rubber is an example of an elastomer (Hollaway, 1990).

Figure 2-22: Types of polymers

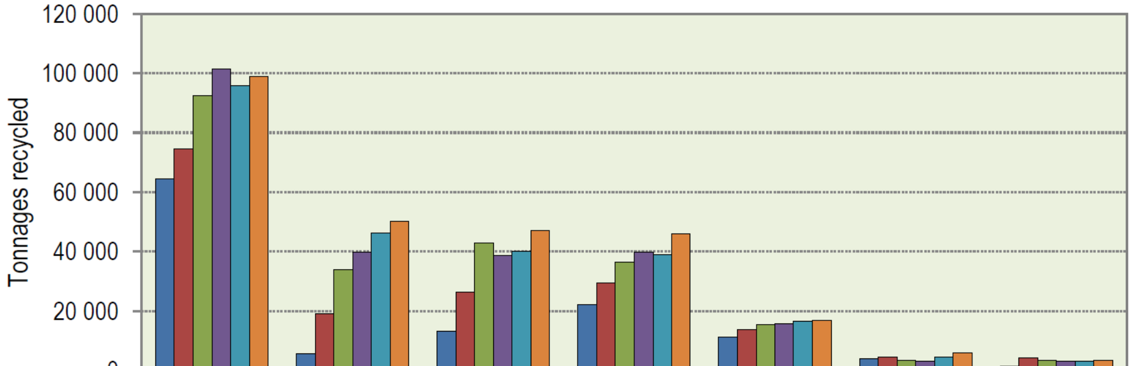
The type of plastic (polymer) selected for this study was a thermoplastic called High-Density Polyethylene (HDPE). Polyethylene (PE) was invented by American chemist Carl Shipp Marvel in the1930's (Gabriel, 2013) . In 1953 Karl Ziegler and Erhard Holzkamp invented high-density polyethylene with Ziegler winning the Nobel Prize in Chemistry for his invention in 1963.

HDPE has many modern day applications such as: drainage pipes, bottle caps, chemical container bottles, milk bottles and shopping bags. HDPE is a semi-crystalline thermoplastic comprised of carbon and hydrogen atoms. Virgin HDPE is made from petroleum (ethylene). Petroleum is already a precious resource and the more recycling that is encouraged, the greater the reduction in petroleum used for plastic. The type of HDPE used accoding to the Society of Plastics Industry (SPI) numbering system for plastic is shown in Figure 2-23.

Acronym Full Name	Typical Consumer Products	SPI Code
PET Polyethylene Terephthalate	▪ Bottles: Soft drinks, honey, liquor, dish detergent, antacid, cold medicine, some oven food trays, and peanut butter jars.	1 PETE
HDPE Natural High Density Polyethylene (without color)	▪ Jugs: milk, cider, distilled water and spring water bottles, juice (not clear), rubbing alcohol, and large vinegar. Grocery bags.	2 HDPE
HDPE High Density Colored Polyethylene	▪ Bottles: laundry and dish detergent, fabric softener, saline solution, bleach, motor oil, and antifreeze.	2 HDPE
PVC Polyvinyl Chloride	▪ Bottles: imported mineral water, salad dressing, salad and vegetable oil, floor polish, mouthwash, liquor, some translucent pharmaceutical bottles, bottle liners and cap coatings, blister pack "bubble" for batteries, tile and drainage pipes.	3 V
LDPE Low Density Polyethylene	▪ Usually appears in flexible film bags for dry cleaning, bread, produce, trash, etc.; also some rigid items such as food storage containers and flexible lids, coatings, and recycling bins.	4 LDPE
PP Polypropylene	▪ Battery cases, medical containers; oil additive containers, some dairy tubs; cereal box liners; bottle caps; rope and strapping; combs; snack wraps; bags; some yogurt cups and lids for containers (those that do not crack easily when bent).	5 PP
PS & HIPS Polystyrene & High Impact PS	▪ Some yogurt cups and tubs, cookie and muffin trays, clear carry-out containers, vitamin bottles, most fast food cutlery, waste baskets, and audio cassette tapes.	6 PS
Other Various Items	▪ Plastics other than the six most common or made of multiple layered resins, blends, or different parts (i.e., water cooler bottles, microwavable serving ware, most snack bags, and squeezable bottles for condiments, etc.).	7 OTHER

Figure 2-23: SPI Plastic numbering system (Kashi, et al., 2001)

HDPE can be recycled as it is possible to melt it down at temperatures above 130°C (Acor, 2003). Figure 2-24 below, extracted from the 2012 Plastics Recycling Survey conducted by Plastics S.A, shows that the amount of recycled plastic is increasing every year with HDPE being the fourth most recycled plastic in South Africa. It must be noted that there was a substantial increase in recycling activity in 2009 compared to 2000, which supports information illustrated by Figure 2-21, that shows a stabilisation in virgin mineral consumption from around 2008 onward after a sharp increase from 2000-2006 (Plastics S.A., 2012). By comparing Figure 2-21 to Figure 2-24 it can also be seen that in 2011, 18.9% of all plastic consumed was recycled and 15.86% of the total plastic recycled was HDPE.



	PE-LD/LLD	PET	PP	PE-HD	PVC	OTHER	PS
2000	64 476	5609	13 290	22 197	11 323	3 971	1 450
2005	74 666	18 964	26 543	29 597	13 752	4 631	4 100
2009	92 381	33 870	43 000	36 547	15 490	3 379	3 389
2010	101 454	39 855	38 614	39 733	15 830	3 112	3 256
2011	95 852	46 276	40 282	38 979	16 704	4 388	3 213
2012	98 971	50 280	47 080	45 950	16 812	6 060	3 394

Figure 2-24 Tonnage of recycled plastic in South Africa (Plastics S.A., 2012)

Figure 2-25 below, illustrates that most of the HDPE in South African waste is sourced from crates and pipes. The quantity of recycled HDPE per a year from 2009 to 2012 also shows an increasing general trend as per Figure 2-24. The recycling rate percentage shown in Figure 2-25 varied however by 1.6% from the Figure 2-24 above, possibly due to statistics on plastics consumption varying, as the tonnage recycled is still consistent with Figure 2-24.

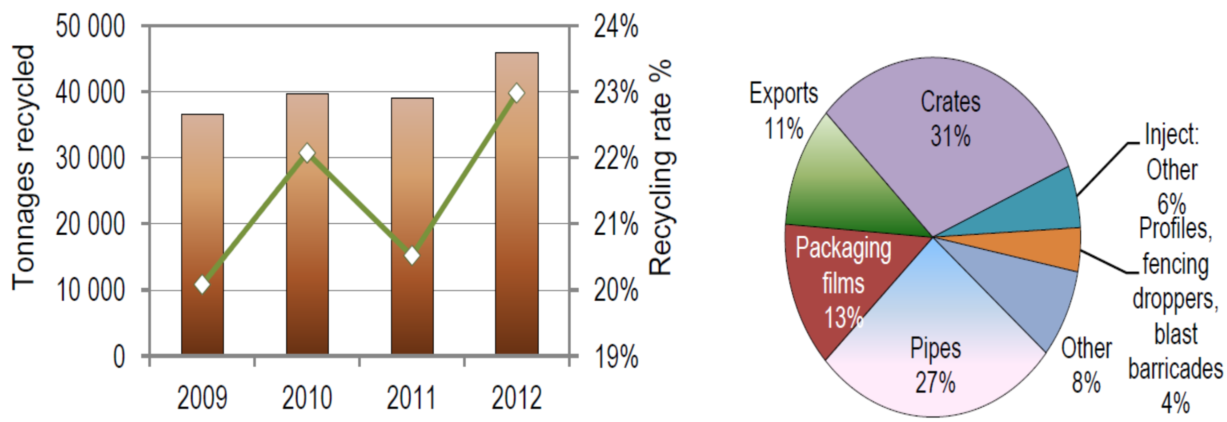


Figure 2-25: Source of HDPE (TEP Smart, 2013)

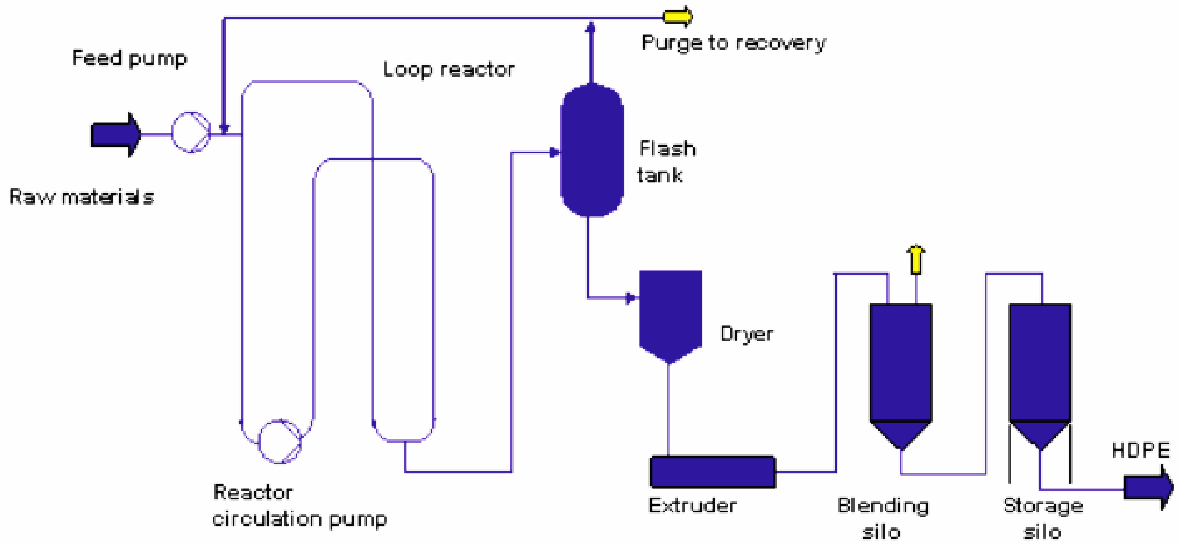
The processes related to HDPE production and recycling will be explained in section 2.2.3.2.

2.2.3.2. **Method of processing**

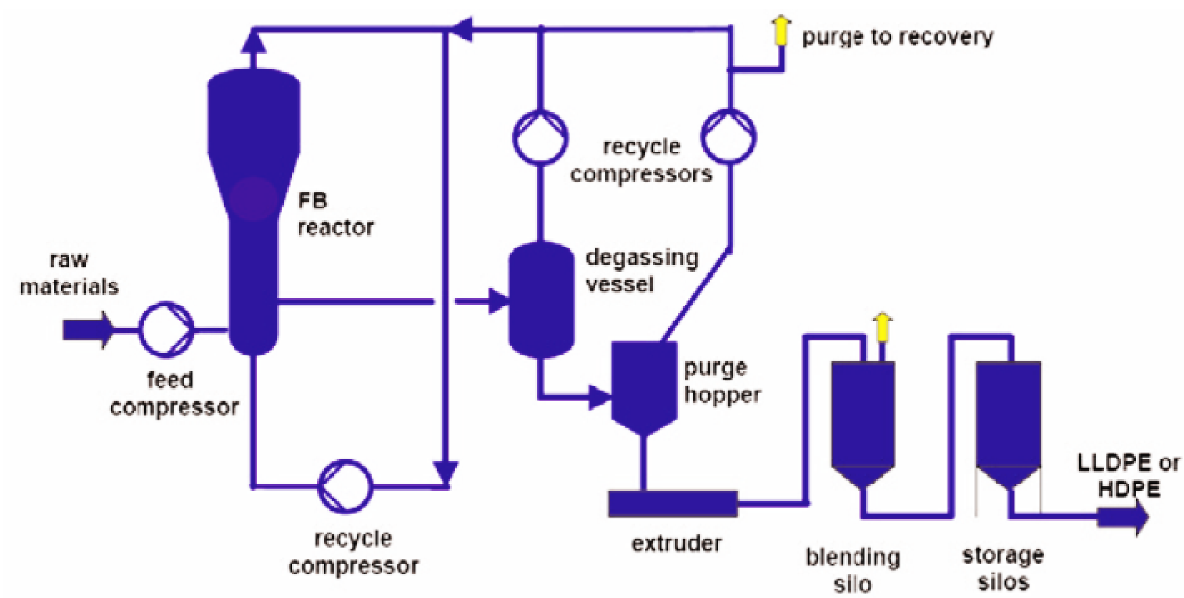
Recycled HDPE was used for this study but to provide the reader with information on how the HDPE itself is formed, the process of virgin HDPE production will be explained. Virgin HDPE is the resulting product of the polymerisation of ethylene and can be manufactured using two different processes called slurry polymerisation and gas phase polymerisation

Slurry polymerisation involves producing the polymers at relatively low temperatures (70-110°C) and low pressure (1-5MPa) in a saturated hydrocarbon medium. After a slurry is formed the reaction medium is removed and the polymer powder separated and mixed with stabilizers. The stabilized polymers are then most often extruded into pellets (Plastics Europe, 2008).

Gas phase polymerisation is when polymers are formed by passing ethylene gas at high speeds or stirring a bed of dry polymer particles at low temperatures and pressures. The powder is stabilized as with the slurry process and often pelletized (Plastics Europe, 2008).



Slurry process (EC 2006)



Gas phase process (EC 2006)

Figure 2-26: HDPE manufacturing process (Plastics Europe, 2008)

As mentioned, recycled HDPE was investigated for this study and the process of recycling is described as follows.

HDPE waste is separated according to colour and manual sorters remove any PVC, metals or glass. Labels and dirt are separated before the pre-wash where plastic caps are removed. The HDPE is then crushed and ground. The HDPE flakes are then cold-washed to remove any water based glues, dehydrated and dried. Finally the dried HDPE is melted, extruded and pelletized which is the form used for this study (TEP Smart, 2013). Figure 2-27 illustrates the HDPE recycling process.

Figure 2-27: HDPE recycling process

Even though recycling still involves the melting of plastics, it is still preferred to virgin pellets usage. This is because its reduces the consumption of petroleum (ethylene) for HDPE production, emits 78% less greenhouses gasses (Envision plastics, 2014) and consumes less energy (Vlachopoulos, 2009).

2.2.3.3. Critical review on the literature about HDPE pellet material properties

The physical properties shown in Table 2-5 below, were used as reference values for this study where applicable.

Table 2-5: HDPE property values summarised from literature

Property	Value	Reference
Particle size	2 mm x 2mm cylinder	RE plastics (supplier)
Relative Density	0.95 (unit less)	(Dynalab, 2013)
Specific heat capacity	2250 J/kg °C	(Designer Data, 2013)
Modulus of elasticity	1.035 GPa	(INEOS, 2009)
Tensile strength	27 MPa	(Polyraw Enterprises, 2014)
Water absorption percent	0.03 %	(INEOS, 2009)
Melting point	120°C	(Dynalab, 2013)
Resistivity	10^{16} ohm.cm	(Leenhouts, 2014)

The relative density of HDPE is below 1, this means that the pellets have the tendency to float toward the surface and effect the uniformity and subsequent strength of the hardened concrete. (Ravindrarajah, et al., 1996). Liu & Song (2010), have shown that by using a short vibration time on a vibration table as opposed to normal vibration times, this effect can be minimised. Pozzolans have also been used to increase cohesion in the mix and reduce the floating of aggregates (Ahmad, et al., 2008). In this study, the HDPE mixes and HDPE + BA mixes were vibrated according to Liu and Songs (2010) recommendations and variations in terms of compressive strength and durability properties were commented on by comparing the HDPE mixes to the conventional (non-waste) mix.

The relatively low modulus of elasticity of the HDPE aggregates will not offer as much resistance to deformation as the stone it partially replaces and hence may lower the elastic modulus of the mix (Lamond & Pielert, 2006).

The melting point of the HDPE pellets are 120°C so if any drying of samples are required drying temperatures must be below this value.

The size of the HDPE pellets means that there could be the possible beneficial effect of particle packing, if the HDPE pellets are evenly distributed in the mix creating well-graded aggregate matrix *(see* Figure 2-28*)*. Good particle packing creates a "lattice effect" where the small aggregate resist settlement of the middle-sized aggregates, which in turn resist settlement of the large aggregates (Shen, 2007). The benefits of this are; improved aggregate segregation resistance, strength and durability (Shilstone, 2009). The strength and durability of mixes with HDPE pellets were investigated in this study.

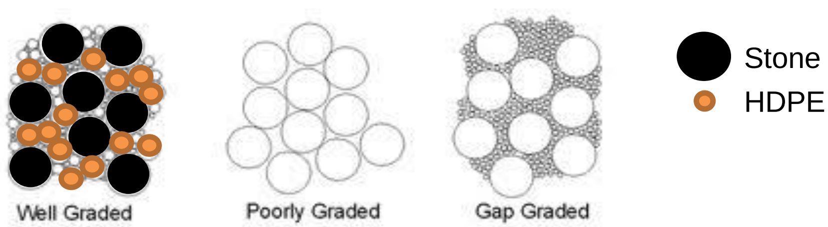


Figure 2-28: Packing of aggregates

In terms of the specific heat of HDPE, the value of 2250 J/kg °C *(see* Table 2-5*)* is high compared to the coarse aggregate 837J/kg °C (Harder, 2008), the HDPE partially replaces as well as concrete in general. As stated, the specific heat of the concrete mix is influenced to an extent by the properties of the aggregates (Lamond & Pielert, 2006), therefore, an increase in specific heat compared to a conventional concrete mix can be expected in theory. However, other factors such as density and void content will also have to be considered when making conclusions on the effect of HDPE on specific heat.

Another factor to consider in terms of material properties of HDPE is the fact that HDPE is chemically inert. This coupled with the smooth surface and hydrophobic nature of HDPE may affect bond strength with the cement thus reducing the strength of the hardened concrete (Choi, et al., 2009). The hydrophobic nature and smooth surfaces of the pellets will however increase workability compared to the conventional concrete mix (Ahmad, et al., 2008). The strength of the HDPE mixes were compared to the workability results to evaluate this statement in this study.

HDPE was also shown to have a high resistivity and is considered an insulator. This may cause an increase in the chloride conductivity test readings which are based on current flow through a specimen (See section 4.9.2 for chloride conductivity analysis).

2.2.3.4. **Past research and applications**

Elzafraney *et al.(*2005) researched the use of recycled plastic aggregates in energy efficient buildings. The researchers used high-density polyethylene, polyvinyl chloride and polypropylene to replace some of the coarse aggregates by 5% volumetrically to try to enhance the thermal properties of the concrete mix. The researchers constructed a control building using conventional concrete and a test building with concrete comprising of a high recycled plastic content. The experimental results were validated with the *SUNREL* energy modelling software. It was found that by introducing recycled plastic aggregates to the concrete mix, the building's heating and cooling loads were reduced by 4% in comparison to the control. This was mainly because of the improved thermal insulation and lower conductivity properties of the plastic aggregate concrete mix. It was also noticed that the higher thermal mass of the plastic aggregate mix, which is related specific heat, assisted in reducing temperature swings due the greater resistance to thermal change (Elzafraney, et al., 2005). It can therefore be said that by introducing recycled plastic to the concrete mix, energy efficiency and comfort can be improved. However, the reserachers did not investigate the influence on thermal properties with increasing volumetric substitutions of plastic. In theory, the aggregates of a concrete matrix have an influence on the specific heat because they make up a large portion of the mix (Lamond & Pielert, 2006). Therefore, if the properties of the aggregates are changed, this should have an influence on the concrete properties and hence yield changes to the specific heat of the mix. Specific heat was therefore investigated in this study with varying volumes of plastic in the mix (see section 4.7).

Al-Manaseer & Dalal (1997) conducted research on concrete with plastic aggregates using proportions of 10%, 30% and 50% substitution of coarse aggregate and compared the results to a conventional mix with no waste. They found that the 10%, 30% and 50% plastic substitutions lead to a 2.5%, 6% and 13% decrease in density respectively when compared to a conventional mix with no waste. This shows that density is inversely proportional to plastic content. The compressive strength of the concrete was also tested and the same trend was noticed. When the proportion of plastic increased for 10%, 30% and 50% substitutions, the compressive strength decreased by 34%, 51% and 67% respectively compared to a conventional mix with no waste (Al-Manaseer & Dalal, 1997). This indicated a relationship between density and compressive strength. As the density decreased, more voids could have been present in the mix and this may have brought about a decrease in compressive strength.

The relationship between density and compressive strength was investigated and related to permeability tests carried out for durability in this study. Al-Manaseer & Dalal (1997) also investigated the effect of volumetric substitutions of plastic on the modulus of elasticity of concrete. It was found that as the volumetric proportions of plastic increased, the modulus of elasticity decreased. This could have been attributed to the low elastic modulus of the plastic aggregates, increased void content and the poor bond between the aggregates and the cement paste.

Rahman *et al.* (2012) studied the potential of recycled HDPE waste materials as partial coarse aggregate substitutes of 10%, 15%, 20% and 25% of total volume. There were notable decreases in compressive strength and density. All mixes yielded a lower compressive strength than that of the control mix (no waste) and the least reduction in strength was noticed at 10% volumetric substitution. The authors showed that the compressive strength of the concrete mixes decreased on average by 0.6 MPa and density by 0.2% (relative to conventional mix), for every volumetric percent of HDPE added. The decrease in strength with decreasing density showed a directly proportional relationship which was similar to the findings by Al-Manaseer & Dalal (1997). It was concluded that concrete modified with polymeric waste aggregates was not suitable for structural applications due to the decline in strength.

Ismail & AL-Hashmi (2007) substituted conventional aggregate with volumetric substitutions of 0%, 10%, 15% and 20% using shredded plastic. The slump, density, compressive strength and flexural strength were tested for the waste mixes and compared to a conventional concrete mix, at curing ages of 3, 7, 14 and 28 days with a fixed W/C ratio of 0.53. The slump showed rapid decreases to 68.33%, 88.33% and 95.33% of the control slump of 78 mm for 10%, 15% and 20% plastic volumetric substitutions respectively. The decrease in slump can be attributed to the irregular sizes of plastic aggregate. The fact that slump decreased using a particle with a smooth surface indicated that the shape of the particle had a more significant effect on the mix than the surface texture. The density and compressive strength also decreased as the volumetric percentage of waste in the mix increased. The study showed the least reductions in strength were at 10 % volumetric substitution, where there was a 25% decrease from 40 MPa to 30 MPa in compression and a 25% drop from 5.5 MPa to 4.1 MPa in flexure when compared to the control (no-waste) mix. A 3% reduction in density was also noticed at 10% volumetric substitution. The results by Ismail & AL-Hashmi (2007) corresponded with research by Rahman *et al.* (2012) and Al-Manaseer & Dalal (1997) in terms of the loss in strength. This loss in strength can be attributed to the decrease in adhesion between the cement matrix and the smooth surfaces of the plastic aggregates.

Suganthy, et al. (2013) carried out research on the use of HDPE as a concrete aggregate substitute in volumetric substitutions of 0%, 25%, 50%, 75 and 100%. The compressive strength at 7, 14 and 28 days was investigated. A decrease in compressive strength was noted for all volumetric substitutions and varied from a 1% decrease in strength compared to the conventional mix at 25% volumetric substitution to a 54% drop in strength at 100% volumetric substitution. The 7 day strength loss showed a constant decline as the volumetric substitution increased. However, the 14 and 28 day substitutions showed a more rapid decline after 25% volumetric substitution. The reason for this is that at 7 days the C-S-H crystals may not have formed enough for the waste aggregates bond properties to have a noticeable effect but at 28 days the C-S-H particles have grown and interlocked with each other and this is when the poor surface bond properties of plastic may be significant. Suganthy, *et al*. (2013) did not investigate this behaviour for low-volume substitutions. This MSc study investigated the variation in 7 and 28 day strength with substitutions ranging from low to high volumes, to possibly find a critical volume at which strength was not negatively impacted.

To establish the best shape of plastic to use, research by Ferreira, *et al* (2012) was referred to. They investigated the effect of the shape (shredded, pellets, fibres) of plastic polyethylene terephthalate (PET) aggregates on concrete. The researchers used volumetric substitutions of 7.5% and 15% and compared results to a conventional mix with no waste substitution. In terms of fresh properties, they found that because of the smooth cylindrical shape of the pellets, workability was improved compared to a conventional mix, whereas the angular shape of shredded and fibre strands decreased workability.

In terms of hardened properties, they found that an increase in volumetric substitution resulted in a decrease in compressive strength, splitting strength and the modulus of elasticity when compared to a mix with no waste. The superior performance of pellets compared to fibres and shredded plastic is shown in Figure 2-29 for compressive strength results. The same trend was shown for the other tested properties.

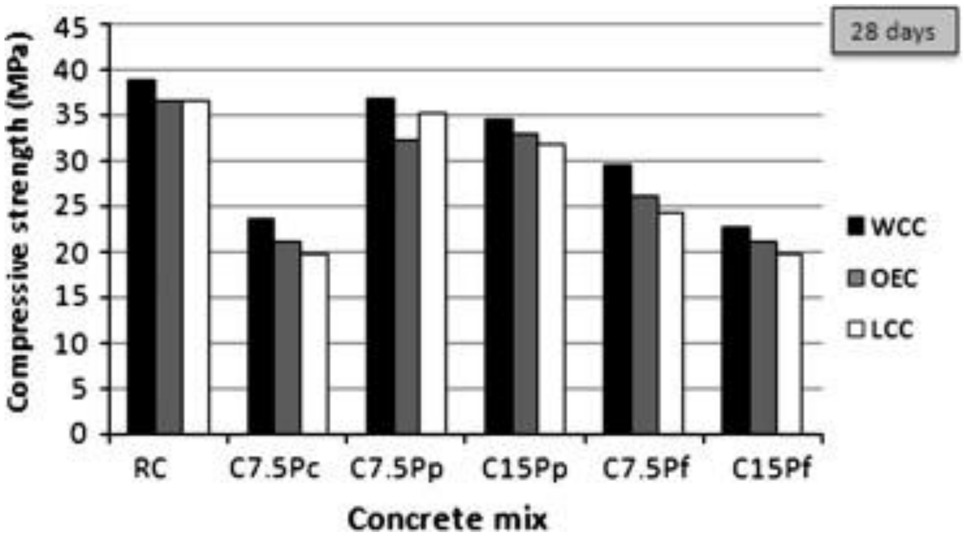


Figure 2-29: Compressive strength results at 28 days (Pc) shredded, (Pp) pellets, (Pf) fibres (Ferreira, et al., 2012)

Table 2-6 shows the hardened property results for pellets, which was the best performing shape. The mixes containing pellets at 7.5% and 15 % substitution decreased by; 2.7% and 5% in compressive strength, 6.8 % and 20% in splitting strength and 9% and 15 % for elastic modulus respectively. It was noticed that the losses in strength and elastic modulus approximately doubled with double the volumetric substitution of plastic aggregates indicating a linear relationship. The reduction in strength and elastic modulus could be attributed to the weak bond strength between plastic and cement mix, as well as the hydrophobic nature of plastic which repels water thus affecting the hydration of cement in the interfacial zone.

Table 2-6: Results of study by Ferreira, et al. (2012)

	Average Compressive strength	**Average Splitting strength**	**Average Elastic modulus**
Control (no waste)	36 MPa	2.9 MPa	32 GPa
7.5 %	35 MPa	2.7 MPa	29 GPa
15%	33 MPa	2.3 MPa	27 GPa

Based on findings by Ferreira, *et al* (2012), PET aggregates performed best when pelletized. Therefore this study investigated the use of pelletized HDPE. However, the Polyethylene terephthalate (PET) used by Ferreira, *et al* (2012) is stronger and stiffer (SubsTech, 2013) than HDPE, so strength and elastic modulus results were predicted to be less than those found by the researchers. By testing a different type of plastic aggregate to Ferreira, *et al* (2012) this expanded on the knowledge of the behaviour of concrete containing plastic aggregates.

2.2.3.5. **Summary of possible benefits of using HDPE pellets in concrete**

The main benefits of using HDPE in concrete, based on the findings from the available literature, are the following:

- Recycled HDPE, in theory, should reduce the density of hardened concrete as it has a substantially lower density than that of stone. This theoretical decrease in density would result in a reduction of dead loads imposed on a structure therefore leading to savings in terms of foundations and reinforcement required (Rahman, et al., 2012).
- Pre-cast units could also benefit from a decrease in density in terms of reductions in the transport and handling costs.
- Improved workability due to smooth surface can be obtained (Ferreira, et al., 2012).
- Improved specific heat/ thermal mass properties (Elzafraney, et al., 2005).
- Plastic is not biodegradable hence recycling it for concrete use is beneficial to the environment.

2.2.3.6. **Summary of HDPE pellet limitations in concrete**

The main limitations of using HDPE in concrete, based on the findings from the available literature, are the following:

- Plastics generally have low bonding properties which results in lowered compressive, flexural and splitting strengths (Ismail & AL-Hashmi, 2007) (Al-Manaseer & Dalal, 1997).
- Reductions in density can also mean increased voids in the mix which may reduce durability.
- HDPE has a low elastic modulus which means a lesser degree of resistance to strain under load and hence a potentially lower concrete elastic modulus compared to conventional concrete.

2.2.3.7. **Concluding remarks on HDPE**

The increase in waste production emphasizes the need to utilize waste materials in order to reduce possible effects on the environment by materials that are not bio-degradable such as plastics. This need for waste usage creates an opportunity for the use of HDPE as a concrete aggregate.

Past research was mainly focused on high-volume substitutions and hence low-volume substitutions were investigated in this study to fill the knowledge gap. Past literature showed that high-volume substitutions (10%, 15% .20% etc.) resulted in decreased strength performance and these losses in strength were also inversely proportional to the density of the concrete mix. The losses in strength performance could therefore be attributed to increased voids in the mix or, the smooth surface and hydrophobic nature of plastic which repels water and limits bonding with the cement matrix. This research investigated if this trend was applicable from low to high volume substitutions, or if there was a critical volume at which performance peaked within a range of 2,5% to 40% substitution. There can also be possible thermal mass benefits to using HDPE, as stated by Elzafraney *et al. (*2005), but based on past research findings, applications utilising these benefits seem to be restricted to non-load bearing purposes due to reductions in strength, durability and elastic modulus properties.

Overall the benefits to using HDPE based on literature were more than the limitations, however, the concerns raised on durability, strength and elastic modulus were significant and were investigated in this study

2.2.4. Coal bottom ash-from sugar mill

2.2.4.1. Background

Concrete, as a popular building resource, has meant that large volumes of cement are required where construction expands rapidly. For example, in China, cement consumption increased by 14% from 2011 to 2013 (PCA, 2013). This further warrants the need for research into cement substitutes to cater for this rapid increase in demand (Swamy, 1986). Pozzolans are examples of such substitutes (see section 2.2.1.2 for general information on pozzolans). Coal bottom-ash is a waste product of the coal burning process and is a proposed pozzolan that was investigated in this study.

South Africa has large coal reserves and is the fifth highest consumer of coal in the world as shown by Figure 2-30 on page 82 (European Commision, 2013). The large consumption of coal can be attributed to the fact that 77% of energy generated in South Africa is from coal fired power-plants (South Africa.info, 2012). The burning of coal generates waste in the form of unburnt particles called, fly-ash (fine particles) and bottom ash (coarse particles), which was used for this study. About 12% of the coal that is burnt in these power plants ends up as bottom-ash (Natures Way Resources, 2006). National South African electricity provider Eskom burns approximately 120 million tons of coal a year (Eskom, 2014) and this roughly translates to 14.4 million tons of bottom ash waste available annually. Another source of ash are the sugar mills. The Sezela mill, where the boiler ash was sourced for this study, utilises bagasse as much as possible, but still burns approximately 50 000 tons of coal annually which equates to 6 000 tons of boiler ash being available from just one mill (Munsamy, 2008).

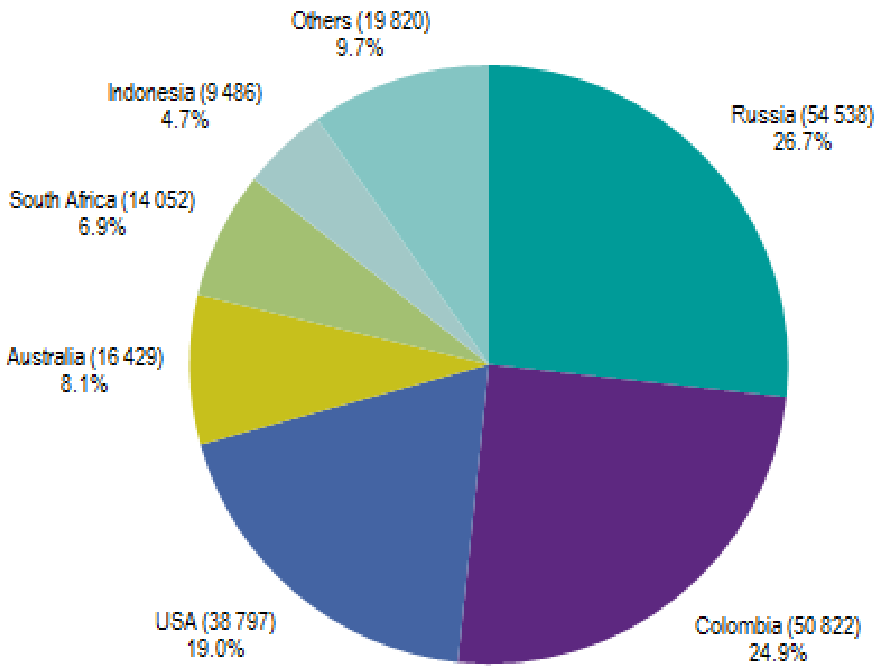


Figure 2-30: Graph of Worldwide coal consumers (European Commision, 2013)

Coal bottom-ash is the incombustible, relatively coarse to fine material found at the bottom of coal boilers after the coal has been burnt (Kurama & Kaya, 2007). In terms of the use of ash in concrete, fly-ash has generally been more popular than bottom ash for partial cement replacement, because bottom ash has a relatively higher unburnt carbon content and is less reactive due to larger particle size (Singh & Siddique, 2013). However, both unused fly-ash and bottom ash are disposed of in ash dumps which can cause a negative impact on the environment, for example, possible groundwater contamination (RMRC-3G, 2012). Hence, a reduction in the amount of bottom ash that gets disposed of in an ash dump will still be of environmental benefit. To maximise the effect of using bottom-ash in concrete, unburnt carbon should be removed by methods such as electrostatic separation (Kurama & Kaya, 2007). Methods such as electrostatic separation however, were not applied in this study as the aim was to avoid additional treatment processes to make the material more economical and easily applicable.

Coal bottom-ash is very similar to fly-ash in terms of the main chemical composition comprised of silica (SiO_2), alumina (Al_2O_3), and iron oxide (Fe_2O_3) as shown in table 5. The reason for the lower reactivity of bottom-ash is therefore due to the larger particle size and lesser specific surface area compared to fly-ash (RMRC-3G, 2012). Silica is consumed in the pozzolanic reaction with $Ca(OH)_2$ *(See section 2.2.1.2.)* to form calcium silicate and aluminate hydrates (Juma, et al., 2010). These hydrates reduce void content and hence permeability of the concrete which results in time-dependent improvements in strength (Kurama & Kaya, 2007).

Table 2-7: Main chemical compound of bottom-ash (Martins, et al., 2010)

Chemical composition of bottom-ash	
Compound	Weight %
SiO_2	52.02
Al_2O_3	23.23
Fe_2O_3	9.11
CaO	6
MgO	2.17
K_2O	1.14

Apart from the possible reduced permeability and increased long-term strength due to pozzolanic reaction, the increasing consumption of natural resources and the environmental hazard posed by ash dumps are further reasons to support research into the use of bottom ash as a cement substitute. Based on Figure 2-32, which shows the ways in which coal ash has been utilized in the U.S.A, the main use for bottom ash is for structural fills and embankments, followed by raw feed for clinker and road sub-bases (RMRC-3G, 2012). Concrete forms just 7.1% of bottom ash usage. This shows that there is potential to expand this sector by studying its use in concrete further to get a deeper understanding of its applications and hence improve knowledge of the materials behaviour for the construction industry.

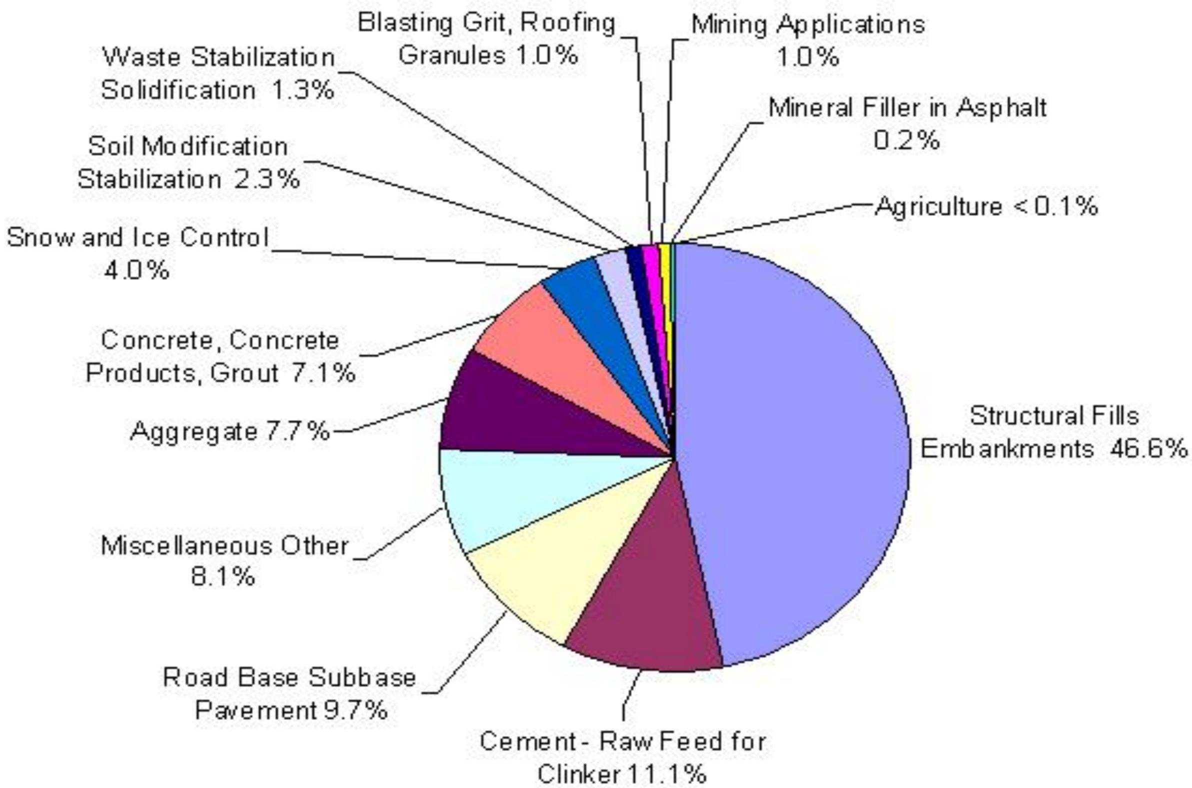


Figure 2-31 Uses of coal bottom ash (RMRC-3G, 2012)

2.2.4.2. **Method of processing**

The most common source of coal–bottom ash is electricity generation facilities. After coal is combusted at temperatures in excess of 1000°C, bottom ash is the coarse ash in a molten state that adheres to the sides of the furnace and falls to the bottom of the boiler. It is then cooled and flushed with water at high pressure. A general diagram of a coal power plant is shown in Figure 2-32 (WRAP, 2010). Essentially, the same coal boiler process used at a power plant also applies to a sugar mill, as confirmed by a power generation technician at Sezela mill. The only difference is that fly-ash is generally removed with precipitators as shown in Figure 2-32, however, at Sezela wet scrubbers were used instead.

Sugar mills also burn bagasse in their boilers. The bottom ash from bagasse, however, was produced in much lower volumes in terms of national production compared to coal bottom ash. This is because most of the bagasse solids are burnt up leaving as little as 3.5% (Phillips, 2011) of the total solids as residual solids (ash) in the boiler. The lower volume production may limit the availability of the material for construction and hence coal bottom ash was chosen for the study.

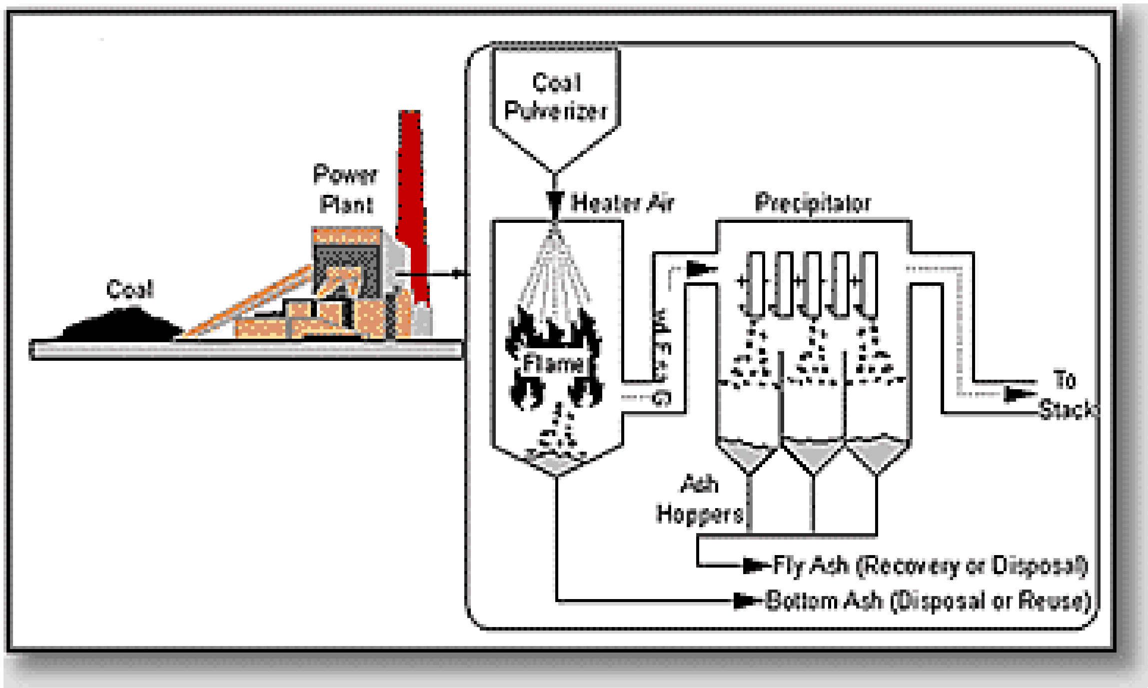


Figure 2-32: Coal processing diagram (Federal Highway Administration, 2012)

2.2.4.3. **Critical review on the literature about BA material properties**

The physical properties shown in Table 2-8 were used as reference values for this study.

Table 2-8 : Bottom ash property values summarised from literature

Property	**Value**	**Reference**
Relative density	2.1-2.7 (unit less)	(Federal Highway Administration, 2012)
Water absorption	0.8-2%	(RMRC-3G, 2012)
Particle size diameter	0.1 mm – 38,1mm	(Benson & Bradshaw, 2011)
Specific heat capacity	840 J/ kg K	(The Engineering toolbox, 2014)

Bottom ash consists of angular, porous particles and this is possibly why it has a water absorption percentage of 0.8-14.1% (Benson & Bradshaw, 2011). Due to a combination of the particle shape, surface texture and water absorption, this may result in the workability of the mix decreasing due to particle interlock and removal of water from the mix. The porous nature of bottom ash does however have the benefit of prolonged internal curing as hydration can be sustained continually by water held in the pores of the bottom ash particles. Hydration products subsequently fill these pores and coupled with the benefit of pozzolanic products after approximately 7 days (RMRC-3G, 2012), concrete strength properties can be improved (Saikia & de Brito, 2012). The porosity of BA was also shown to increase the water sorptivity when introduced to a concrete mix through increased capillary action (Andrade, et al., 2007).

The size of particle has a large variance from as large as 38,1mm to less than 0.075 mm. The amount of coarse particles present in bottom-ash particles is a reason why it is considered less reactive than fly-ash because of a reduced surface area. The finer the particles, the higher the reactivity. This study attempted to use bottom ash as a cement substitute and hence bottom ash was passed through a 1400 μm sieve before use to reduce the average particle size and possibly improve pozzolanic reactivity by providing a larger reaction surface area in accordance with research by Cheriaf, *et al*. (1999).

The specific heat of bottom ash is similar to that of the cement it replaces so in theory the specific heat capacity of bottom ash introduced to the mix should not influence the mix significantly in terms of changes to specific heat capacity. However, if air voids in the hardened mix are reduced through pozzolanic reaction, this may decrease the specific heat capacity of the mix because the specific heat capacity of air (1000 J/kg °C) is higher than the aggregates and binder in the mix (The Engineering toolbox, 2014).

2.2.4.4. **Past research and application**

Research into the use of bottom-ash has mainly been focused on its applications as an aggregate replacement due to its coarser grain size compared to fly-ash, as carried out by Aggarwal *et al.* (2007) and Andrade *et al.(* 2006). Research by Kurama & Kaya (2007) investigated the pozzolanic properties of substituting cement with bottom ash, but only on mortars and not concrete. The lack of research on application as a cement substitute in concrete was seen as a knowledge gap that could be fulfilled with this study.

Aggarwal *et al.* (2007) carried out an experimental investigation on the effect of adding bottom ash to concrete as a replacement for fine aggregate in substitutions of 20%, 30%, 40% and 50% by weight. The authors investigated the workability, compressive, flexural and splitting strengths in comparison to conventional concrete.

The workability decreased for all mixes from 3% at 20% substitution, to 9,7% at 50% substitution, relative to the control mix. The reason for this was the increased surface area compared to the fine aggregate it replaced which increased water demand and hence decreased workability. The effect of BA on workability was investigated in this study in section 2.3.3.

Figure 2-33 illustrates the compressive strength results for the M1 (control), M2(20%) , M3(30%) , M4 (40%) and M5(50%) mixes, from the study carried out by Aggarwal, *et al*. (2007).

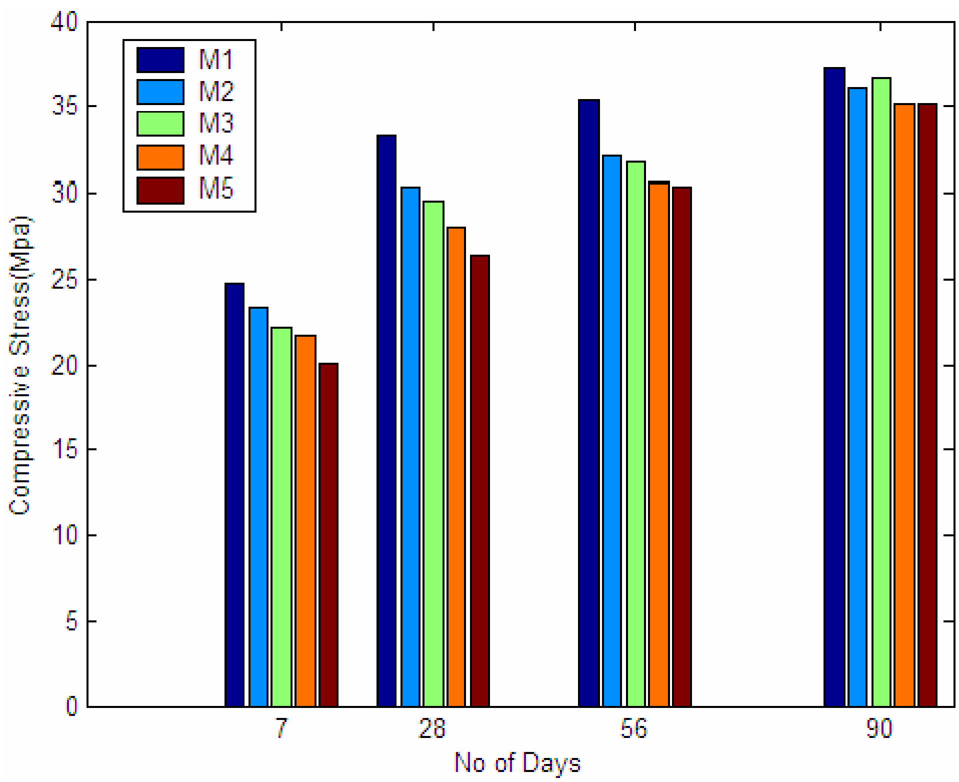


Figure 2-33: Compressive strength results (Aggarwal, et al., 2007)

It was found that the mixes containing ash all showed decreases in compressive strength relative to the control mix, irrespective of the curing age. The least reductions in compressive strength of 5.9%, 8,7%, 9,1% relative to the control mix, were at 20% substitution after, 7, 28 and 56 days of curing respectively. However, after 90 days, the least reduction of 1,1% relative to the control mix was at the 30% substitution. This was because after 90 there was a greater amount of pozzolanic products for the 30% substitution compared to the 20% substitution and hence better compressive strength results. Beyond 30% substitution however the porous nature of the BA may have offset any benefits yielded from the pozzolanic products.

Figure 2-34 illustrates the flexural strength results for the M1(control), M2(20%) , M3(30%) , M4 (40%) and M5(50%) mixes, from the study carried out by Aggarwal, *et al*. (2007).

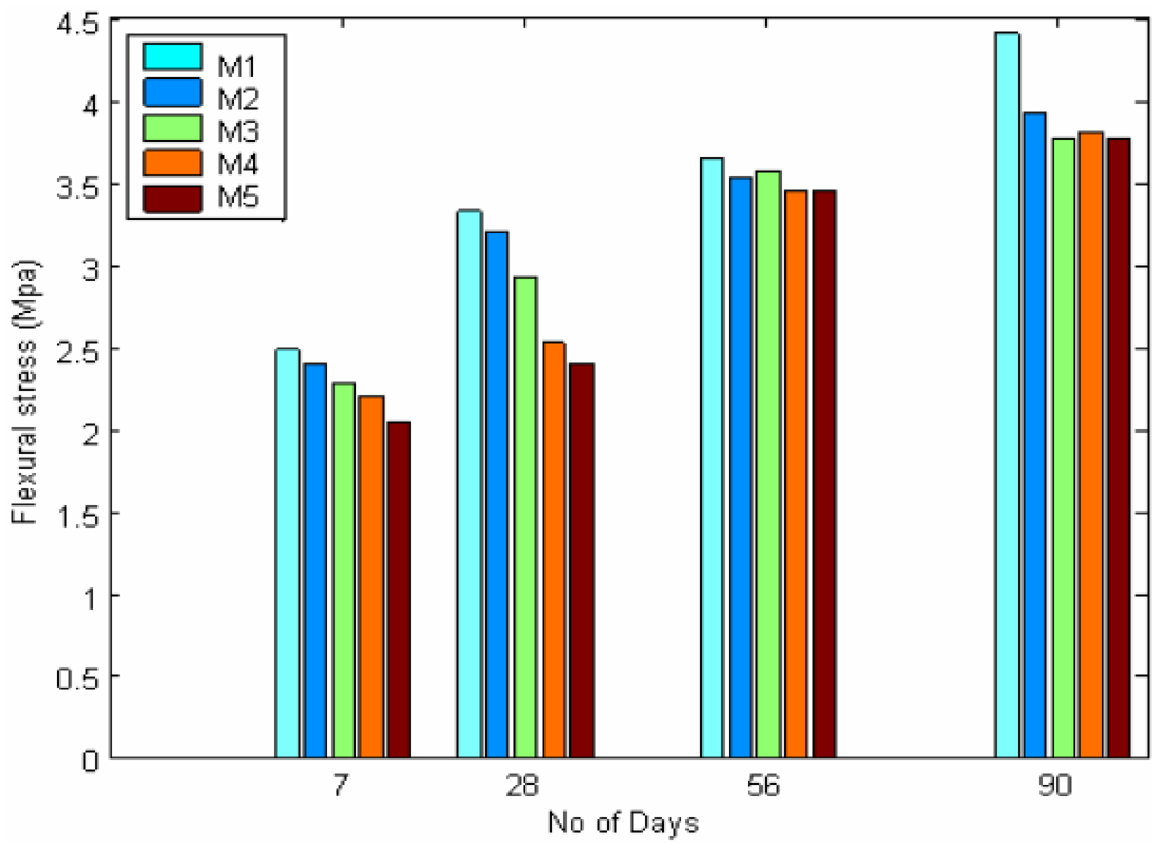


Figure 2-34: Flexural strength results (Aggarwal, et al., 2007)

For flexural strengths there was a decrease observed at all curing ages for all substitutions. After 7 and 28 days of curing there was a large drop in flexural strength after 30% substitution, however, at 56 and 90 days the flexural strength results varied within a range of 4% of each other. This was because after 56 days the effects of the pozzolanic reaction was noticeable to a greater extent than at the other curing ages. At 90 days however, the difference between the control and the waste mixes was the most, this may be due to slight variances in the cement composition compared to the other curing age batches. The 20% substitution generally yielded the best flexural strength results showing the least reductions relative to the control mix of 3,22%, 3,6% ,5,5% and 11% at 7, 28, 56 and 90 days respectively.

Figure 2-35 illustrates the splitting strength results for the M1(control), M2(20%) , M3(30%) , M4 (40%) and M5(50%) mixes , from the study carried out by Aggarwal, *et al*. (2007).

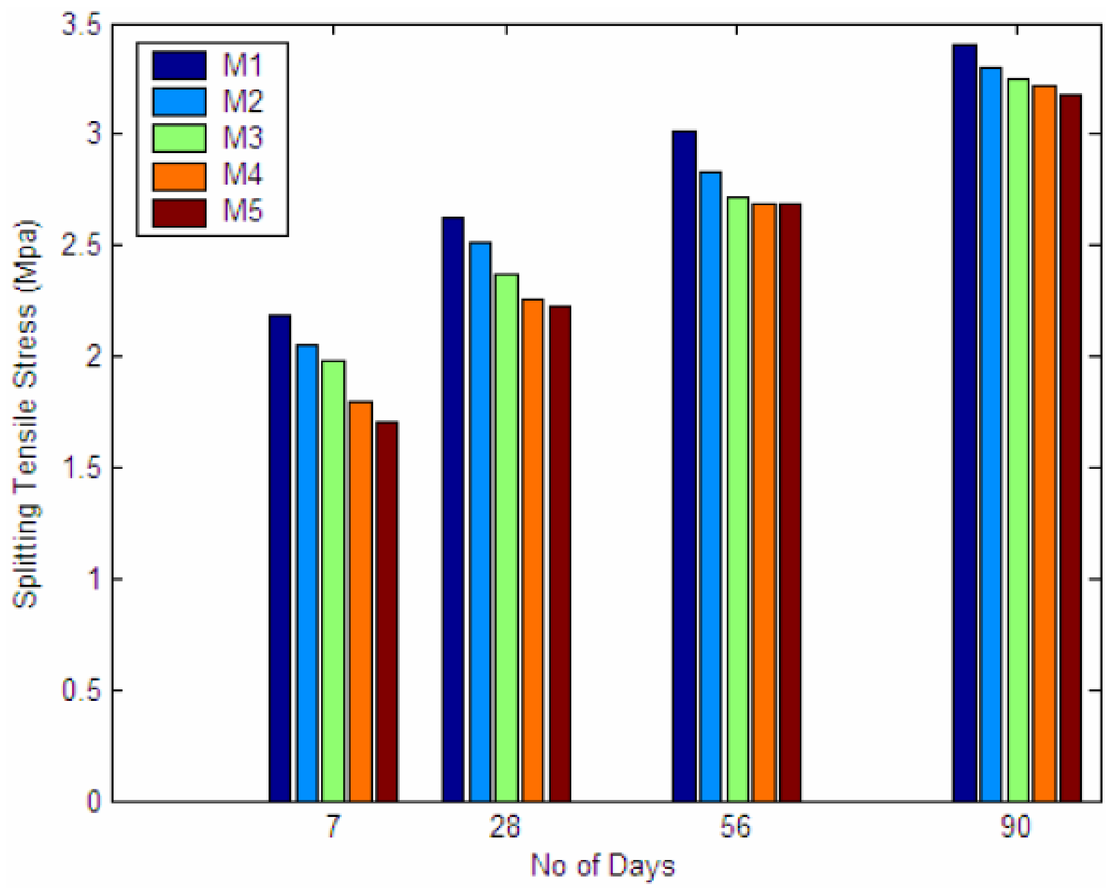


Figure 2-35: Splitting strength results (Aggarwal, et al., 2007)

In terms of splitting strength, all mixes at all curing ages showed reductions in splitting strength relative to the control mix, irrespective of curing age. The 20% substitution showed the least reductions in strength of 6.39%, 3.82%, 5,98% and 2,9% at 7, 28, 56 and 90 days respectively. As with compressive strength, the change in strength difference between the control and the waste mix was because the pozzolanic reaction was more significant after 90 days of curing.

It was concluded that the substitution that showed the least reductions in strength overall was found to be 20% by weight (Aggarwal, et al., 2007).However, the strength at this substitution was less than the control and the target strength at 28 days and therefore it would not be viable for structural use.

Andrade, *et al*. (2006) studied the use of coal bottom ash from a thermoelectric power station as a substitute material from sand. The mechanical and pore formation properties were tested. It was acknowledged that bottom ash can be used as either a cement or fine aggregate replacement. The introduction of bottom ash was theorised as having an influence on the water demand as well as mechanical properties through the degree of pozzolanic reaction. The volumetric substitution method was used to analyse high volume substitutions (25%, 50%, 75% and 100%) of fine aggregate with BA at 3, 7, 28 and 90 days. It was found that the introduction of coal ash led to better filling of pores, however, a greater amount of water was absorbed through capillary action. The compressive strengths all decreased with increasing proportions of bottom ash for tests at 3, 7 and 28 days. The least reduction in terms of compressive strength was for the 25% substitution, which after 28 days showed a 20% decrease in compressive strength and after 90 days showed an 18% decrease in compressive strength relative to the control specimens. Therefore, based on their research, BA was not viable at high volume substitutions.

Kurama & Kaya (2007) tested concrete specimens modified with coal-bottom-ash. Cement was partially substituted in increments of 5%, 10%, 15% and 25% by volume and specimens were cured for 7, 28 and 56 days. The average compressive and flexural strength results of their study are show in Table 2-9.

Table 2-9: Compressive and Flexural strength results (Kurama & Kaya, 2007)

Mixture	Compressive strength (N/mm^2)			Flexural strength (N/mm^2)		
	7 days	28 days	56 days	7 days	28 days	56 days
TS EN 197–1	Minimum 16	Minimum 32.5	–	Minimum 4.0	Minimum 5.5	–
Std	27.8	40.9	42.65	6.08	6.60	6.86
BC5	28.09	40.38	44.08	6.27	6.75	7.35
BC10	28.22	40.24	45.1	6.07	6.62	7.60
BC15	26.47	33.57	43.45	5.77	6.57	7.20
BC25	19.79	29.13	41.33	4.72	6.22	6.69

Their findings showed that both flexural and compressive strengths generally peaked in performance at 5% volumetric substitution after 7 and 28 days of curing, and at 10% volumetric substitution after 56 days of curing. This was possibly because after 56 days the pozzolanic reaction was more significant than at 7 and 28 days and the higher percentage of ash allowed for a greater degree of pozzolanic products to form. Above 10% substitution, strength began to decrease for both compressive and flexural strengths because the loss of cement hydrates exceeded any benefit yielded from the generation of additional pozzolanic products. The BA particles were also more porous than the cement particles and hence this may also have negatively affected the strength results.

As shown in Table 2-9, after 7 days of curing all samples up to and including 10% substitution did show slight increases (1,04%-1,51%) in compressive strength relative to the control mix. This was probably due to slight variances in cement composition, because at 28 days the respective waste mixes were slightly lower in compressive strength (1,2%-1,62%) relative to the control mix. At 56 days however, there were significant increases in compressive strength up to 15% substitution relative to control mix, with the peak reached at 10% substitution which yielded a 5,74% increase in compressive strength.

In terms of flexural strength the 5% substitution performed better after 7 and 28 days with improvements of 3% and 2% respectively, when compared to the conventional mix. After 56 days the 10% substitution yielded the best results showing an increase of 10.78% relative to the control mix.

It can be said that the introduction of bottom-ash does increase compressive and flexural strength properties of concrete for substitutions up to 10% and although the strength results were above the minimum target strength after 28 days, significant improvements were only realised after long-term curing (>56days). Applications of bottom-ash from past literature included base materials in road construction and as a blasting grit.

2.2.4.5. **Summary of the possible benefits of BA in concrete**

The main benefits of using BA in concrete, based on the findings from the available literature, are the following:

- The pozzolanic reaction of bottom ash can potentially reduce the pore size in the concrete which can potentially decrease permeability and hence help improve resistance to corrosion (Nunuz-Jaquez, et al., 2012).

- Coal bottom ash is essentially a free resource.
- By utilizing it, both parties benefit as ash dumps are cleared out for the sugar mills, reducing their disposal costs.
- Concrete costs are reduced for the consumer as well as consumption of natural resources due to a lowered cement content.

2.2.4.6. **Summary of the possible limitations of BA**

The main limitations of using BA in concrete, based on the findings from the available literature, are the following:

- The angular, coarse bottom ash particles may increase the workability of the mix (Benson & Bradshaw, 2011).
- Coal bottom ash can be hazardous to work with. Proper personal protective equipment (PPE) such as gloves, goggles and a breathing apparatus would be required for its handling (TVS, 2001) as with cement.
- Documented barriers for the use of coal-ash and other concrete replacement materials have been poor marketing, failure to provide a quality assured product as well as practical reasons such as additional storage facilities being required for a constituent material (Dhir, 1986).
- Strength benefits were only shown in past literature after 56- 90 days.
- Tensile strength is lower compared to conventional concrete (Aggarwal, et al., 2007).
- Slow compressive strength generation (Aggarwal, et al., 2007).

2.2.4.7. **Concluding remarks on BA**

Bottom-ash is primarily obtained from the coal burning process to generate power. In the South African context Eskom alone generates 140 million tonnes of bottom ash annually, which ends up on ash dumps posing an environmental risk. By using BA as a partial cement replacement, not only is the virgin mineral consumption reduced, but ash is also diverted away from dumps with the possibility of lowering the cost of concrete (see section 4.12 economic analysis). However, by substituting a portion of cement with bottom ash, there are less calcium silicates available for hydration and hence there could be a possible decrease in early strength, because the pozzolanic reaction is slow and any beneficial effect may only be realised at 28 days and beyond. The 7 and 28 day compressive strengths would shed light on this behaviour. If the 28 day compressive strength does not meet a required minimum strength then the economics are irrelevant (see section 4.6.1 for compressive strength analysis).

BA being a coarser particle than cement, has predominantly been used as a fine aggregate and research into using it as a cement substitute was limited. Therefore, this study developed knowledge on its use as a cement substitute.

Past researchers have shown the importance of concrete age when considering the effect of pozzolans on the concrete properties. Andrade, *et al*. (2007) showed an increase in water sorptivity before 28 days of curing due to the porous nature of bottom ash, however, between 56 and 90 days permeability decreased and compressive strength improved. Similar findings were also shown by Nunuz-Jaquez, *et al* (2012), who showed a reduction in permeability and improvements to strength between 56-90 days. This study tested specimens at 7 and 28 days in compression, and permeability after 28 days. Results were compared to findings by Andrade, *et al* (2007) of a similar curing age in terms of the effect of BA on compressive strength and permeability (see section 4.9 for durability analysis).

The particle size has been shown to have a large impact on the reactivity of pozzolans in general (Cheriaf, et al., 1999) hence the BA used in this study was passed through a 1400 μm sieve to reduce the average particle sized and potentially improve reactivity. BA has also been shown to reduce alkalinity of the mix (Gram, 1986) and this could preserve the strength of SCBF. This study investigated this by testing combinations of ash with SCBF as well as HDPE.

Research on BA was mainly based on higher volumetric substitutions without investigation into lower volumes to establish if potential critical volumes existed within a range of 2,5% - 40% substitutions. This was undertaken as part of this study to expand on knowledge on the material.

2.3. Literature on the tested properties in this study

The primary aim of this research was to establish if waste concrete mixes containing HDPE, SCBF and BA described in section 2.2, were viable in the local context in terms of strength, durability, workability and cost. These properties were stated by Mishra (2014) and Bjegović & Oslaković, (2013), to be key in determining the selection of concrete. Further insight was also to be provided on the effects of varying waste substitutions, mixtures of waste materials and properties such as elastic modulus, density, SEM analysis, specific heat and the moisture effects.

To achieve the research aim, this study primarily involved experimental analyses but also consisted of an economic analysis. This section of the study explains the theoretical knowledge behind the testing carried out to further enrich the understanding of why the testing was carried out and what standards were required for respective tests, where applicable. Further descriptions of processes were deemed more relevant to the Methodology and were shown in Chapter 3.

Section 2.3 was structured in a manner that corresponds with the order of tasks that were carried out for the study. Firstly, the tests carried out and literature investigated to establish material properties were explained, as these were needed not only for the mix design but also to aid commenting and reasoning behind any changes to the tested concrete properties. The mix design process selected was then explained followed by a theoretical background of the concrete property tests that were carried out (workability, strengths properties, hardened density, elastic modulus, specific heat capacity, durability properties, SEM analysis, and moisture state analysis). Lastly, the theory behind the economic analysis was discussed which explained the cost estimation and the sources of rates chosen.

2.3.1. Material properties

The material properties selected were:

- Fineness modulus of sand;
- Bulk density of stone;
- Relative densities of materials obtained from experimental analysis or literature;
- Moisture absorption and moisture contents of waste materials (see 2.3.10)
- Sizes of waste materials obtained from experimental analysis or literature.

2.3.1.1. **Fineness modulus (FM)**

Finesse modulus (FM) is a dimensionless empirical factor that indicates the average fineness or coarseness of the aggregate. FM is not the actual size of the particles in millimetres but an approximation of the average size of the particles. The particle distribution needed for the calculation of FM shown below, is obtained from a sieve analysis which indicates the proportions of material mass retained on pre-determined sieve sizes (Addis, 2008).

FM = $(\sum of\ culmintive\ percentages\ of\ retained\ material\ on\ each\ sieve\ 0.15mm\ and\ coarser\ ) \div$ 100

An increase of 1 in FM relates to an increase of two times the particle size. A low FM (around 0.5) entails a very fine material and a high value (about 3.5) indicates a very coarse material. A range of between 2 and 3 is preferable for high-quality concrete (Addis, 1986). Particles from 150μm to 300μm and above have more of an effect on workability than the finer particles (<75μm). A deficiency of particles between 150μm and 300μm can result in the mix becoming harsh and prone to segregation, whereas if in excess the mix can become too sticky. The finer particles (<75μm) have an impact on the strength and durability of the mix. This is because the size of the particles affects the surface area available to interact with the cement matrix as well as the particle packing of the mix which affects void content (Addis & Owens, 2001).

2.3.1.2. **Bulk Density**

The bulk density is defined as the mass of aggregate that will fill a space/container. If the aggregate is not compacted in the container, then loose bulk density (LBD) is calculated, and this value is used for volume batching. If the aggregate is compacted in the container then compacted bulk density (CBD) is calculated, and this value is used to assess packing capacity and for the Cement and Concrete Institute (C&CI) mix design stone content formula. The factors that affect the bulk density of an aggregate are the relative particle density and particle shape (Addis, 2008). For the purpose of this study bulk density of the stone aggregate was calculated using the following test method:

TMH 1-method B8 (CSIR, 1986) – *see section 3.2.2*

2.3.1.3. **Particle relative density (RD)**

In general terms, the particle relative density (RD) is the mass divided by the volume of the particle, relative to the density of water. It is used for batching and mixing calculations. The higher the density, the greater the tendency for the aggregate to settle and expel water through gravitational forces, hence there is a possibility of increased bleeding and segregation (Addis, 2008).

Two approaches had to be adopted due to the varying particle sizes of the aggregates. The first was TMH 1-method B14, for the coarse constituents (stone, HDPE) and the second was TMH 1 method B 15 for fine constituents (coal ash).

Method B 14 used the formula (CSIR, 1986) (see section 3.2.3.1):

$$\text{Relative density}_{\text{stone, HDPE}} = \frac{Oven\ dry\ mass}{Oven\ dry\ mass - (submerged\ mass - mass\ of\ holder)}$$

Method B 15 used the formula (CSIR, 1986) (see section 3.2.3.2):

Relative density ash .sand =

$$\frac{(mass\ of\ oven\ dry\ sample\ and\ flask - mass\ of\ dry\ empty\ flask)}{(mass\ of\ flask\ filled\ with\ water - mass\ of\ empty\ flask)dry - (Mass\ of\ flask\ with\ water\ and\ sample - (mass\ of\ oven\ dry\ sample\ )}$$

Relative densities were converted to kg/m^3 by multiplying by 1000, assuming density of water as 1000kg/m^3 (Elert, 2014).The literature reference relative densities are shown in Table 2-10.

Table 2-10: Relative densities of materials

Name	Relative density (unit less)	Reference
Sand	2.6	(Reade Advanced Materials, 2006)
Stone	2.775	(Addis, 2008)
Cement	3.07	(NPC, 2013)
Bottom ash	2.4	(Federal Highway Administration, 2012)
SCBF	1.25	(C&CI, 2013)
HDPE	0.95	(Dynalab, 2013)

2.3.1.4. **Particle shape and texture**

The aggregate particle shape and texture have a significant effect on the workability and the bond interface properties of the resulting concrete. A spherical or cylindrical shape tends to increase workability (decrease water demand) and a more angular and rough textured particle tends to lower workability (increase water demand). The main reasons for the impacts on workability due to particle shape and texture are:

- Particle interference
- Variations to specific area.

Particle interference according to Weymouth (1933) occurs when there is insufficient space between particles to allow free-passage of smaller particles, hence workability is reduced due to the friction and interlocking between particles. The increased interlock of particles found in rough textured or angular particles is not necessarily only negative however, as it may improve bond characteristics with the cement matrix, whereas smooth surfaced particle may reduce bond strengths.

In terms of the specific surface, which is the surface area per unit volume of a particle, spheres have a lower specific surface than particles that are long and flat. The high specific area of particles that are long and flat restrict movement in the mix, and have a greater surface area that needs to be wetted, which decreases workability. In contrast, smooth rounded aggregates can increase workability as they move easily in the mix. However, having a spherical shape does not automatically mean the workability will be increased: if the aggregate is very coarse such as a coarse sand, it may have a spherical shape, but the increased interlocking due to surface texture may offset the effect of having a lower specific surface, thus workability may remain constant.

The constituents were not measured for shape and texture properties with a specific test, but were related to the study using literature and observations where applicable. For example the HDPE pellets used in this study were observed to have a cylindrical shape and smooth surface thus reducing bonding properties and hence increasing permeability due to the irregular cement aggregate interface.

2.3.2. Concrete mix proportioning

Generally, when proportioning the water, cement, sand and stone quantities for a concrete mix design, the mix must be workable without aggregate segregation in the fresh state, and the required strength, durability and dimensional stability in the hardened state should be attained at minimum cost (Addis, 2008). Various approaches can be adopted for mix proportioning, namely: nominal proportions, table of trial mixes, eye-ball mix design and the Cement and Concrete Institute (C&CI) method of mix design. However, for the purpose of laboratory research which required repeatable results, the C & CI method was the only appropriate method and hence was adopted as the basis for mix-proportioning for this study.

2.3.2.1. Material properties

In order to carry out the C&CI mix design which is based on the ACI standard 211.1-91 (American Concrete Institute, 1997), the following material properties discussed in section 2.3.1, had to be established:

- Relative densities
- Compacted bulk density
- Fineness modulus of sand

Moisture contents of the waste materials were not considered for the mix design and mix water proportions were kept constant, in order to quantify the effect of the waste material moisture properties on concrete properties. This method was similar to that found in literature research by Rao (2010). When moisture contents of aggregates are calculated, the purpose is to adjust the water content in order to achieve the specified slump, for example, when trying to achieve a consistent batch product for commercial use. If moisture content was to be considered in the mix design, the mix would be adjusted as follows as per Neuwald (2010):

Adjusted mass material = Mass of material (Stone/sand/waste) x moisture content

Adjusted Water content = initial specified water quantity-
$\sum(\text{mass of aggregate x surface moisture \%})$

In terms of the effect of moisture on concrete properties, if the moisture content is less than the aggregate moisture absorption, then the aggregate will absorb water from the mix and hence decrease W/C ratio and increase strength. However, if the moisture content of the material was greater than the moisture absorption of the material, then the excess free water would be added to the mix and hence strength would be decreased due to the increased W/C ratio (Neuwald, 2010). Even though mix water proportions were not adjusted to cater for absorption properties of the aggregates for reasons mentioned above, the effect of variations to moisture content was investigated in this study. This was done to demonstrate an understanding of aggregate moisture states by testing samples for compressive strength and workability using aggregates at dry state and comparing the results to samples with aggregates at natural state (See section 4.11).

2.3.2.2. **Water: cement ratio**

The W/C ratio effect has an influence on the hydration of concrete and hence the mechanical properties of concrete (Addis, 2008). The W/C ratio is selected to meet a certain strength requirement based on Abrams law, which states that the strength of concrete is inversely proportional to the water-cement ratio (Chen & Lui, 2005). Therefore, generally the lower the water-cement ratio, the stronger the hardened concrete, provided there is sufficient water to hydrate the cement effectively (Addis, 2008). The W/C ratio has to therefore be balanced to suite the concrete application, as too little water (low W/C ratio) can lead to an unworkable mix, whereas if the W/C ratio is increased, the mix becomes more workable due to increased lubrication and fluidity around aggregates. However, if the free water is in excess, the porosity is increased because of additional capillary pores formed by the increased water volume upon drying. Excess free water also increases the space between calcium silica hydrates (C-S-H), thus lowering interlocking potential (*see Figure 2-36)* and when coupled with the increased porosity mentioned, the concrete strength is reduced (Addis & Owens, 2001) (Harrison, 1992). The W/C ratio is therefore kept as low as practically and economically viable and a ratio of between 0.45 to 0.8 is typically used for conventional concrete (Addis & Owens, 2001).

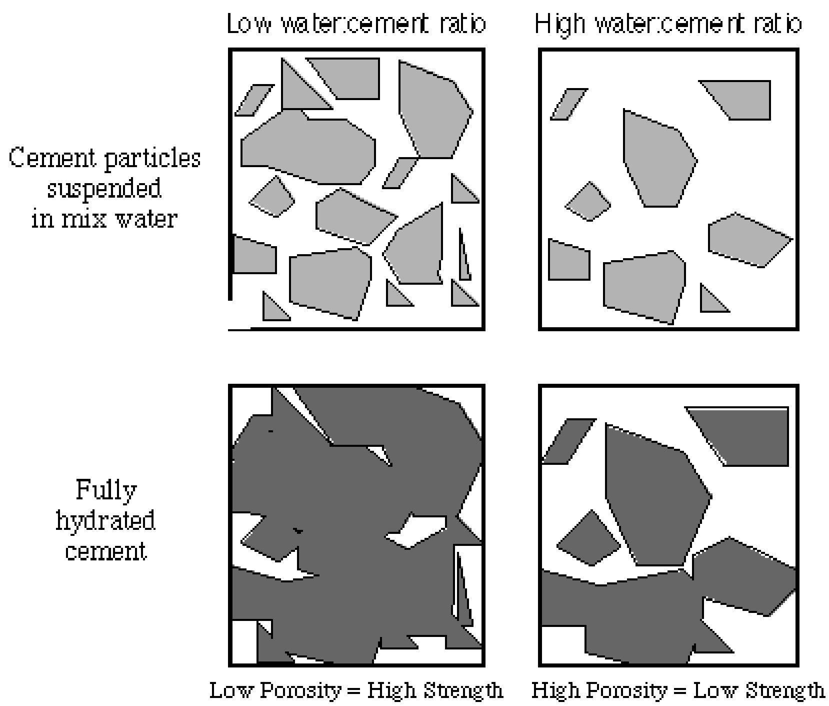


Figure 2-36: Relation between porosity and strength (MAST, 2014)

2.3.2.3. **Water content**

Only potable water should be used, to limit impurities that may contaminate the mix (Iugaza, 2014). The book, “Fundamentals of Concrete” by Addis (2008), states the recommend water content values based on the stone sized used. Water content determines the amount of free water present in the mix which influences properties such as workability, porosity and strength (Neuwald, 2010). Changes in water demand can occur due to varying material properties (shape, surface area, texture, absorption) (Addis, 2008).

For example, a decrease in the slump, would imply an increase in water demand as more water would be required to achieve a specified slump. Changing the water content to achieve a specified slump is necessary for batch plants to keep results consistent (Afrisam, 2012). For example, the contractor will specify the required strength and slump and the batch plant would have to adjust the constituent proportions of the mixes to cater for variations in aggregate types, moisture conditions, etc., to meet these requirements.

Based on findings by Rao (2010), by not adjusting water content, the effect of varying material can be quantified in terms of the effect on concrete properties, for research purposes.

2.3.2.4. **Stone content**

The stone content used for a mix is dependent on the packing capacity of the stone, the vibration method, the desired slump and the fineness of the sand used. The compacted bulk density gives an indication of the packing capacity of the stone used and this is related to the fineness of the sand used to yield a specified concrete workability. If the sand is coarse then a lesser quantity of stone would be required than for a finer sand, so the mix is not too harsh.

The stone quantities for a conventional mix as per the C&CI method, are based on the stone size, fineness modulus (FM) and compacted bulk (CBD). Using the formula below (Addis, 2008):

Stone content = CBDst (K-0.1FM) [kg/m^3]

Where:

CBDst = Compacted Bulk Density of stone [kg/m^3]

K = Factor depending on workability and nominal stone size

FM = Fineness Modulus of sand

The K value used in the formula, is an empirical factor that approximately yields a specified slump for the type of stone and degree of vibration used. Table 2-11, extracted form “Fulton’s Concrete Technology” by Addis & Owens (2001), shows the K-Factors used for varying stone sizes in relation to approximate slump ranges.

Table 2-11: K – factors for stone content formula (Addis & Owens, 2001)

Approximate slump range,	Placing requirement	K				
		Maximum size of stone, mm				
		9,5	13,2	19,0	26,5	37,5
75 - 150	Hand compaction	0,75	0,84	0,94	1,00	1,05
25 - 100	Moderate vibration	0,80	0,90	1,00	1,06	1,10
0 - 25	Heavy vibration	1,00	1,05	1,08	1,10	1,15
60 - 125	Pumped	-	0,83	0,86	0,87	-
25 - 50	Concrete roads*	-	-	-	-	1,2
* Calculated on CBD of 37,5-mm stone when using a blend of 37,5- and 19-mm stone.						

2.3.2.5. **Sand content**

Sand is used as a filler material in the mix (Addis, 2008). As per "Fulton Concrete Technology" (Addis & Owens, 2001), the sand content is calculated using the absolute volume method. The absolute volume method uses the deficit between the sum of the other materials (stone, cement, water) and 1000 litres (equivalent to 1 m^3) to give the sand content of a mix. It was noticed that the fineness of sand is not considered in the absolute volume method of determining sand content, however it is indirectly considered in the stone content formula discussed in section 2.3.2.4. For example, the coarser the sand, the lesser the stone content and therefore the higher the sand content from the absolute volume method.

2.3.2.6. **Adjustment of trial mix**

The trial mix is used as mix proportion acceptance criterion as per "Fulton Concrete Technology" (Addis & Owens, 2001). The cohesiveness of a trial mix is assessed using a slump test, which must show reasonable cohesion and fall within the recommended error margins stated in Table 2-12, extracted from the "Afrisam Concrete Technical Reference guide" (Afrisam, 2012), for it to be accepted.

Table 2-12: Slump tolerance (Afrisam, 2012)

Specified slump, mm	Tolerance, mm
50 and less	-15 to +25
More than 50, up to 100	± 25
More than 100	± 40

If the mix is rejected for example, if there is a lack of cohesion, then the design has to be modified and re-tested by reducing stone or increasing the sand quantity. If not, the proportions are accepted.

2.3.2.7. **Waste proportioning**

To proportion the waste content of mixes, the method of volumetric substitutions of aggregates and cement was found to be used by researchers such as Al-Manaseer & Dalal (1997), Rahman, *et al*.(2012) and Sivarja, *et al*. (2010). The method is based on replacing the conventional constituents (stone or cement) concerned with an equivalent volume of waste based on the percentage of material substitution used.

2.3.3. Workability

The workability of concrete is the consistency, mobility and relative ease at which fresh state concrete can be placed, compacted and finished without the individual constituents separating (Addis, 2008). If the concrete is not workable and cannot be properly compacted, this will result in increased voids in the mix and hence a reduction in strength and durability.

The workability required is dependent on the concretes application and tests such as the slump test give an indication of how workable a concrete mix is (Addis, 2008). For example, a slump of between 100 mm to 175 mm would be suitable for sections with congested reinforcement that require a workable mix, whereas roads require a less workable mix with a slump of between 0 mm to 25 mm. Generally a slump of between 50 to 100 mm is used for normally reinforced concrete sections.

Typical slump classifications are shown in Table 2-13 and the applications generally associated with the respective slump classes are shown in Table 2-14.

Table 2-13: Slump classification table (Paving Expert, 2014)

Class	*Slump range*	*Target slump*
S1	10 ~ 40	20
S2	50 ~ 90	70
S3	100 ~ 150	130
S4	160 ~ 210	180
S5	210 ~ n/a	220

Table 2-14: Concrete applications based on slump classes (Aggregate Industries, 2014)

	Application
S1	Kerbs, pipework bedding
S2	Strip footings and cast in-situ hard-standing slabs, Normally specified for most concrete applications
S3	Trench-filled foundations where a high flow is required, thin sections
S4 & S5	Specialist applications

The material properties and proportioning of raw materials have a significant influence on workability. For example, using a rounded particle, such as the HDPE pellets use in this study, would result in better workability than an angular particle, which would produce a more cohesive but less workable mix (See section 2.3.1 for the discussion of material properties). Free-water has a lubrication effect by separating particles with a water film thus reducing inter-particle friction and increasing workability (Weymouth, 1933). Too much water content can however lead to segregation, lowered cohesion and decreased strength (See section 2.3.2.2 for the discussion on W/C ratio which relates water proportion to workability).

To investigate the effects of waste material properties on workability, the slump test mentioned above was used for this study. The slump test is widely used because of its simplicity and the fact that slump results can be obtained without the need for skilled operators to conduct the test (Addis, 1986). The slump test basically involves placing a sample of concrete in a conical mould tamping the sample then removing the conical mould to measure the amount the concrete subsides. This is referred to as the *"slump"* (Addis, 1986), as shown in Figure 2-37 (See section 3.4 for full test method description). The slump test is suitable for plastic to cohesive concrete with an expected slump of 5mm -175 mm. Beyond this, tests such as the flow test would be more appropriate (Addis & Owens, 2001). The test method used in this study for the slump tests was based on SANS 5862-1 (SABS, 2006) (see section 3.4).

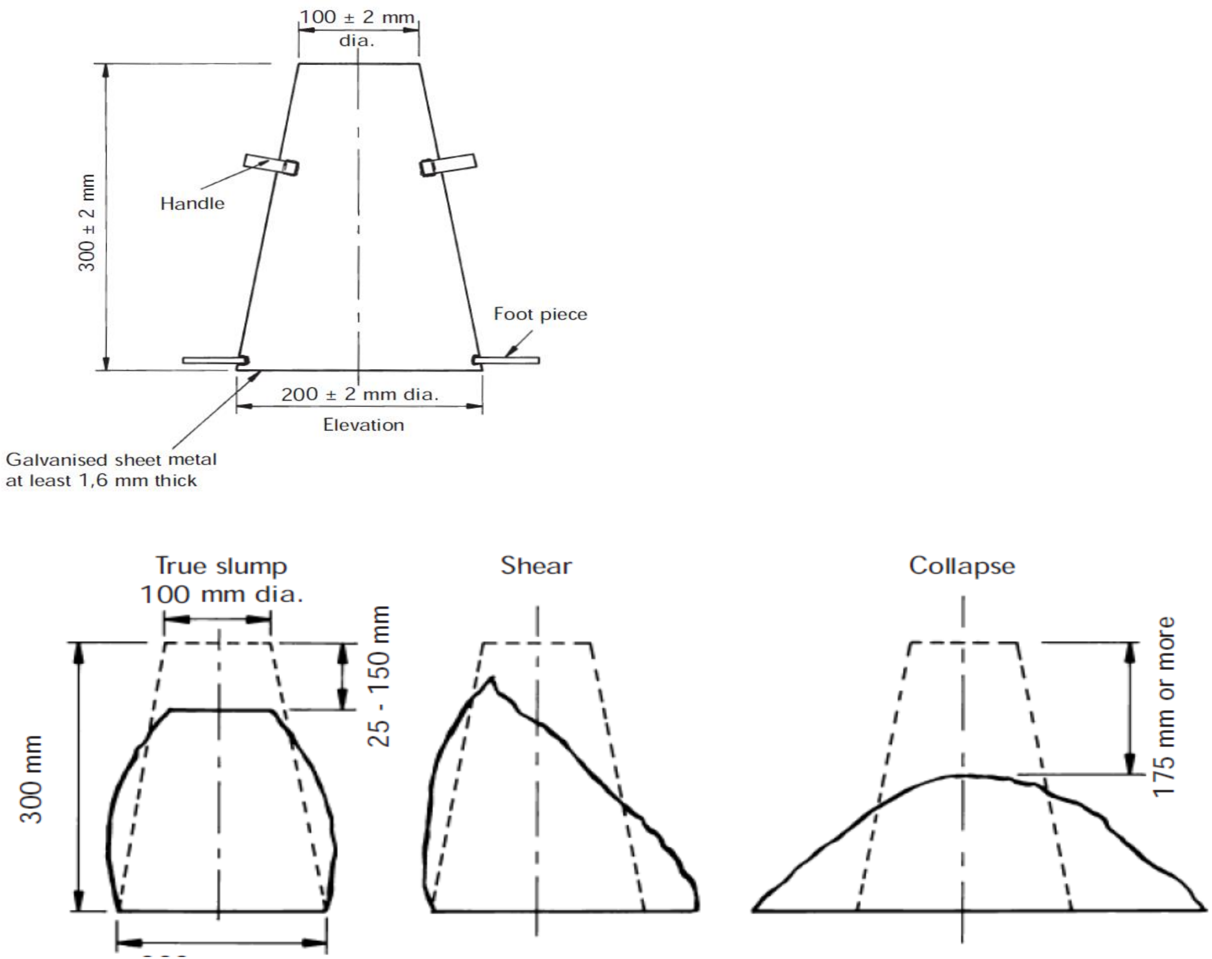


Figure 2-37: (Top) Slump cone, (bottom) measuring slump (Addis & Owens, 2001)

2.3.4. Saturated Hardened Density

The hardened density of concrete is relevant to many facets of concrete such as durability, strength, thermal properties and permanent loading on structures. The density of conventional concrete, as per literature is 2400 kg/m^3 (Elert, 2001), which was used as a reference in this study to compare the results of the conventional concrete mix.

The density of concrete is largely influenced by the properties of the aggregates because they constitute 65% to 75% of the concrete mix by mass (MAST, 2014). For example, in research carried out by Al-Manaseer & Dalal (1997), the use of plastic aggregates, which were lighter than the stone they partially replaced, showed a decrease in the density of the concrete mix. The cement paste also has an influence on the density of concrete because as the hydration reaction proceeds though curing, hydration products grow and occupy void spaces, therefore increasing density (Lamond & Pielert, 2006). Density results were therefore related to the compressive strength and permeability of the concrete because of the inverse relationship between pore volume that exists.

The saturated hardened density measured in this study was calculated using the formula shown below:

Density = $\frac{Mass\ (kg)}{Volume(m^3)}$ [kg/m^3]

The density results in this study were discussed by investigating the changes in density from 7 days to 28 days as well as the effect of changes to the proportions of respective wastes in the concrete mix (see section 4.5). The test method used for measuring the saturated hardened density in this study was SABS 863-1994 (SABS, 1994) (see section 3.6 for test method).

2.3.5. Strength properties of concrete

The strength of concrete, being the maximum load it can withstand, is key criteria when defining its use and is of fundamental importance to structural designers, in particular, compressive strength. Moreover, excessively strengthened concrete can become uneconomical hence it is key to find the balance between strength and cost. The compressive strength of fully compacted concrete is largely dependent on the water to cement ratio (Amsterdam, 2000). The effects of varying W/C ratio were not investigated in this study, because, as stated in section 2.3.2.2, this study focused on showing the effect of waste aggregates in varying proportions on selected concrete properties and not the effect of varying W/C ratios using concrete with waste aggregates in the mix. The implied effects on W/C ratio due to material properties were however discussed where appropriate, such as the effect of moisture content on concrete properties discussed in section 2.3.10.

Concrete is inherently brittle and the failure of concrete is always in tension when the weakest of the aggregate, cement paste or aggregate paste interface fails (Addis, 2008). The aggregate paste interface is generally the weakest point of failure and can be improved by increasing the roughness of the aggregate, decreasing the W/C ratio, using pozzolans like ash and improving tensile strength with fibres (Addis, 2008).

This study gauged the effect on strength due to adding waste materials (HDPE, SCBF, and BA) to the mix by testing the respective specimens for compressive strength, flexural strength and splitting strength.

2.3.5.1. **Compressive strength**

Concrete is designed to withstand compressive loads as concrete is strong in compression. The primary intrinsic factors that affect the compressive strength of concrete are the strength of the matrix, the interface strength, aggregate strength and the interaction between the paste and the aggregate. Alexander (1965), had proposed that failure under compressive load is a combination between tensile failure parallel to the direction of compressive load and the sliding failure between the mortar and coarse aggregates, where the interface is at a sufficiently small angle relative to the direction of compression. The interface between aggregates and the cement paste is also under a higher stress than the rest of the mix, as shown by Newman (1965), who investigated the stress distribution under compressive loading. He discovered that the stress at the aggregate and cement (paste) interface is 2.8 times the average stress experienced by the specimen as a whole. The increased stress at the aggregate-paste interface and the mode of failure, described by Alexander (1965), explain the failure mechanism in compression, which involves the propagation of random cracks from the transition zone which link up with micro-cracks in the cement paste and spilt the specimen as shown by Figure 2-38.

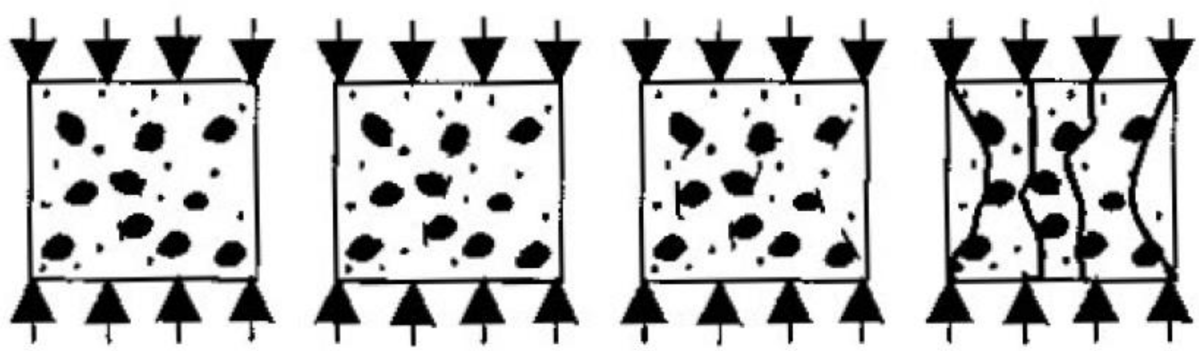


Figure 2-38: Compressive failure mechanism

As mentioned, for failure to occur, micro-cracks from the aggregate-pate interface must link up with micro-cracks in the cement paste. The strength of the cement paste depends on the porosity, W/C ratio and age of the mix (Addis, 1986). Hoff *et al*. (1980) deliberately added voids to a conventional concrete mix to investigate the effect of porosity on strength. It was concluded that a 5% increase in porosity roughly decreased the compressive strength of the mix by 50%. The relationship between porosity and strength could be explained by the fact that because of the increased water filled voids in the paste, cement particles were spaced further apart and there were less nuclei for the hydrates to grow from (see Figure 2-36). There was subsequently a larger spatial gap between particles and this reduced strength, because the mix is dependent on the interaction between hydrated cement particles either physically (interlock) or chemically (van der Waals forces) (Popovics, 1998).

Concrete compressive strength is generally classified according to the Table 2-15 :

Table 2-15: Compressive strength classification (AfriSam, 2008)

	Compressive strength	Applications
Low-strength	15 MPa	Unreinforced foundations, blinding
Medium strength	25 MPa	Reinforced foundation, slabs, driveways, light duty floors
High-strength	30 MPa	Structural beams and slabs, pre-cast items, heavy-duty floors

To evaluate compressive strength, in South Africa, the cube test is used in accordance with SABS 863 (SABS, 1994). However, in America the cylinder test is used. The correlation between the results of these two tests is approximately:

$f_{ck} = 0.8\ f_{cu}$

Where f_{cu} = cube strength and f_{ck} = cylinder strength

In the context of this study, cubes are loaded in compression and once the test specimen has failed, the compressive cube strength at failure in Mega-Pascal (MPa) is given by the load at failure divided by the cross-sectional area, as shown by the formula below.

$$f_c = \frac{F}{A}$$

f_c = compressive strength [MPa ($\frac{N}{mm^2}$)]

F = load at failure [N]

A = cross-sectional area of test cube [mm^2]

Tests on compressive strength have been shown to be sensitive to changes in mix materials and mix proportions (Afrisam, 2012), which are two aspects investigated in this study. The test method used in this study for compressive strength was the following: SABS 863-1994 (SABS, 1994) (see section 3.5.1.1 for method).

2.3.5.2. **Flexural strength**

Concrete is weak in tension and when concrete fails, it fails in tension. The tensile strength in a broader sense of design is also related to the cracking behaviour of concrete (Addis, 1986). Tensile strength can be measured directly or indirectly. However, difficulties associated with the direct tension test resulted in the adoption of the flexure or splitting tests to measure the tensile strength of concrete (Addis, 1986). Figure 2-39 shows that tensile strength and compressive strength have been shown to be proportional to each other (Addis, 1986). The reason for this is probably because as the hydration reaction takes place, pores within the matrix are blocked and hence this restricts crack propagation paths improving tensile and compressive strength. Flexural strength is generally 10% to 20% of compressive strength (NMRCA, 2000) and ranges from 3 to 5 MPa (conventional concrete) (The Engineering Toolbox, 2014).

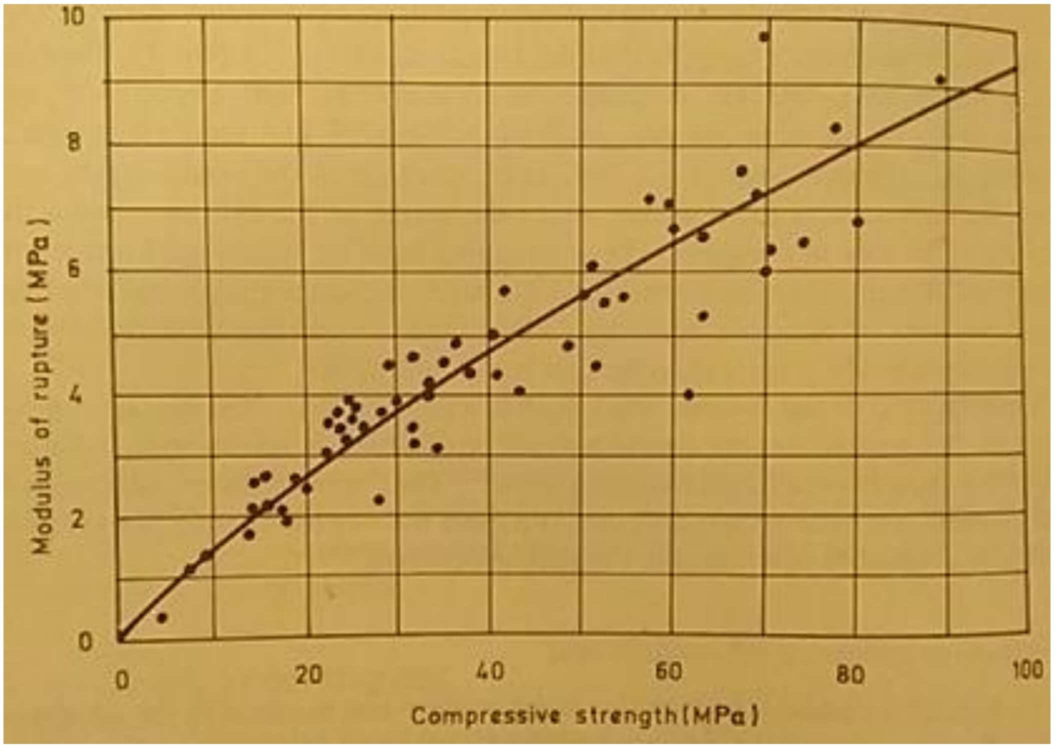


Figure 2-39: Relationship between modulus of rupture and compressive strength (Addis, 1986)

The aggregate properties that mainly affect the flexural strength of concrete are the shape, surface texture and the modulus of elasticity. The elastic modulus of an aggregate is related to tensile strength, because generally the higher the elastic modulus the higher the tensile strength of a mix due to a greater resistance to deformation (The Constructor, 2014). The critical factor in terms of tensile strength, however, as with compressive strength, is the bond between the aggregate and hydrated cement. Therefore, the use of angular and crushed aggregates can improve flexural strength by increasing the surface area of the aggregates used in the mix (Addis, 1986). In relation to this statement, Shah and Slate (1965) also concluded that the bond between the mortar and the aggregate is the weakest link in the concrete and bond-cracks are primarily responsible for the weak tensile strength of concrete. This study utilized waste materials in concrete, one having a smooth surface (HDPE) whereas the other was fibrous in texture (SCBF). The effect on tensile strength due to the varying types of waste was evaluated in this study (See section 4.6.2).

As mentioned, two tests were utilized to evaluate tensile strength in this study, namely: flexural strength and splitting strength. For the flexural strength (modulus of rupture), essentially a beam is supported at two-points and a transverse load is applied which causes flexure in the beam (see Figure 3-15 on page 161 for apparatus layout diagram). The flexural strength at failure is then calculated using the bending moment (PL) and dimensions of the beam, assuming the beam behaves elastically (Addis, 1986). The formula used to calculate the flexural strength (modulus of rupture) extracted from “Fundamentals of Concrete” (Addis, 2008) is shown below.

$$f_f = \frac{PL}{bd^2}$$

f_f = Flexural strength [MPa]
P = breaking load [N]
L = Horizontal distance between support rollers of testing apparatus [mm]
b = width of test specimen [mm]
d = depth of test specimen [mm]

Flexural strength of standard concrete generally ranges from 2 N/mm^2 to 7 N/mm^2 (Schlumpf, et al., 2013). Flexural strength is usually utilised in the design of pavements and beams (NMRCA, 2000). The European Concrete Committee had suggested that splitting strength, which is discussed in the next section (2.3.5.3), is approximately 60% of flexural strength (European Concrete Committee, 1963). These relationships are however dependent on the mix proportions and the type of aggregate and will be investigated in this study (see section 4.6.3). The test method used in this study for flexural strength was the following: SABS 864-1994 (SABS, 1944) (see section 3.6.2 for the test method).

2.3.5.3. **Splitting strength**

For splitting strength, elastic theory was assumed as for flexural strength, which states that solids deform under applied external load and regain their shape after the load is removed (S.Dhar, 2012). In terms of applications, splitting strength can be used to relate to the shear resistance of beams.

For the splitting test, the cube specimens were subject to compressive forces applied along two diametrically opposed lines. The tensile stress in the plane that joins the two lines, induced by the applied compressive force, is what causes failure. The formula to calculate the splitting strength is shown below.

$f_s = \frac{2P}{\pi a^2}$

f_s = splitting tensile strength (MPa)

P = compression load at failure (N)

a = diameter of cylinder (mm)

L = length of cylinder (mm)

a = size of cube (mm)

The difference between flexural and splitting strength is that flexural strength gives the tensile strength at the controlled tension surface of the beam, whereas splitting strength indicates tensile strength that can be initiated anywhere in the portion of the plane that is in tension (Popovics, 1998). Flexural strength is usually higher than splitting strength, because the flexural test involves the bending of beams and hence there is some compression involved at the compression face of the beam which provides increased resistance (Oregon State University, 2014). The combination of splitting and flexural strength was used to describe the tensile behaviour of concrete consisting of waste in this study, and relationships were also investigated with compressive strength properties. The test method used in this study for splitting strength was the following: SABS 1253-1994 (SABS, 1994) (see section 3.6.3 for the test method).

2.3.6. Specific heat capacity

2.3.6.1. Background

Specific heat capacity (J/kg °C) is by definition, the amount of energy in joules (J) required to heat a kg of material by 1°K (Àbout.com, 2013). The other important thermal properties of materials are thermal conductivity, which is the ability of a material to conduct heat (W/mK), and thermal diffusivity, which is the relative ability of heat to flow through a material. However, the latter two were not covered in this study due to apparatus not being available for tests to be carried out.

Concrete is a mixture of materials and the specific heat capacity of the mix is the averaged sum total of the specific heat capacities and proportions of the individual components (Current, 2012). Generally, the denser the material, the higher specific heat capacity (Holiday, 2013). An exception to this would be water which is less dense than concrete but has a much higher specific heat capacity (La Roche, 2012). .

An increase in specific heat capacity can improve the temperature stability of a structure as there is a lower temperature rise with a given amount of heat. In the construction industry, specific heat capacity is related to thermal mass, which is the amount of heat a material can store.

When considering the implied effect of specific heat on thermal mass, a high specific heat will allow a material to absorb excess heat during the day, keeping the structure interior cool and during the night the stored heat can help keep the interior of the structure warm. This is known as the "*fly-wheel effect"* (The concrete society, 2013). Thermal mass has virtually no effect however when temperatures are constant on both sides of the material (steady-state heat flow) and no temperature gradient is present (Autodesk Education Community, 2014). Low specific heat materials are generally used for applications that require a quick thermal response such as hot-plates (One-School.net, 2014). Hence a material with a higher specific heat is more relevant to construction in terms of passive heating and cooling measures. Figure.2-40 illustrates how thermal mass works for a house.

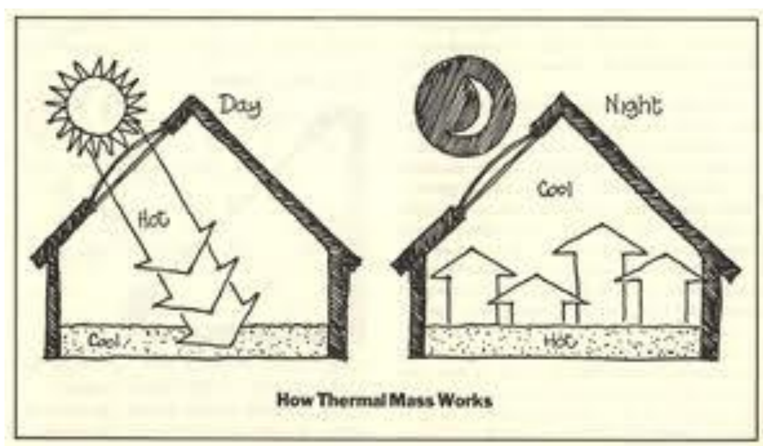


Figure.2-40:Thermal mass diagram (Hibshman, 1983)

Table.2-16 shows the literature reference values used for the thermal experimental analysis.

Table.2-16: Material specific heats

Material	Specific heat (J/kg °C)	Reference
Water	4184	(eHow, 2013)
Concrete	880	(The Physics hypertextbook, 2013)

Cement	840	(Polytee, 2013)
Stone	840	(Polytee, 2013)
SCBF	460	(Shrivastav & Hussain, 2013)
Coal bottom ash	730	(Torrenti, et al., 2013)
HDPE	2250	(Designer Data, 2013)
Air	1000	(The Engineering toolbox, 2014)

Even though specific heat is independent of object mass, it does depend on type of material (Al Faruq, et al., 2013).The high specific heat of HDPE, as shown in Table.2-16, can result in an increase to the specific heat of the concrete mix, which can yield possible thermal benefits as discussed. The effect of specific heat was investigated in this study (See section 4.7).

2.3.6.2. **Specific heat testing theory**

The testing procedure did not follow any international standard due to the instruments and software required being too costly to purchase or not readily available at the time of this study. In order to obtain meaningful data for this study an experimental lab procedure from a Chem139 lab practical for specific heat conducted at Western Carolina University (Western Carolina University, 2013) was followed. Test results were validated by doing the control test in triplicate and comparing the value obtained in literature. An error margin of less than 10% was deemed as acceptable considering the assumptions made and possibility of human error. The modified mix designs were done in duplicate to show repeatable results. Knowledge on heat and mass transfer was also used to support the method and results obtained.

The theory behind the experiment was based on the equation that states:

$Q = mc\Delta T$; where

Q = heat energy [J]

M = mass [kg]

ΔT = change in temperature [C]

If it is assumed that the polystyrene cup is perfectly insulated and energy is constant, then the energy of the cup with water can be equated to the energy of the cup with water and the heated specimen. The negative sign indicates that the heat energy is being lost by the sample and gained by the water.

$Q_{sample} = -Q_{water}$

And hence;

$m_s c_s (100-t_f) = m_w c_w (t_f-t_i)$

$c_s = \frac{m_w c_w (t_f - t_i)}{m_s (100 - t_f)}$ [J/kg °C]

m_s = Mass of sample [kg]

m_w = Mass of water [kg]

t_i = initial temperature of water [°C]

t_f = final temperature of water and sample. [°C]

In order to evaluate results, it was assumed that the specific heat of the individual materials involved may give an indication of the effect on specific heat of concrete. This hypothesis was supported by literature by Howlader *et al.* (2012), which showed that the change in aggregate specific heat can have an impact on the concrete specific heat. The book *"Significance of testing and properties of concrete and concrete aggregates"* (ASTM, 1956) also showed that the specific heat of aggregates have an influence on the specific heat of the concrete, further proving that the hypothesis is plausible.

2.3.7. Elastic modulus (Young's modulus) of concrete

The Young's modulus is a value that gives an indication of the deformation resistance of the hardened concrete specimen by quantifying elastic stiffness in terms of the relation between influencing stress and the resulting deformation (Addis & Owens, 2001). The higher the elastic modulus, the more resistant the material is to deformation.

Concrete is not a perfectly elastic material as the stress-strain behaviour of concrete is not entirely linear and does not completely obey Hooke's law (Addis, 1986), which states:

"For relatively small deformations of an object, the displacement or size of the deformation is directly proportional to the deforming force or load" (Encyclopedia Britanica, 2014).

However, there is an initial portion of the stress-strain plot, shown in Figure 2-41, that is linear. This is where elastic deformation occurs immediately after a load is applied and is reversible after the stress is removed, thus the specimen returns to its original shape (Addis, 2008). The gradient of this linear portion of the graph is where the elastic modulus (Young's modulus) may be taken and is referred to as the initial tangent modulus. The reason why elastic deformation is not permanent is because the atomic structure is not re-arranged but rather the bonds between the atoms are stretched (Mamlouk & Zaniewski, 1999). When a sample is strained to a point where it becomes irreversible, it reaches a semi-plastic (partly reversible) zone and finally a plastic deformation (permanent) zone. Once in the permanent deformation zone, micro-cracks will develop at the weakest locations within the concrete, being the hardened cement paste and aggregate interface. The tangent and the secant moduli can be used to define elastic modulus at stresses at these regions beyond the linear region of the stress-strain graph, but this was not within the scope of this study.

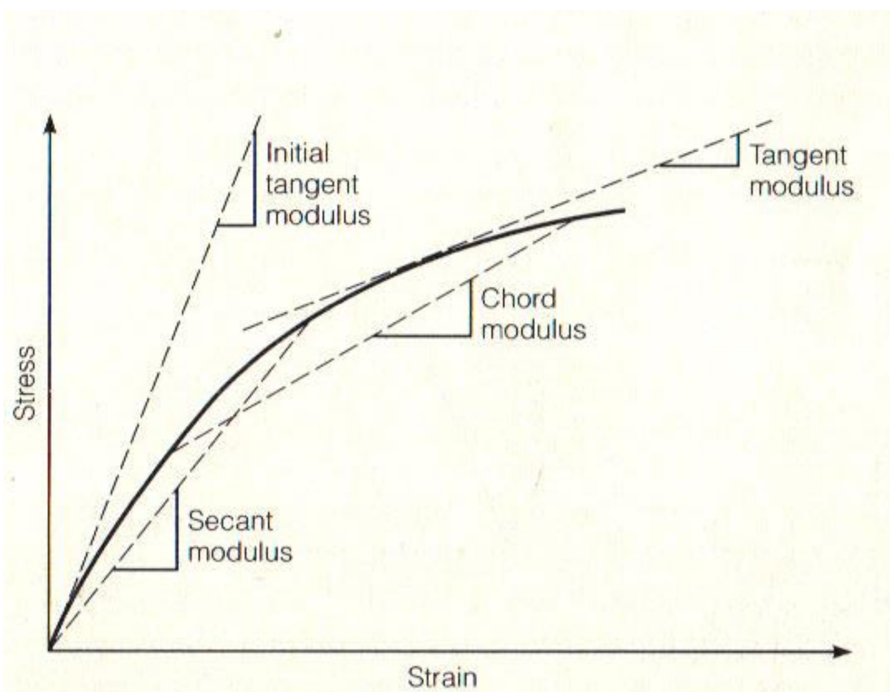


Figure 2-41: Methods of approximating elastic modulus (Mamlouk & Zaniewski, 1999)

Kaplan (1959) tested a range of aggregates and showed that by varying the properties and volumes of aggregates, there is a significant effect on the modulus of elasticity. The modulus of elasticity can be effected by factors such as:

- Characteristics of the hardened cement paste;
- Characteristics of the aggregate cement past interface;
- Aggregates properties;
- Age of the concrete.

In terms of the influence of material properties on the concrete elastic modulus, research by Okajima (1972) on the effect on compressive strength by introducing different types and proportions of aggregates to a cement paste matrix (elastic modulus generally 10-30 GPa), showed that compressive strength always decreases if the aggregate has a modulus of elasticity lower than the matrix. This is probably because the low-modulus aggregate offers less resistance to micro-crack propagation and hence decreases compressive strength. The findings of (Okajima, (1972) were investigated in this study where the SCBF of low-modulus was of particular relevance.

In terms of the aggregate cement interface, it is related to elastic modus in a sense that a good bond between the aggregate and the cement paste allows for a more efficient transfer of load and hence improved resistance to deformation under loading. Therefore, aggregates that have a smooth surface or do not form strong interfacial bonds such as HDPE may result in reduction in elastic modulus due to poor load transfer (Refer to section 4.8 for the discussion on elastic modulus).

With regards to the relationship between compressive strength and elastic modulus, generally the elastic modulus of a mix will increase with an increase in the compressive strength as shown in Figure 2-42 from the investigation carried out by Davis & Alexander (1989). This is possibly due to the hydration reaction over time which fills voids and hence increases the stiffness and compressive strength of the mix.

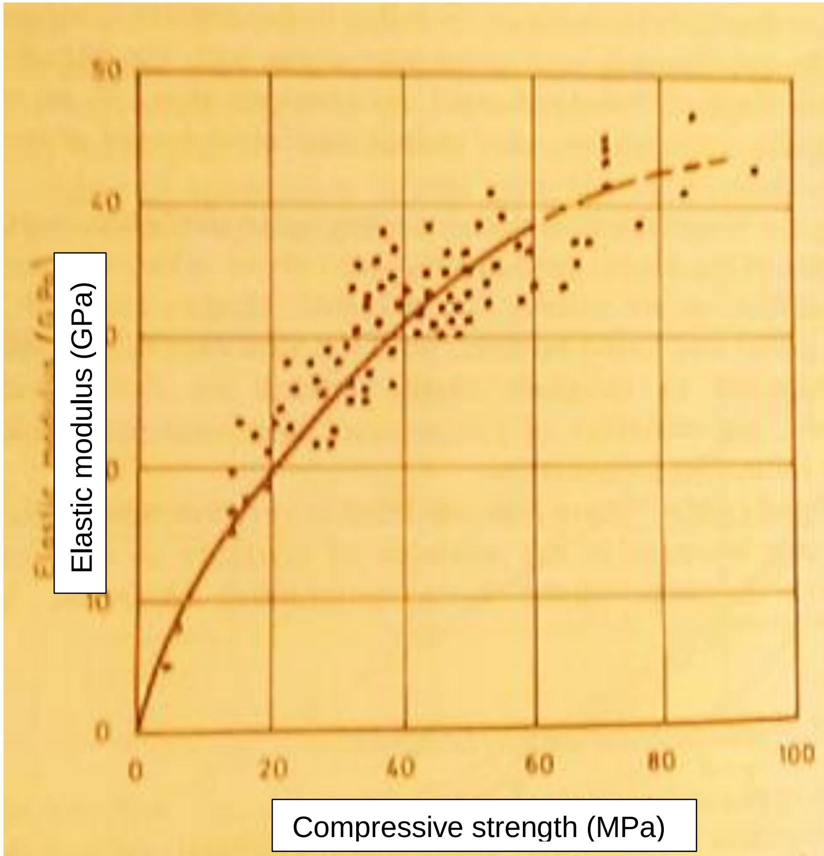


Figure 2-42: Relationship between compressive strength and elastic modulus (Davis & Alexander, 1989)

In terms of defining a relationship between the elastic modulus and compressive strength, the British code BS 8110 part 2 (BSI, 1985) proposed an empirical formula to predict the elastic modulus of concrete as (Addis, 1986):

$E_{28} = K_o + 0.2\ f_{cu28}$

Where:

E_{28} = elastic modulus at 28 days

K_0 = 19 for Coedmore quartzite (used in this study)

f_{cu} = compressive cube strength at 28 days

This prediction was evaluated in this study when comparing the predicted modulus of elasticity to the tested conventional concrete value. The formulas relevance to waste aggregate specimens may not be significant, due to the varying material properties of the wastes compared to conventional aggregates. This was investigated in section 4.8.

In order to calculate the elastic modulus of concrete, either the dynamic or static method can be used. The static test characterizes the deformation resistance against a constantly increasing or constant load and the standard procedure for calculating the static modulus of elasticity is the BS 1881: Part 121 1983 (see section 3.8 for the methodology). The dynamic test essentially involves the exciting of a specimen with a variable frequency oscillator. The oscillations are detected by an amplitude indicator and the frequency is varied until resonance is obtained. The static modulus method was selected as it was the apparatus available at the time of the study.

E = $\frac{Stress}{Strain}$

E = Elastic modulus

Stress (MPa) = $\frac{load}{cross\text{-}sectional\ area}$

Strain (mm) = $\frac{\Delta l}{l}$

Δl = change in length

L = original length

The test method used for measuring the static elastic modulus in this study was based on BS 1881: Part 121 1983 (BSI, 1983) (see section 3.8 for methodology).

2.3.8. Durability of concrete

The durability of a structure determines the time a structure can fulfil the function for which it was designed and constructed by maintaining its impermeability, strength and dimensional stability (Afrisam, 2012). Durability is a performance concept as opposed to an intrinsic material property and refers to the ability of concrete to resist abrasion, chemical attack and weathering action. The importance of ensuring the durability of concrete is considered when specifying a material, in order to lengthen a structures service life and reduce the cost of replacing damaged structures (Mays, 1992).

Poor durability characteristics can cause concrete structures to deteriorate prematurely due to environmental attack and corrosive damage to embedded re-bar. Environmental attack occurs when salts, oxygen, chlorides or carbon dioxide penetrate through concrete which eventually leads to steel corrosion. Corrosion occurs when there is a change in the pore solution surrounding the re-bar, such as the ingress of chloride ions. When steel corrodes, it expands in volume causing cracking, rust staining and spalling as shown in Figure 2-43 (Mays, 1992).

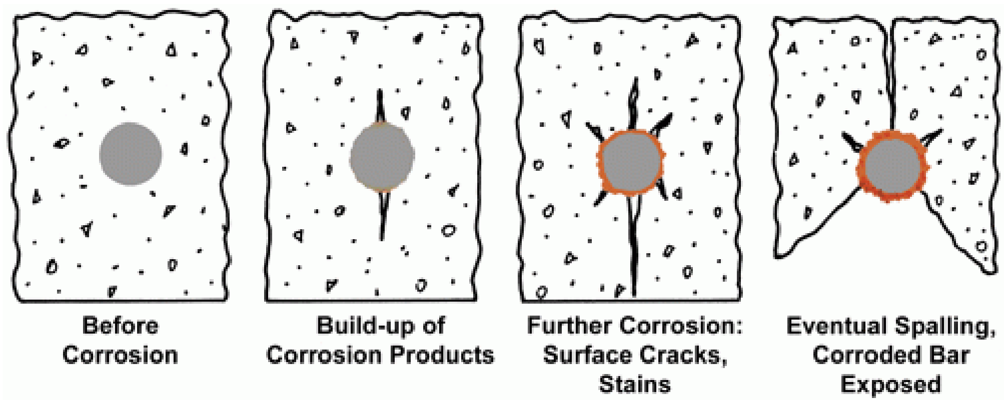


Figure 2-43: Corrosion of steel (Galvanizeit, 2014)

The corrosion protection afforded to the steel reinforcement is dependent on the chemical and physical characteristics of the concrete. In a chemical sense, the high alkaline hydration products present in the pore moisture of concrete form a protection film of gamma ferric oxide around the steel to ensure complete electrochemical passivity. As long as the gamma ferric oxide layer is maintained by a sufficiently high pH, then the steel will be protected. The pH of concrete can be reduced through carbonation and leaching out of calcium hydroxide, the rates of which are dependent on the permeability of the concrete (Rehm & Moll, 1960).

The risk of corrosion can therefore be decreased by reducing the permeability of the concrete and increasing the cover to re-bar:

- Durability can be considered as being inversely proportional to permeability (Addis, 1986). Liquids and gases can permeate through capillary pores in the cement paste, pores and cracks in the aggregate and channels and cavities within the concrete matrix. Capillary pores form when there is excess free-water in the mix that dries out and leaves a system of voids (Mays, 1992). The permeability of concrete can be improved by the introduction of a pozzolan such as BA which can reduce void spaces due to the pozzolanic reaction which reduces void spaces in the mix (Investigated in this study – section 4.9).

- The cover to re-bar is intended to be a barrier between the environment and the re-bar. It is the layer of concrete that is most susceptible to early-age drying and penetration of aggressive agents from the environment. Therefore, the quality of the cover layer can be said to control the durability of the concrete.

The durability index, which was utilized in this study, was developed to provide a more practical approach to assessing the durability properties of concrete, by considering parameters that influence the deterioration process in the cover layer (Alexander, et al., 2009). The three transportation mechanisms (permeation, absorption and diffusion) were measured using the oxygen permeability, water sorptivity and chloride conductivity tests respectively. The transportation mechanisms were influenced by factors such as capillary porosity, interconnection between capillaries and the degree of hydration. Fluid transport decreases with increasing hydration and decreasing W/C ratio as shown in Figure 2-44.

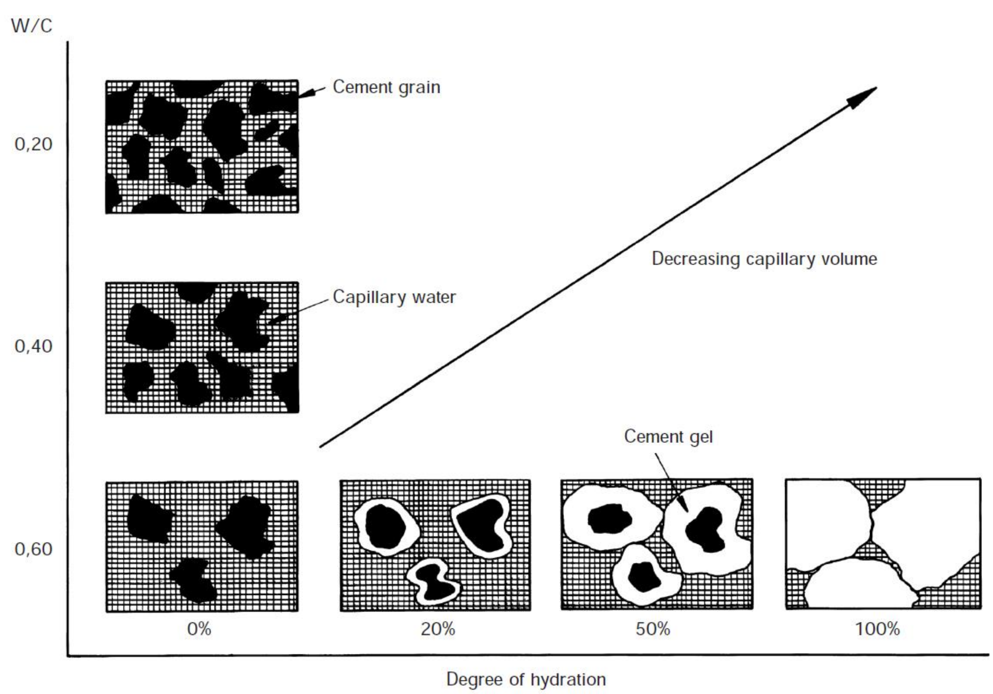


Figure 2-44: Relationship between the degree of hydration and W/C ratio (Addis & Owens, 2001)

In order to apply the durability index as a practical guideline, Alexander, *et al*. (2009) suggested limits for durability index values shown in Table 2-17 , for the OPI, Sorptivity and CC tests (Refer to sections 2.3.8.1 to 2.3.8.3 for the theoretical background of each test).

Table 2-17: Table of suggested durability parameters (Alexander, et al., 2009)

Durability class	OPI, log scale	Sorptivity, mm/-h	Conductivity, mS/cm
Excellent	> 10	< 6	< 0.75
Good	9.5 - 10	6 - 10	0.75 - 1.5
Poor	9.0 - 9.5	10 - 15	1.50 - 2.5
Very poor	< 9.0	> 15	> 2.5

2.3.8.1. **Oxygen Permeability index (OPI)**

The Oxygen permeability index (OPI) is a logarithmic value that is a measure of the pressure decay of oxygen over time, which gives an indication of the permeability of the specimen. Permeability affects the extent of entry and rate of movement of liquids or gases through a porous medium due to a pressure gradient (Mays, 1992). The concrete microstructure, density and the characteristics of the permeating agent have an influence on permeability. Permeability is the least threatening transport mechanism and can be mitigated by simply reducing the W/C ratio or increasing cementitious content to neutralize the pressure gradient (Hycrete, 2011).

The OPI test is sensitive to macro-voids and cracks and has been to found to be effectively applicable to assess the state of compaction, presence of bleed voids and channels as well as the degree of pore interconnectivity.

The OPI test index formula shown below, is a simplification of the D'Arcy coefficient of permeability (k), which is determined from the logarithmic of the ratio of pressure vs. time.

$OPI = -\log_{10}k$

The oxygen permeability index logarithmic values generally range from 8 to 11. The higher the index value the less permeable the concrete (The Concrete Research Portal, 2014). The oxygen permeability test is also more reliable than the water permeability test used by researchers such as Omoniyi & Akinyemi (2013), because oxygen flow through a sample does not react with hydrates or alter the pore structure (Prajapati & Arora, 2011).

The test method used in this study for OPI was the following: SANS 516-2 (SABS, 2009) (See section 3.9.1 for OPI test methodology).

2.3.8.2. **Chloride conductivity (CC)**

Chlorides can enter the concrete microstructure through capillary absorption, permeation and diffusion. The ingress of chloride, which occurs largely in marine environments, can lead to the breakdown of the protective film around steel due to the formation of soluble ferric chloride (Evans, 1948). It is for this reason that the permeability and the cover thickness to steel are critical in preventing corrosion (Mays, 1992). When chloride ions react with Fe $(OH)_2$ as per the equation below, this will depassivate the steel in isolated spots.

$$Fe(OH)_2 + 6Cl^- \leftrightarrow FeCl_6^{-3} + 2OH^- + e^-$$

The complex ferric chloride produced is soluble in the pore solution and hence reduces the corrosion protection on the steel surface (Hurley & Scully, 1999). Once steel is depassivated it corrodes through electrochemical processes.

Diffusion is the process by which a liquid or gas moves through a porous material due to a concentration gradient. Diffusion is a slow process and hence a rapid chloride test was developed in South Africa by the University of Cape Town and Witwatersrand (Streicher & Alexander, 1995), to provide results after a short duration (24 hours) (Benhausen & Alexander, 2008). The test has also been shown to have a good correlation with long-term diffusion testing.

Conductivity is linearly related to diffusivity, as per research by (Streicher & Alexander, 1995), which is the property that governs the rate of chloride ion penetration. Conductivity is an electrochemical process that reflects the ease at which ions can move unimpeded through a concrete specimen. The test measures the conductivity of the concrete specimens in milliSiemens per centimetre (mS/cm) and this gives an indication of the resistance of concrete to the ingress of chloride ions. Conductivity is mainly dependent on the interfacial zone and pore structure of the mix (Ganesan, et al., 2007) (Yuksel & Demirtas, 2013). This was substantiated by Poon *et al* (2000), who showed that fly-ash could lower chloride conductivity due to the improved interfacial bond between the aggregate and the cement paste. However, in terms of bottom ash, research by Basheer & Bai (2003) reported that chloride permeability increased because of the porosity of BA.

Due to the electrochemical nature of the chloride conductivity test, the electrical resistance properties of the aggregates may also have an influence on the conductivity results (Alexander, et al., 2009).

Typical chloride conductivity values range from 0.75mS/cm to 3mS/cm. Permeability and chloride conductivity have been shown to have a reasonably linear correlation.

The test method used in this study for CC was the following: SANS 516-4 (SABS, 2009) (See section 3.9.2 for the CC test methodology).

2.3.8.3. **Water sorptivity (WS)**

Sorptivity is defined as the rate of movement of a wetting front through porous material and it does not require an external pressure head to occur. The water sorptivity test measures the uni-directional mass of absorbed water from the bottom face of the sample in mm/root hour. This indicates the degree of water movement under the influence of capillary action. Capillary absorption can cause the most potential damage out of the three transport mechanisms and transports water approximately 1 million times faster than pressure permeability (Hycrete, 2011). Capillary action is also independent of a pressure gradient and reducing the W/C ratio or increasing cementitious content would not ensure a decrease in WS, such as with permeability. The strength of capillary forces exerted by the pore structure to draw water into the concrete, is dependent on the pore geometry and saturation level of the specimen. A linear relationship exists between the mass of water absorbed and the square root of time. Therefore the sorptivity index is subsequently obtained from the slope of the straight line produced (Alexander, et al., 2009).

Water absorption due to wetting and drying is an important fluid transport mechanism near the concrete surface (cover area), but becomes less significant with depth. The WS values can vary from 5mmh to 20mmh. The lower index value, the better potential durability because a low value implies lesser pores in the mix due to lower capillary suction.

The test method used in this study for WS was the following: SANS 516-3 (SABS, 2009) – see section 3.9.3 for WS test methodology.

2.3.9. SEM imaging analysis

For the purpose of enriching this study, scanning electron microscope (SEM) imaging was conducted to show the interface between HDPE and SCBF with cement as well as BA particles on a microscopic level.

Scanning electron microscopy (SEM) provided detailed high resolution images by detecting secondary or backscattered electron signals from an electron beam. SEM can reach magnification of up to X50000. SEM imaging is mainly used for characterisation of particulates and defects, examinations of grain structures and coating thickness measurements (Ceram, 2013).

Figure 2-45: Zeiss Ultra Plus FEGSEM

The SEM apparatus was however limited to small samples of approximately 5mm^2. This meant that only the aggregate interaction between HDPE and SCBF with the cement mix and areas of interest for the BA sample were investigated with the SEM and not the entire concrete matrix due to spatial constraints.

In order for the SEM imaging to add value, literature was consulted to identify what images are of interest when an SEM is conducted for concrete.

Figure 2-46 shows typical Calcium-Silica-Hydrate (C-S-H) imaging from literature. It shows that C-S-H gel grows in the form of flaky crystals from the hydration of tricalcium-silicate (C_3S).

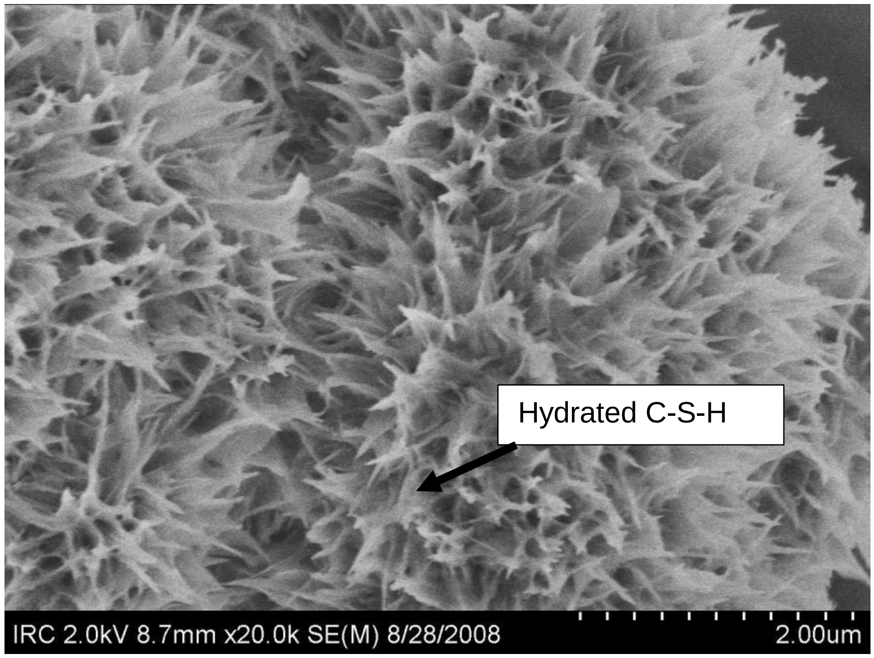


Figure 2-46: C-S-H gel (Cement Lab, 2011)

Figure 2-47 shows Calcium Hydroxide (C-H) with a distinguished hexagonal shape on the surface of tricalcium-silicate (C_3S).

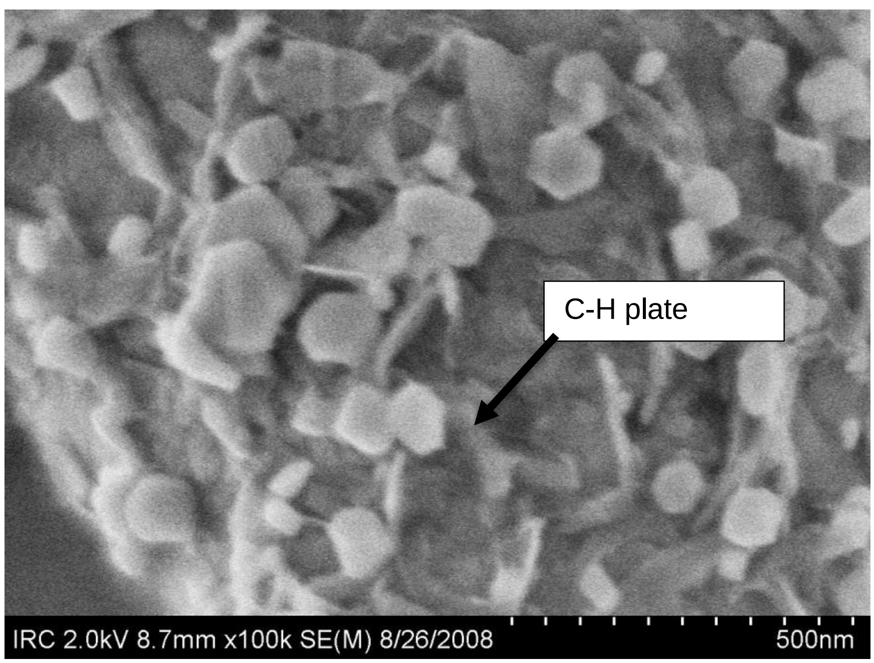


Figure 2-47: Calcium Hydroxide plates (Cement Lab, 2011)

Figure 2-48 shows a magnified image of the interface between C-S-H and C-H.

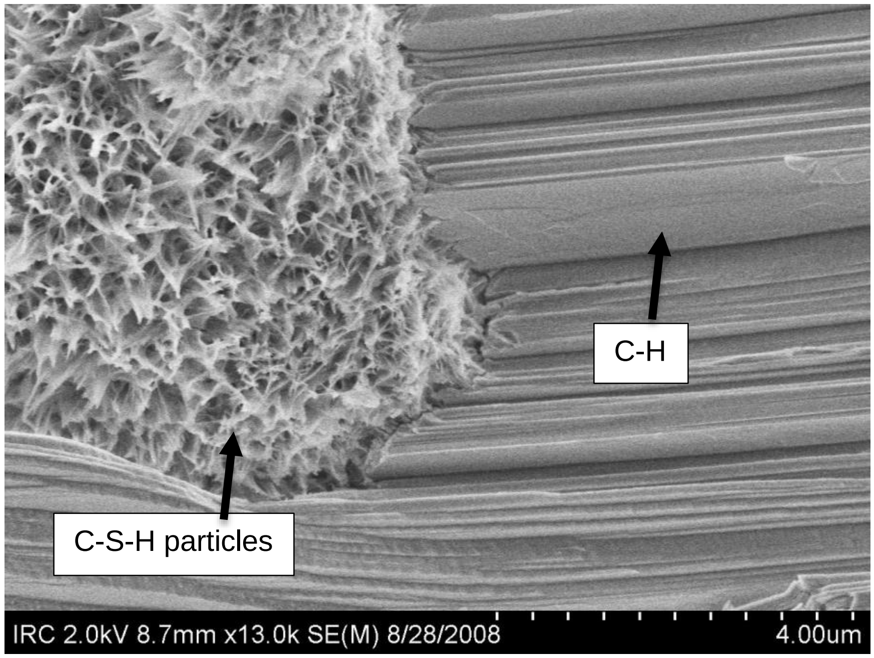


Figure 2-48: Interaction between C-S-H and C-H (Cement Lab, 2011)

Figure 2-49 shows the surface of bagasse fibre covered with C-S-H.

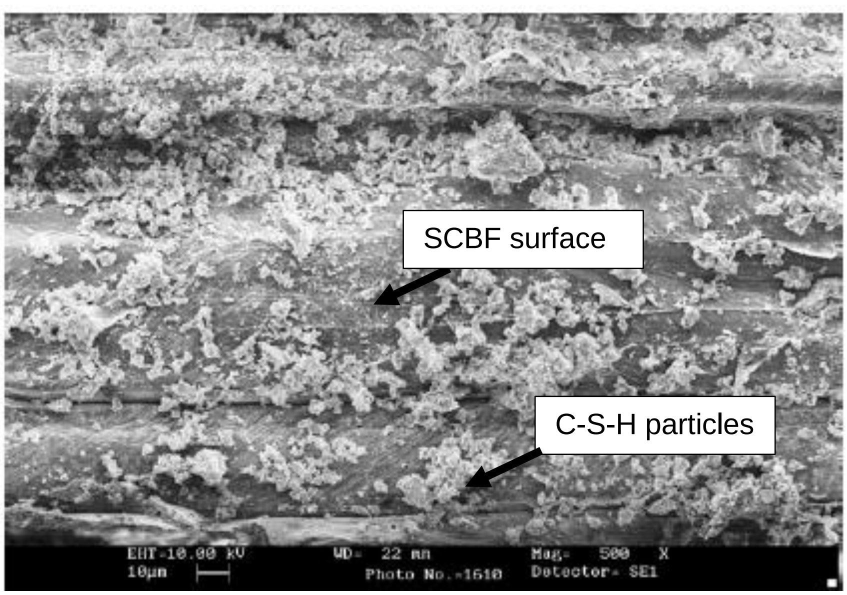


Figure 2-49: Surface of bagasse fibre in concrete (Sivarja, et al., 2010)

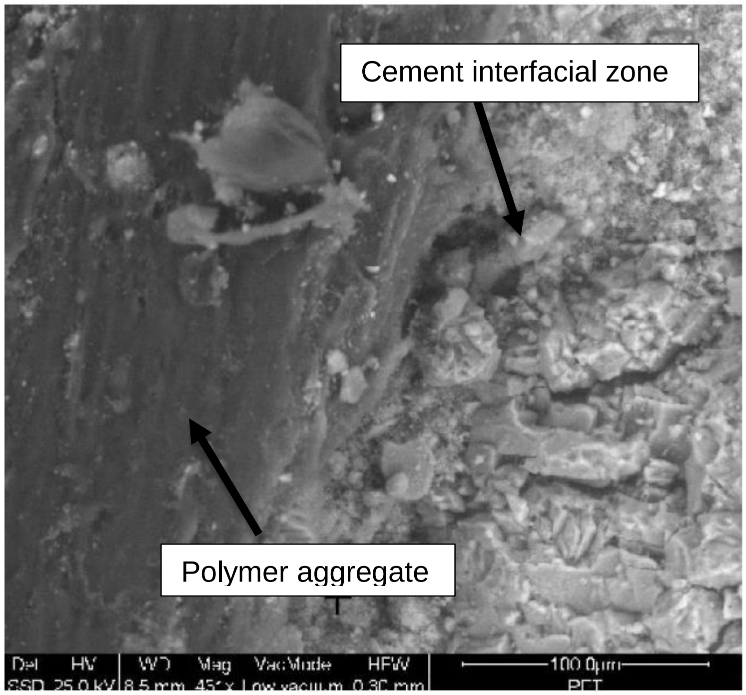


Figure 2-50: Polymer aggregate-mortar interface (Gavela, et al., 2003)

The typical SEM image of an ash particle from literature is shown in Figure 2-51.

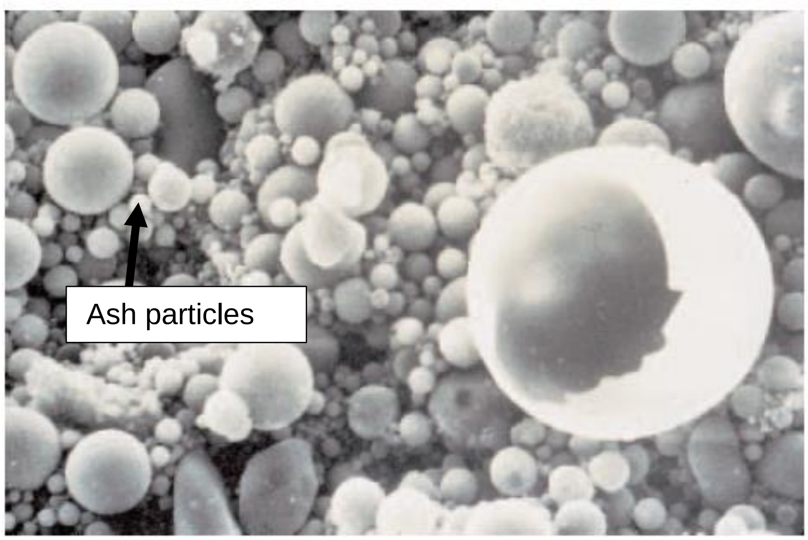


Figure 2-51: Ash particle SEM image (University of Memphis, 2014)

The literature images shown were used as references for the SEM analysis done in section 4.10.

2.3.10. Effect of waste aggregates moisture properties on concrete

The moisture state and moisture absorption properties of an aggregate can have a bearing on concrete properties such as W/C ratio and workability (Sallehan & Mahyuddin, 2014). Aggregates contain pores and the extent to which these pores are filled with liquid determines the absorption of the aggregate. The moisture state of an aggregate refers to the degree of pore saturation and potential absorption.

There are four recognised moisture states for aggregates:

- **Oven dry:** All moisture is removed from the aggregate's pores by oven drying for 24 hrs. At this state the material has its maximum potential absorption and would yield the greatest reductions in W/C ratio in the interfacial zone and workability of the mix, compared to the other moisture states.
- **Air dry:** The surface of the aggregate is dry but there may be water present in the pores. This state is also considered as natural state. The materials moisture content is less than its moisture absorption and it can still absorb water from the mix, but less than at oven dry- state.
- **Saturated surface dry:** Pores are completely filled with water under lab conditions but no free water exists on the surface of the aggregate. At this state, the materials moisture content is equal to its moisture absorption and it cannot absorb more water from the mix.
- **Damp/wet:** All pores are filled with water and there is free-water present on the surface of the aggregate. At this state, the materials moisture content is greater than its moisture absorption and the free water is added to the mix, which would increase the W/C ratio in the interfacial zone as well as workability.

If the moisture absorption of an aggregate is 1% or less, then the moisture absorption of that aggregate is deemed to not have a significant effect on the mix (Addis & Owens, 2001).

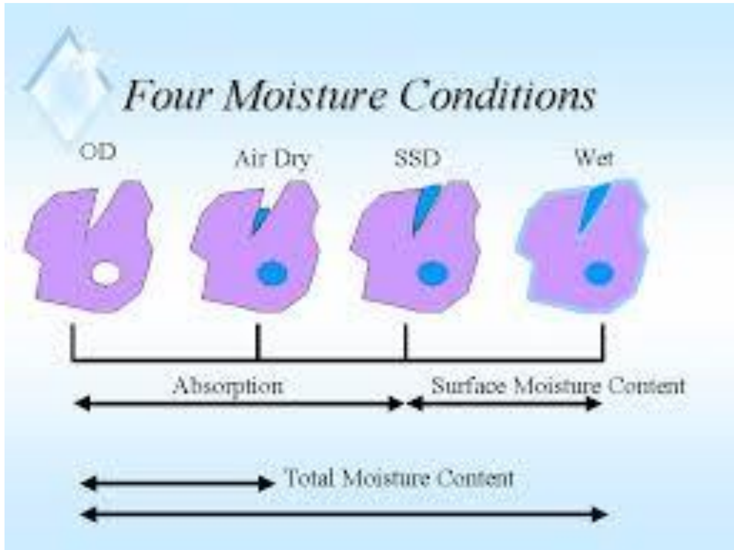


Figure 2-52 : Four aggregate moisture states (Mobasher, 1999)

The wastes material concrete specimens were tested using wastes at natural state, as this was the state that would not require any special preparation such as pre-saturation on site. However, in order to gauge the effect of moisture states, waste materials were also oven dried and added to concrete specimens, which were tested in compression and workability to compare with the results of specimens using aggregates at natural moisture state. The oven-dried state was chosen as this was the state where absorption would be at its maximum, hence there is the highest potential decrease in W/C ratio in the interfacial zone due to the absorption of water in the mix and therefore the highest potential strength increase, as per research by Sallehan & Mahyuddin (2014). However, drier aggregates have also been found to decrease workability as per Gardiner & MacDonald (2013). Comments could then be made on whether the theorised decrease in workability would restrict the use of a potentially stronger mix (See section 3.2.4 for moisture absorption and content method, 3.11 moisture investigation methodology and 4.11 for moisture analysis)

2.4. Economic analysis

Economic viability is a key component in determining if a material is viable for implementation, as the construction industry is profit-centred (Cato, 2009). Cost estimation is used as a method of forecasting construction costs to evaluate economic viability. Essentially, cost estimation involves the tallying of components and the application of cost rates to develop project costs (Sabol, 2008).

There are four general methods for cost estimation with varying degrees of accuracy (Dalton, et al., 2011):

- **Project Comparison Estimating**
  Used in the early planning phase of a project when little information is known and is generally based on historical data of the total costs related to similar passed project.

- **Square/Meter Estimating**
  Used in preliminary and intermediate budgeting and is based on historical data. This method uses a unit cost table in cost/m^2 such as Table 2A from UFC 3-701-01, which based on the type of structure being built and applies it to the measured floor area.

- **Parametric Cost Estimating**
  Intermediate level estimating when design drawings are between 10-35% complete. Parametric estimation involves the costing of several task into single units. For example, the task required for a foundation: Excavations, formwork, reinforcing, concrete and backfill would be priced using a single unit rate called foundations.

- **Quantity Take off (QTO) Estimating**
  The estimation costs are found by multiplying prices per item (e.g. No. of bricks) by the quantity of the item required (e.g. bricks required for a masonry wall). Factors such as delivery and material costs are considered.

The Project Comparison Estimation, Square/Meter Estimation and Parametric Cost Estimation were based on having precedent, which was not available for the waste mixes and although they could be applied to the control mix scenario, for research purposes, the most accurate method was needed and this was the QTO method. Also, the only method that could be used to estimate the cost of a design mix in addition to building a scenario cost model was the QTO method as it was based on pricing each individual component of an item.

In order to build a cost model, as per the construction methodology used by Ingram (2014) and stated in the book titled *"Unified Facilities Criteria (UFC): Handbook: construction cost estimating"* by Dalton, *et al*. (2011), the tasks that need to be priced for the constriction item must be identified. In relation to this study, the items to be priced were the material cost for the concrete mixes and the two scenario analyses described below:

> *The scenario analysis was based on the PRASA rail re-signalling project. It was selected due to its relevance to contemporary South Africa because of the current upgrade of passenger rail infrastructure taking place, which consisted of both cast in-situ and precast applications for the waste material mixes to be costed on. The cast in-situ application selected, was a (8,4 x 3,48) signalling equipment room and the precast application was platform copings used to provide a hard-wearing surface for passengers to board and depart the platform. See Appendix K and L for drawings and diagrams.*

As stated by Dalton, *et al*. (2011), once the tasks related to the items are identified in terms of the main cost categories, shown in Figure 2-53, as per Ingram (2014), cost rates need to be obtained to carry out the cost estimation based on the QTO method.

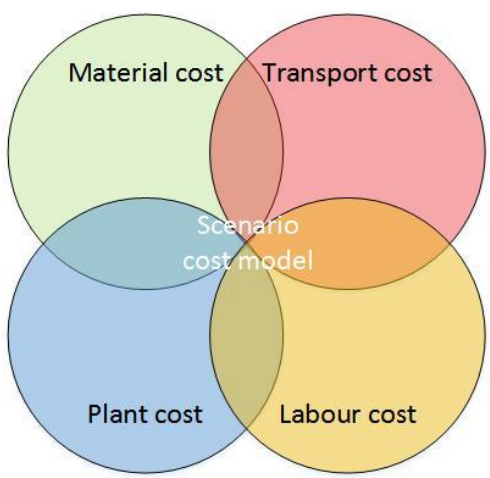


Figure 2-53: Diagram of scenario components

Costs rates are obtained from various sources such as cost books, historical data or parametric databases. Costs can either be classified as direct or indirect costs. Direct costs are those directly related to the construction work, such as materials and labour and indirect costs are those that cannot be attributed to a single construction work task, such as overheads, mark-up and bond (Dalton, et al., 2011). Only direct costs were considered in this study as the cost estimation was aimed at showing the effect on construction costs by using waste concrete.

Although general cost rates were supplied by the contractor in the context of this study certain costs had to be researched from suppliers. A separate costing is generally carried out for each cost category (material, plant, labour, transport) in the QTO method (Dalton, et al., 2011) (Ingram, 2014) and based on these categories the researched costs are explained as follows.

Material

Pricing for materials are generally from pricing services, cost book catalogues, quotations and historical data records (Dalton, et al., 2011). Despite stating the sources for obtaining material costs in general, literature by Dalton, *et al*. (2011) did not adequately explain the costing procedure in terms of a concrete mix. Therefore, alternative literature was consulted for this aspect of the cost analysis. As per Popescu, *et al*. (2005), in terms of concrete rates, costs estimators generally obtain them from batch plants, however, on occasions the job may require mixing on-site. When costing a mix on-site the quantities of cement, sand, stone and any added material must calculated per cubic metre of concrete. The unit for each material (bags, kg, m^3) is at the discretion of the estimator and is generally based on the unit of the cost rate used (Popescu, et al., 2005).

To cost the concrete mixes, rates quoted from suppliers are shown in Table 2-18. Since material and transport costs are considered separately (Dalton, et al., 2011), undelivered costs are used for concrete mix costing.

Table 2-18: Rates used for material costing

Material	Undelivered Rate	Delivered Rate#	Source/supplier
Cement	R 67.50 per 50 kg bag	*R200 delivery charge	Aberdare Hardware
13,2 mm Stone	R 300.30/ m^3	R 343.36 / m^3	Lafarge aggregates
Umgeni Sand	R 198/ m^3	R 223 / m^3	Aberdare Hardware
HDPE pellets	R 9,20 / kg	N/A	Re-SA
Bottom ash	Free	N/A	Sezela Mill
SCBF	*see explanation below	N/A	Sezela Mill
Water (industrial purpose)	R16,47 / kl	N/A	(eThekwini Municipality, 2014)

 # delivered to Pinetown train station

*The rates for SCBF were not readily available according to the Sugar mill research institute (SMRI) as they are used for confidential power generation feasibility studies. A method was suggested by SMRI to calculate the value of SCBF by relating it to the value of coal which is also used as a fuel source for the boilers. To do this calorific values and the cost to generate 1kW of electricity are generally used.

Calorific values are defined as *"the heat energy produced from the combustion of a unit weight of a material and is measured in kilocalories per kilogram*". The typical calorific values sourced from literature are shown in Table 2-19.

Table 2-19: Calorific values

Material	Calorific value kcal/kg	Reference
Coal	5500	(India Solar, 2013)
SCBF	2200	(India Solar, 2013)

As shown in Table 2-19, coal has a higher caloric value than SCBF. Hence, more heat energy is released per kilogram. This meant that it would take roughly 2.5 times more bagasse by mass to generate the same amount of energy as coal.

Labour

Labour costs are generally obtained from a cost book. Rates are usually given in R/hour (in South Africa). When pricing for labour, the rate and the work hours required are essential (Dalton, et al., 2011). General labour was utilised and the rates researched from a local contractor and the time required to construct the scenarios from the contractor's experience are shown in Table 2-21 and Table 2-20.

Table 2-20: Construction times

Application	Labour time
Equipment room	126 hours / 14 days
Coping	0.6 hours / coping

Table 2-21: Labour rates

Designation	Rate (R/h)
General labour	29
Foreman	45

Plant

Plant refers to the tools, instruments and machinery required to perform the construction work. Plant is usually derived in a similar manner to labour, using a rate obtained from a supplier or regional manual. The mobilization of plant is generally included in the costing procedure (Dalton, et al., 2011) and was considered under site establishment. The typical plant costs researched from a local contractor are shown in Table 2-22.

Table 2-22: Plant costs

Item	Rate
230 mm Skill saw	R 150/day
Plate compactor	R10 / day (fuel)
Hilti Breaker	R10/ day (fuel)
Troxler density	R 3639.99/ month
Concrete mixer	R210/day
Poker vibrator	R95 (drive unit) R95 (needle) R190/day (total)
Site establishment	R 3000 (once-off)

**Note any small excavations were by hand and hence no provisions were needed for a bobcat or excavator.*

Transport

The *"Unified Facilities Criteria (UFC): Handbook: construction cost estimating"* by Dalton, *et al*. (2011), merely stated that transport (freight and handling) cost form part of the cost estimation but did not provide much guidance on deriving transport costs and hence alternative literature had to be consulted for this aspect of the study. Carter & Troyano-Cuturi (2009) developed a method to model transport costs that considers both fixed (hire) and variable costs (fuel).

Fixed costs are generally obtained from hire companies and fuel costs are based on the fuel consumption, distance travelled and fuel price (Carter & Troyano-Cuturi, 2009). The fuel price of R12,25 /litre (correct at the time of study) (Engen, 2014) was researched and the average fuel consumption and distance travelled are shown in Table 2-23 and Table 2-24 respectively.

Table 2-23: Vehicle fuel consumptions

Vehicle	Fuel Consumption	Source	Fixed Hire cost	Source
Isuzu KB 250 (4 x2)	9.9 litres /100 km	(Isuzu, 2014)	R150/hour	Kempston Hire
Qixing 10 ton dump truck	24 litres / 100 km	(Qixing Auto, 2014)	R254/hour	Contractor

Table 2-24: Distance to waste sources from Pinetown

Destination	Distance from Pinetown station (one-way)
Re-SA Prospecton	26.5 km (N2 via M7)
Sezela sugar mill	86.3 km

The QTO method (Dalton, et al., 2011) combined with costing methods for materials (Popescu, et al., 2005) and transport (Carter & Troyano-Cuturi, 2009) provided a sound costing methodology to develop the costing models required for this study.

It must also be considered that projects may not always be solely profit-centred. Considering that the construction industry contributes largely to natural resource depletion and pollution there is a need to integrate environmental requirements into the decision making process (Addis & Talbot, 2001) (Carroll, 1999). This study relates to both reductions in natural resource depletion as well as pollution by utilizing the waste materials (HDPE, SCBF, BA) mentioned.

2.5. Chapter summary

The building and construction industry consumes 40% of the raw materials annually (Enercon, 2013). Incorporating waste products and renewable materials as partial substitutions for conventional aggregates or cement in concrete, can potentially reduce virgin mineral consumption in the construction industry (United Nations Environment Programme, 2010). However, in order for waste materials to be utilized as aggregate and cement substitutes, the resulting concrete has to firstly be of an acceptable strength and durability whilst being economically viable to encourage integration into industry.

The review investigated HDPE pellets, SCBF and bottom ash, as well as the material properties (relative density, workability, compression elastic modulus etc.) to be tested and analyses to be carried out in the study. The intended outcome of the literature review was to provide a thorough theoretical background for this study and identify potential knowledge gaps in past research which could be addressed in this study.

Materials

The HDPE selected for this study was readily available from recycling plants located in major CBD's (Durban, Johannesburg and Cape Town), however the SCBF and BA obtained from the sugar mill were limited by the locations of sugar mills. The implications in terms of transportation associated costs were subsequently evaluated in the costing scenario analysis (see section 4.12 of this study).

The volumetric substitutions of the wastes used in this study were shown in past research to have a significant impact on concrete properties depending on whether a low <5% or high volume >10% substitution was used (Magistri, et al., 2011). High volume substitutions generally yielding worse results in strength and durability than conventional concrete. It also was noticed that past research was mainly focused on a particular type of substitution (low or high) and research into the critical volume at which performance peaked between low and high volume substitutions was limited. In the case of BA and HDPE, low-volume substitution research was also limited and hence investigation into low to high volume substitutions formed a significant component of this study to fill the knowledge gaps.

With regard to SCBF, the strength and durability of SCBF mixes were of concern due to the tendency of SCBF to degrade in alkaline environments such as concrete and thereby introduce voids to the concrete mix, increasing permeability. This was confirmed in findings by Racines & Pama (1978) who showed at high volume substitutions (>10%) compressive and flexural strength decreased. Findings by Omoniyi & Akinyemi (2013) also showed that for substitutions above 3%, permeability increased. This warranted a further study into the strength and durability of SCBF in concrete which was carried out in sections 4.6 .and 4.9 respectively, as these properties have a large bearing on the viability of the concrete mixes.

The use of HDPE was limited to non-load bearing applications based on research by Rahman, et al. (2012), due to reductions in strength. However, these findings were based on high volume substitutions and hence low-volume substitutions were also investigated in this study to evaluate if findings by Rahman, *et al*. (2012) applied to both low and high volume substitutions. HDPE was also shown to improve thermal performance by Elzafraney, *et al*. (2005). This was assessed in section 4.7 of this study.

BA was shown to be sensitive to changes in particle size. BA samples were therefore sieved before use in this study. BA was also shown in past research to reduce 7 to 28 day strength of concrete relative to a conventional mix. This was due to the pozzolanic reaction being a slow reaction and the hydrates in the mix being reduced because of cement substitution. The decreased rate of strength development was investigated in his study by comparing the 7 to 28 day compressive strength of the BA mixes with a conventional mix (see section 4.6.1). The use of BA was shown by Cheriaf, *et al*. (1999) to reduce the deterioration of natural fibres by lowering alkalinity in the mix. The effect of BA when mixed with SCBF was therefore investigated in this study in terms of compressive strength performance and rate of strength gain compared to a conventional mix (see section 4.6.1).

Tested properties

Strength, workability and durability are key aspects of concrete. This study investigated these main aspects as well as tested further properties such as specific heat and SEM analysis to further enrich the study. The following properties and analyses were discussed and carried out in this study.

- Material properties (waste absorption, R.D, F.M, shape observations)
- Workability
- Saturated hardened density
- Strength (compression, flexure, splitting)
- Durability (OPI, CC, WS)
- Elastic modulus
- Specific heat
- SEM analysis
- Moisture effects
- An economic scenario analysis

Testing procedures were mostly standard. Specific heat and the waste material absorption tests used however were non-standard due to the lack of specialist equipment and recognised testing procedure respectively.

The different material properties of each waste constituent such as: absorption, shape and texture, were shown in past research to have varying influences on the end properties of the concrete mix. For example, the absorption and long fibre lengths of SCBF may cause reductions in workability, in contrast to HDPE, which can potentially increase workability due to its smooth surface and cylindrical particle shape. It was for this reason that factors such absorption were investigated in this study in order to comment on such variations to concrete properties (see sections 4.2.4 and 4.11).

In terms of concrete properties to be tested, it was noticed from literature on the use of waste materials in concrete, that strength, elastic modulus, durability and workability were common concrete assessment criteria.

Compressive strength is a critical factor in concrete specification. If the compressive strength does meet the required target strength, the mix is not fit for purpose. The critical volumetric substitution was therefore based on the best performing volumetric substitution from 2.5%-40% in terms of compressive strength. This critical volume was used for further material property research into elastic modulus, oxygen permeability index, water sorptivity, chloride conductivity, moisture effects and cost analysis.

Direct measurements of the effect on cracking behaviour are relevant to the fibres, however this testing was beyond the scope of this study, due to the diverse nature of crack behaviour in concrete that would serve as a separate research topic. Testing on elastic moduli and tensile strength properties from this study could however give an indication of the mixes resistance to deformation under load and the load at which possible cracks could occur.

The economic analysis was based on costing methodologies by Sabol (2008), Dalton, *et al*., (2011), Popescu, *et al*. (2005), Carter & Troyano-Cuturi (2009) and Ingram (2014). It was carried out to evaluate the monetary significance of using the waste mixes, in terms of the difference in material cost, from conventional concrete. It also gave an indication of the percentage of the total project cost attributed to concrete cost. Materials such as BA and SCBF may be cheaper to purchase (see section 2.4) but the cost to transport the materials may potentially render them unfeasible (See section 4.12 for the economic analysis).

The next chapter will explain the methodology used for this study to achieve research objectives and aims.

Chapter 3 **Methodology**

3.1. Introduction

The term “green concrete” is used in in this thesis to define conventional concrete modified with proposed “waste” materials. The study involved an experimental analysis, SEM imaging analysis and cost analysis to compare waste mixes containing volumetric substitutions of Bottom-ash, HDPE pellets and SCBF at natural moisture state, with conventional concrete, to assess their viability.

The concrete properties, material properties and volume fractions identified after a critical review of literature and past research were the starting point for the experimental approach taken for the study. The sieve analysis, relative density, bulk densities, measurements of fibre length, moisture absorption and moisture content were tested to carry out the mix designs and explain results from the concrete testing.

The mix design was carried out in accordance with the Cement & Concrete institute (C&CI) method. For all mixes the W/C ratio was kept constant and there were no corrections to water demand for the varying wastes. This was done so that the effects of adding the waste can be identified by any variations in results (slump, changes to W/C ratio hence compression etc.) for the concrete property testing. A volumetric substitution of waste materials was utilized to substitute coal bottom ash for cement, and coarse stone aggregates with SCBF and HDPE pellets. The volumetric substitutions of 2.5%, 5%, 10% and 40% were used to get a range of concrete property results and identify the changes in compression, flexure, workability, density, specific heat and splitting strengths due to varying proportions of waste at natural moisture state. Standard testing procedures normally associated with conventional concrete were utilized so that a comparison could be carried out with the conventional concrete mix. The appropriate SABS/SANS standards were used where possible to maintain local relevance and render the test results credible.

The SEM analysis was also carried out, but because of limitations in terms of sample size able to be analysed by the machine, the mixes would be similar, containing mostly cement paste as stone aggregates were too large. Therefore, only the aggregate interaction between HDPE and SCBF with the cement mix was investigated with the SEM as well as the imaging of BA on a microscopic level.

After testing the mixes of varying proportions, the critical volume fractions were obtained from the peak compressive values for each waste. Even though compressive cube strength is not the only factor that determines concrete performance, compressive strength is ultimately what concrete is specified with. If the mix did not achieve a target compressive strength it could not be used for structural purposes. The critical volumes were subsequently the volume fractions used for any further testing to narrow done the research and identify the most viable mix for each waste.

Apart from indicating to the reader what the limiting volume fractions were for each waste, the critical volumes were also the volume fractions selected for each waste when the mixes with combinations of wastes were designed (SCBF + BA, HDPE +BA, HDPE +SCBF + BA). The mixes using waste combinations were also tested for compression, flexure, workability, density, specific heat, SEM analysis and splitting strengths.Further tests for elastic modulus, oxygen permeability, water sorptivity and chloride conductivity were carried out for individual wastes and mixed waste combinations at critical volume fraction to expand on knowledge on the behaviour of the waste concrete mixes. The comparisons of the tested concrete property results between the critical volume results of each waste and the mixed waste combinations indicated whether the materials were best used independently or in combinations with each other.

The effect of moisture was investigated by firstly relating the tested moisture absorptions and moisture content values to compressive and slump tests carried out. Then the effect of the waste aggregate moisture state was assessed by testing for compressive cube strength, using waste aggregates that were oven-dried and comparing the results to the mixes tested with aggregates at natural state, keeping all other variables such as W/C ratio and mix design fixed.

The economic analysis was carried out by creating a pricing model for waste mixes and conventional concrete using current market rates and applying them to a cast-in situ scenario (signal room) and a pre-cast scenario (coping blocks). Costs were considered for material, labour, plant and transportation to reflect a more realistic idea of the impact on total cost the waste material had. Based on the outcomes of the tested properties, economic analysis and using the points based quantitative analysis as a tool, the potential viability of the waste mixes was determined by drawing comparison with conventional concrete. A flow chart illustrating to the reader the processes taken is shown in Figure 3-1.

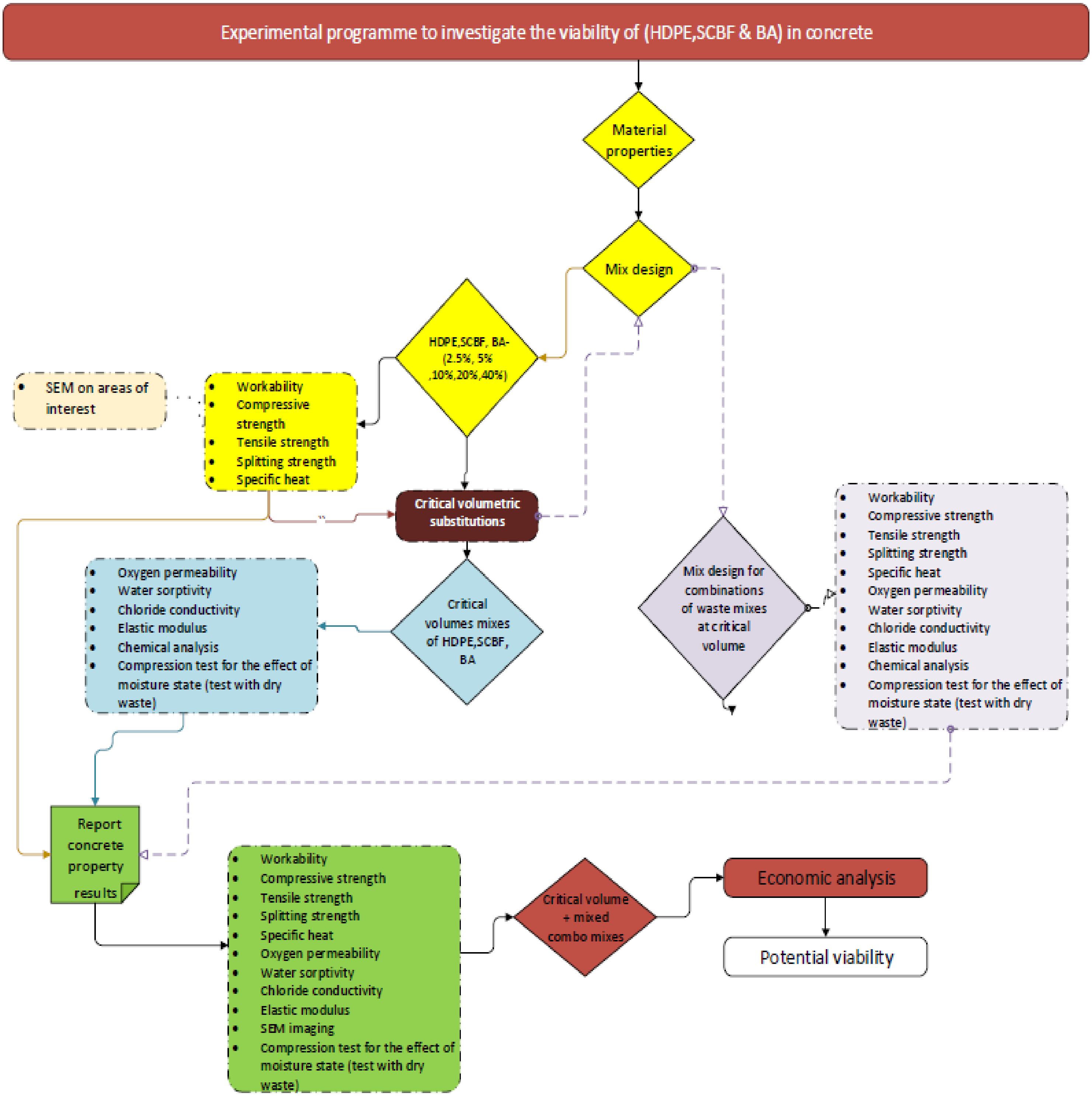


Figure 3-1: Experimental programme

3.2. Constituent material properties

In order to carry out the mix design and comment on results from the concrete property testing, selected material properties had to be tested / obtained from literature.

Mix design

- In order to carry out the concrete mix design the relative densities for both fine and course aggregates, fineness modulus for the fine aggregate (sand), and the compacted bulk density for the 13.2mm stone aggregate had to be calculated or obtained from a reliable source.
- The relative densities were used to convert calculated mass values to volume and vice-versa, in order to obtain the quantities required for the mix design according to C&CI design calculations. The relative densities were also used for the purpose of the volumetric substitutions of aggregates. Relative densities for SCBF and cement were obtained from reliable literature by the Cement and Concrete Institute (C&CI, 2013) and Natal Portland Cement (NPC, 2013) respectively.
- The relative density of SCBF was obtained from literature because the TMH 1 method for calculating relative density was impractical, as fines would float and not give an accurate mass reading during the test.

Moisture contents

- The moisture contents of the waste materials were tested by a local company (Contest) using non-standard tests. This data was needed to provide insight into the possible impact on water/cement ratio and subsequently the strength and slump readings of the concrete. The water demand was not adjusted in the mix in order to show the degree of influence of moisture content and absorption on the respective material properties.

General properties

- Length of bagasse fibres and sizes for HDPE pellets were measured to further classify the materials and comment on the influence of particle size and length on the mix.

The test procedures for each of the above will now be explained.

3.2.1. Fineness modulus of sand (fine aggregate)

Fineness modulus is a measure of the coarseness or fineness of an aggregate and a sieve analysis is used to calculate it (Addis, 2008). The process was according to standard concrete design method B13 from TMH1 (CSIR, 1986).

Apparatus

- 500g of dry Sample
- 4.75mm, 2.36mm, 1.18mm, 0.6mm, 0.3mm and 0.15mm sized aperture sieves.
- Cleaning brush
- Vibrating machine

Method

A 500g sample is dried for 24hrs in an oven at 108°C to remove all moisture. The sample is then poured into a tower of sieves arranged as follows from top to bottom: 4.75mm, 2.36 mm, 1.18 mm, 0.6 mm, 0.3 mm, 0.15 mm and placed in a vibrator for 2-minutes. Each sieve is brushed clean and the mass of material retained on each is weighed out. The formula shown below is then used to obtain the fineness modulus (FM) of the sand.

FM =
$(\sum of\ culmintive\ percentages\ of\ retained\ material\ on\ each\ sieve\ 0.15mm\ and\ coarser\ )$
÷ 10

Figure 3-2: Sieve analysis apparatus

3.2.2. Compacted bulk density

The bulk density is the mass of a material that will occupy 1m^3 of space. If the material is just placed in the container and measured, then this is the loose bulk density. The compacted bulk density is calculated by using the mass of a compacted sample material in a fixed volume container/mould. This is calculated simply by taking the mass (kg)/ volume (m^3). The process was according to standard concrete design method B8 from TMH1 (CSIR, 1986).

Apparatus

- 1x Mould/container
- Mass balance (with capacity greater than predicted sample mass)
- Sample material
- Rigid rule

Figure 3-3 Mass scale, Mould, 13.2 mm stone

Method

Samples obtained from the stockpile were filled in a mould until the mould was overflowing. The mould filled with the sample was then dropped on a hard surface 20 times from a height of approximately 15-20mm above the surface. The mould was then levelled off with a ruler or rigid flat tool to remove excess sample. This mass was then weighed and divided by the fixed volume of the mould to obtain the compacted bulk density.

Density = $\frac{mass}{volume}$ [kg/m^3]

3.2.3. Relative density

The relative density of a material is the *“mass of a particular volume of a substance when compared to the mass of an equal volume of water at 4°C”* (Cambridge dictionaries, 2013). It is needed when adjusting the aggregate content for the mix design.

3.2.3.1. HDPE & Stone aggregate

The process was according to standard concrete design method B14 from TMH1 (CSIR, 1986).

Apparatus

- Scale with a 6000 g capacity and a 0.1 g accuracy
- 210 mm diameter sieve with a 63 μm screen size.
- String
- Mosquito net
- Suitable table or work area with a hole(approximately 50mm in diameter)
- 300 mm diameter bucket

Figure 3-4: Relative density apparatus

Method

The sample (HDPE/13,2mm stone) is dried for 24hrs in an oven at 108°C and subsequently measured in air. The sample is then placed in a holder and submerged in water of a known density. The submerged mass is recorded and applied to the formula below in conjunction with the mass, in air, to calculate the specific gravity. From this, the density can be calculated assuming a known density of submersion liquid. In this case water is approximately 1000kg/m^3.

$$\text{Relative density} = \frac{Oven\ dry\ mass}{Oven\ dry\ mass-(submerged\ mass-Mass\ of\ holder)}$$

3.2.3.2. **Coal ash and sand**

The process was according to standard concrete design method B15 from TMH1 (CSIR, 1986). The coal bottom ash was sieved through a 1400 μm aperture sieve of 210mm diameter to remove foreign particles such as stones.

Apparatus

- Volumetric flask
- Filler bottle
- Funnel
- Distilled water

Figure 3-5: Volumetric flasks for relative density tests

Method

The mass of the dry empty flask is taken. It is then filled with water and measured again. The mass of the oven-dried sample is taken and finally it is added to the flask filled with distilled water and the mass of the flask, sample and water is recorded. The formula below is used to calculate the relative density (RD) from method B15 in TMH1.

$$RD = \frac{(mass\ of\ oven\ dry\ sample\ and\ flask - mass\ of\ dry\ empty\ flask)}{(mass\ of\ flask\ filled\ with\ water - mass\ of\ empty\ flask)dry - (mass\ of\ flask\ with\ water\ and\ sample - (mass\ of\ oven\ dry\ sample\ )}$$

3.2.3.3. Cement & SCBF

The relative densities of NPC black 42.5 N cement and SCBF were obtained from the supplier data sheet and C & CI respectively.

3.2.4. Moisture properties

Moisture absorption and moisture contents were obtained from an external concrete laboratory (Contest) who used non-standard test procedures to saturate the waste materials. The testing procedures used to calculate the moisture content and moisture absorption were based on TMH 1, method A7 and method B15 respectively.

The moisture absorption and moisture content of the materials were obtained from the following formulae extracted from the above mentioned testing methods:

Moisture absorption = $\frac{SSD-OD}{OD}$ x 100

Moisture content = $\frac{natural\ state-OD}{OD}$ x 100

Where,

OD = Oven Dry mass [g]

SSD = Saturated Surface Dry-mass [g]

Natural state = Mass at Natural State from storage [g]

3.2.5. Material sizes

The SCBF fibres and HDPE pellets were measured with a ruler and the BA particle size was obtained from an SEM analysis.

3.3. Mix design and test specimen preparation

The research required a mix design method that was repeatable and controlled. The C & CI method of concrete mix design was therefore utilized. This study used the C & CI method as a base and adapted it for volumetric aggregate substitution. The steps taken in summary were as follows:

- A water cement ratio was selected to achieve a target concrete strength using a specified cement type;
- Water volume was based on stone aggregates used;
- Cement content was obtained using the W/C ratio and selected water content;
- Mass of stone was calculated by using the C&CI stone content formula;
- Amount of sand was calculated in terms of absolute volume required to fill the mix and produce a cubic metre of concrete;
- Once the control mix was designed and accepted, the waste mixes were designed by calculating the waste volumetric substitutions of cement and stone aggregates with reference to the control mix.

Figure 3-6 shows a flow diagram which summarizes the mix design method based on the C & CI method used for this study.

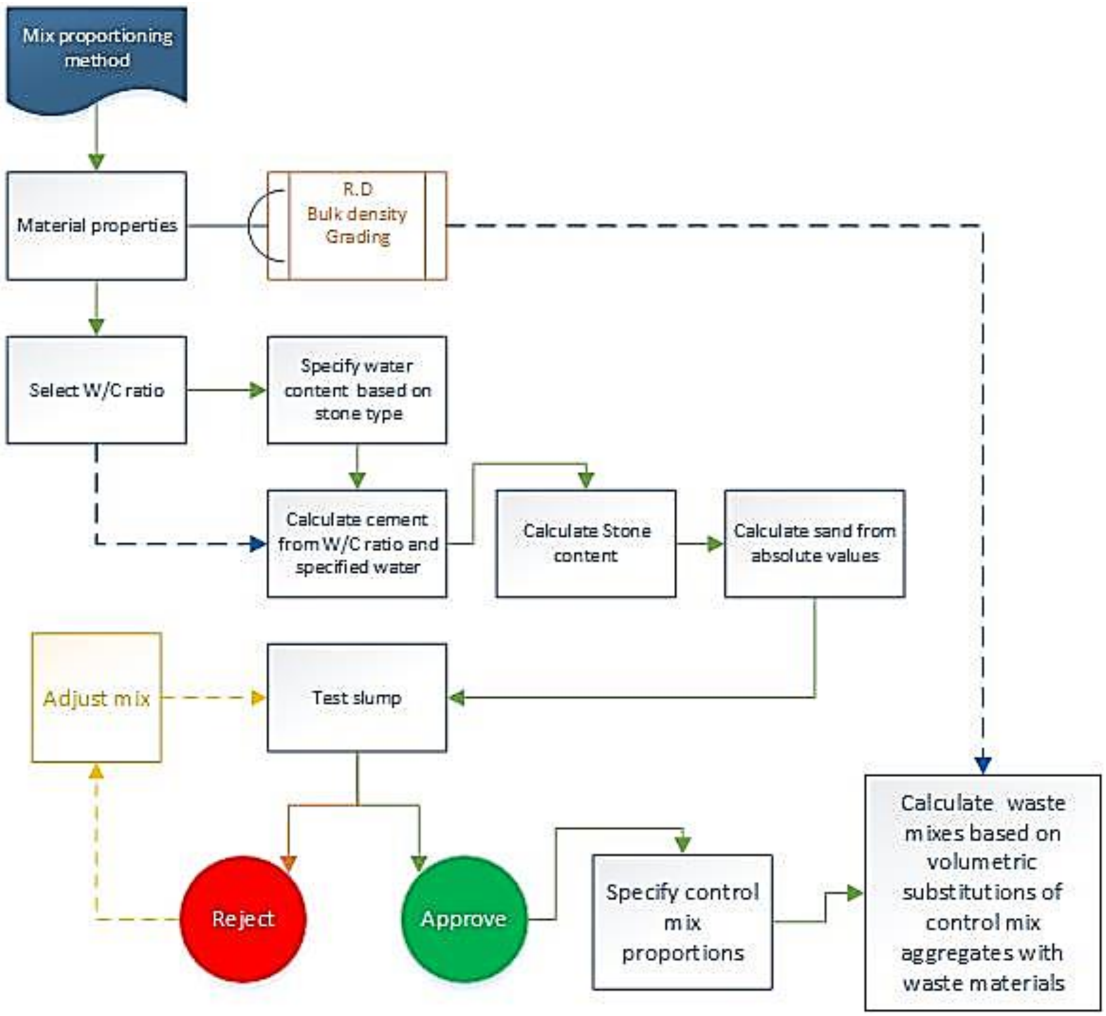


Figure 3-6 : Overview of Mix design method used for this study

3.3.1. Water: Cement ratio

The water cement (W/C) ratio was read off the graph *(*Figure 3-7*)* used by the University of KwaZulu-Natal for laboratory mix designs based on the Natal Portland Cement (NPC) W/C ratio curves. For this study, a W/C ratio of 0.57 was used based on the typical range stated in section 2.3.2.2. NPC Original black 42.5N cement was utilized as it was readily available at the University of KwaZulu-Natal. The corresponding NPC Plus plot was used to read off the W/C ratio for a control mix target compressive cube strength of f_{cu} = 35 MPa at 28 days.

The study did not aim to assess the effects of using varying cement types, which can constitute a separate research topic on its own. The study instead aimed to evaluate the effect of adding various waste materials to concrete, keeping other material constituents (sand, cement, and stone) constant, and subsequently assessed the relative variation in concrete properties compared to a "control" mix with no waste added. The W/C ratio was not varied either because the focus was on showing the effect of waste aggregates in varying proportions on selected concrete properties and not the effect of varying W/C ratios using concrete with waste aggregates in the mix.

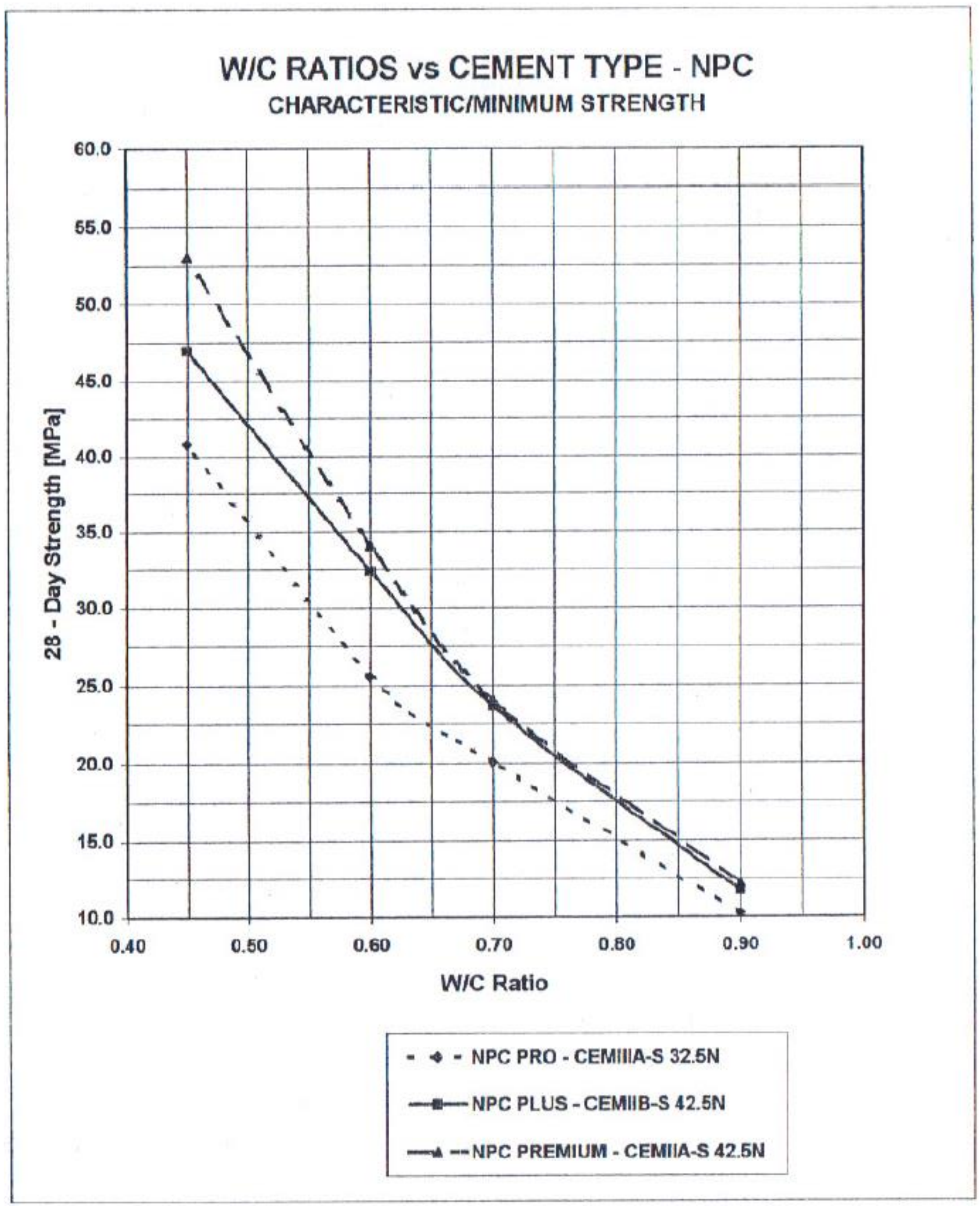


Figure 3-7: Graph of W/C ratio (University of KwaZulu-Natal, 2009)

3.3.2. Water content

Based on the C&CI mix design method, a water content was selected from the recommended guideline value for a concrete mix using a 13,2 mm stone size, stated in "Fundamentals of Concrete" (Addis, 2008).

The concept of keeping water content constant and quantifying the effect on concrete properties through variations in test results compared to a control mix, was used for this study. This method has been adopted by published researchers in alternative aggregates such as Rao (2010). The effect that the waste aggregates had on the water demand was also assessed in terms of slump and compressive strength in the moisture analysis (section 4.11).

3.3.3. Cement content

Using the selected water content (l/m^3) and the W/C ratio, the quantity of NPC original black – CEMII B-S 42.5N cement required was calculated as show below:

Water content (l/m^3) ÷ W/C ratio = volume of cement (litres) per cubic metre of concrete

3.3.4. Stone content

A 13.2 mm stone at natural moisture state, was used for this study to reduce the variance in particle size between stone and the waste aggregates to improve particle packing. The stone content (St) for a cubic metre of concrete was then found using the formula below assuming a K factor of 0.9 as per the UKZN mix design guideline (University of KwaZulu-Natal, 2009) for moderate vibration and 13.2mm stone.

St = CBDst (K-0.1FM) [kg/m^3]

Where;

CBDst = compacted bulk density of stone [kg/m^3]

K = Factor depending on workability and nominal stone size

FM = Fineness modulus of sand

The CBD was calculated as per section 3.2.2 and the FM of the sand was calculated as per section 3.2.1. The k-factor was selected from Table 2-11 on page 101 and was based on the degree of vibration and the stone size used. A vibrating table was used for the study, so moderate vibration and 13.2 mm stone size were the input variables for the table to obtain the K-factor used in the stone content formula.

3.3.5. Sand content

River sand at natural moisture state was used for this study. The sand was passed through a large 300 mm diameter, 132 mm aperture sieve to remove any large stones that may have contaminated the stockpile. Using the absolute volume method stated in "Fulton's Concrete Technology", the sum of the volumes had to equate to $1m^3$ or 1000 litres. All calculated values were converted to volume values using the respective relative densities. The volume of sand required was then calculated from the sum total of the other constituent material volumes subtracted from $1m^3$ or 1000 litres. All volumes were then converted to mass by multiplying by the respective relative densities.

3.3.6. Adjustment of trial mix

Once a trial mix was done for the control mix, the slump and cohesiveness were assessed. If the mix was rejected, for example, if there was a lack in cohesion or it did not comply within error margins stated in Table 2-12 on page 101, then the design was modified and re-tested by reducing stone or increasing the sand quantity. If not, then the proportions were accepted as the control mix for the study.

3.3.7. Volumetric waste substitution

Once the control was accepted as stated in section 3.3.6, the waste mixes were calculated based on the substitution of stone and cement from the control mix proportions.

All waste materials were used at natural moisture state except for the moisture analysis, which used waste materials at oven dry state to evaluate the effect of moisture on workability and compressive strength.

The bottom ash was passed through a 210 mm diameter, 1400 μm aperture sieve to remove foreign particulate matter and obtain the ash powder. The quantities for each mix were then weighed for volumetric substitutions of (2.5%, 5%, 10%, 20% & 40%).These substitutions were selected to build upon knowledge gained from past research explained in the literature review by covering both low-volume (2.5%) to higher volume (40%) volumetric substitutions.

The quantities of each mix were easily obtained by calculating the amount of concrete required to fill the desired moulds, dividing 1 m^3 by this value and then dividing the proportions of each constituent by the quotient.

Three specimens were required per a test for the compressive, flexural and splitting tests and amount of concrete volume required per mould was:

3 cubes = 0.15^3 x 3= 0.010 m^3

3 beams = 0.1 x 0.1 x 0.3 x 3= 0.009 m^3

3 cylinders = π x 0.15^2 x 0.3 x 3= 0.06 m^3

When batching 10% was added for spillage.

The weighed materials for each mix were then added to the concrete mixer and mixed for 5 minutes.

Figure 3-8: Drum type concrete mixer

Figure 3-9: Beam moulds on vibrating table (left) and cube mould (right)

The moulds were oiled to allow for easy removal of concrete. The concrete was then poured into beam moulds, cube moulds and cylinder moulds for each mix design and compacted with a table vibrator.

Figure 3-10: Specimens curing in controlled curing room

The moulds were stripped after 24 hours, the date of casting and sample number were marked with chalk and the samples were placed in curing tanks. The concrete cube and beam specimens were cured for 7 and 28 days, and the cylinders cured for 28 days at a controlled room temperature of 24°C before testing

3.4. Workability

The workability for all individual waste samples 2.5%, 5%, 10%, 20%, 40% and mixed combinations was assessed by using the slump test on freshly mixed concrete according to the SANS method 862-1:1994. This gave insight into the effects on conventional concrete due to the physical properties (absorption, moisture content, shape, texture, size) of the waste aggregates substituted into the mix.

Apparatus

- A 300mm high conical hollow frustum mould with foot-holds and lifting handles at two-thirds vertical height. The cone must have a top diameter of 100mm and a bottom diameter of 200mm (2mm tolerance).
- Tamping rod with a 16mm diameter and 600mm length with one hemispherical end
- Level base plate.

Method

The apparatus was cleaned and the mould was placed on the level base and held in place by standing on the foot-holds. The mould was filled in thirds and each layer was tamped with 25 even stokes per layer ensuring that the taping rod penetrated to layers below the layer being tamped. After the third layer was poured and tamped the excess concrete was struck off using the tamping rod with a sawing action. The mould was carefully removed, turned over and slump measured to the nearest 5mm by measuring the length in millimetres from the top of the up-side down slump cone to the top of the slumped concrete *(See* Figure 3-11*).*

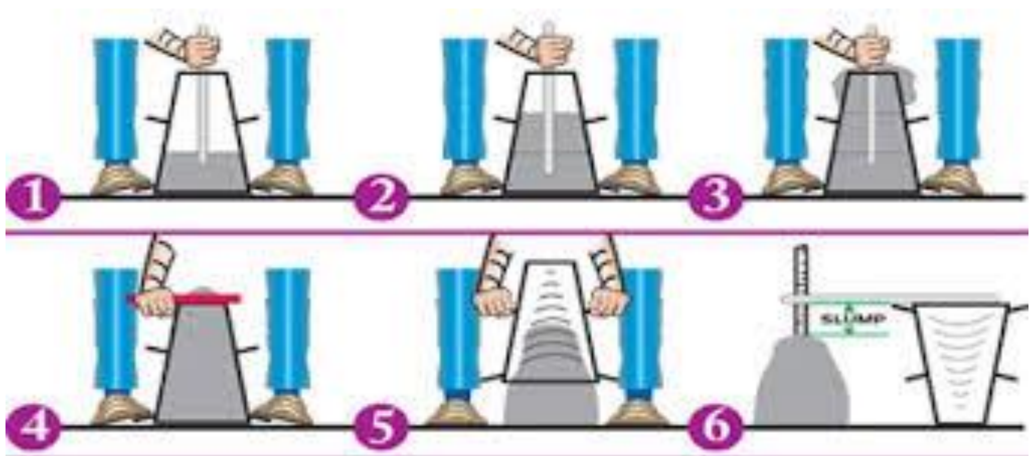


Figure 3-11: Slump test (Blogspot.com, 2014)

3.5. Saturated hardened Density

Density of saturated samples after 7 and 28 days of curing was calculated for all individual waste samples 2.5%, 5%, 10%, 20%, 40% and mixed combinations using the method from the compression testing standard SABS 863:1994 (SABS, 1994). Density was assessed in terms of the concrete specimens change in mass per unit volume, as well as the relation between density to the durability index tests and strength tests carried out. A lower density and higher permeability reading could imply that there were more voids present in the mix, increased porosity and therefore a lower strength. The saturated hardened density readings between 7 and 28 days were also compared to identify the effect of hydration on the saturated hardened density of the concrete.

Apparatus

- Scale accurate to at least two decimal places

Method

Samples are removed from the curing tank and weighed. Dimension's measure and volume calculated. The density is then simply the quotient of mass over volume.

Density = $\frac{Mass\ (kg)}{Volume(m^3)}$ [kg/m^3]

For this study average the density of three cubes was used per mix.

Figure 3-12: Mass scale

3.6. Strength property experimental testing

Each waste (HDPE, SCBF, BA) in volumetric proportions of 2.5%, 5%, 10%, 20% & 40% as well as the mixed waste combinations were tested using the standard compression (SABS 863:1994), flexure (SABS 864:1994) and splitting tests (SABS1253:1994). This was done in order to quantify the effects of substituting the waste materials into a conventional concrete mix in terms of strength (compression, flexure & splitting), one of the main performance criterion for concrete. The strength tests carried out were as follows.

3.6.1. Compressive strength

In South Africa cube strength (f_{cu}) is normally specified (Addis, 2008). Cube strength was tested for the local context of the study. Three 150 mm cubes were crushed per a mix according to SABS 863-1994 at 7 and 28 days after casting.

Apparatus

- Compression testing machine
- 150mm cube mould
- Scientific scale

Method

The saturated samples were removed from the water bath with grit and surface water removed before weighing to calculate density as per section 3.5. The bearing surfaces of the compression machine were cleaned and the specimen placed at the centre of platens, such that load was applied to the surface opposite the as-cast faces, i.e. the sides not exposed when the cube was in the mould. Load was applied until failure and recorded. The compressive strength in MPa was calculated as follows:

$$f_{cu} = \frac{F}{A}$$

f_{cu} = compressive strength MPa [$\frac{N}{mm^2}$]

F = load at failure [N]

A = cross-sectional area of test cube [mm^2]

The average of three cubes was calculated to the nearest 0.05MPa and the test was considered valid if the highest and lowest recorded strength did not exceed 15% of the average value.

Figure 3-13: Compression testing machine

3.6.2. Flexural strength

Three beams (300mm x 100mm x 100mm) were tested according to SABS Method 864:1994 at 7 and 28 day strengths. Flexural testing is one the two methods to assess tensile strength. Flexural strength is usually applicable to concrete used for flooring, beams, etc. (Addis, 2008).

Figure 3-14: Failure of beam in flexural apparatus

Apparatus

- Steel mould to cast specimens (300mm x 100mm x 100mm)
- Compression testing machine
- Scientific scale
- Two-pairs of rollers

Set-up as per Figure 3-15.

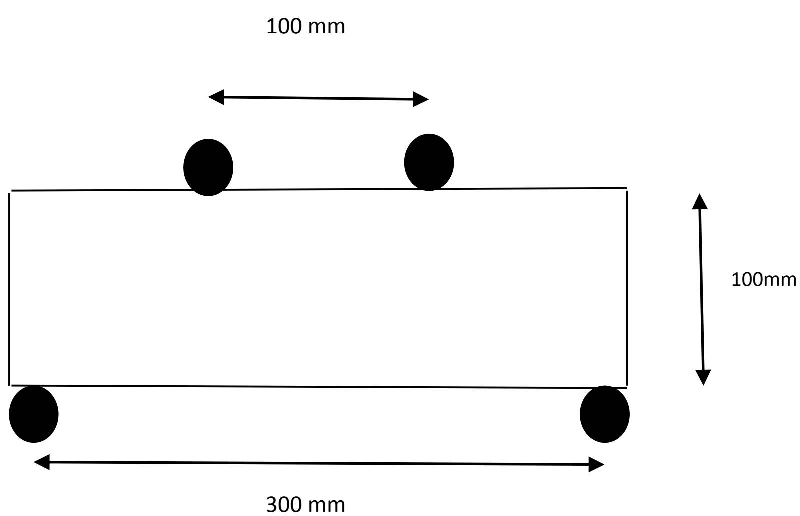


Figure 3-15: Layout of flexural testing apparatus

Method

The saturated samples were removed from the water bath with grit and surface water removed before weighing to calculate density. The rollers of the two-point load testing apparatus was cleaned and specimen placed centrally with load being applied to sides opposite to as-cast sides, ensuring axes of rollers were normal to the specimen's longitudinal axis. Load was applied until failure and recorded. The flexural strength was calculated from the results as shown below.

$$f_f = \frac{PL}{bd^2}$$

f_f = Flexural strength [MPa]

P = breaking load [N]

L = Horizontal distance between support rollers of testing apparatus [mm]

b = width of test specimen [mm]

d=depth of test specimen [mm]

The average of three specimens was calculated to the nearest 0.05MPa and the test was considered valid if the highest and lowest recorded strength did not exceed 15% of the average value.

3.6.3. Splitting strength

Three 150 mm splitting cubes were tested at 28-days according to SABS1253:1994. The splitting strength test gave a measure of the indirect tensile strength of the specimen when a compressive force is applied along a plane joining load lines. Particular attention was paid to the mixes with fibre substitution, because if they performed beneficially as crack arrestors as they are intended, the splitting strength would be higher than a conventional mix.

Apparatus

- Compression testing machine
- 150mm cube mould
- Scientific scale
- Splitting test frame and rollers

Method

The saturated samples were removed from the water bath with grit and surface water removed before weighing to calculate density. The bearing surfaces of the compression machine, splitting test frame and rollers were cleaned and the specimen placed at the centre of rollers such that load was applied to the surface opposite the as-cast faces, i.e. the sides not exposed when the cube was in the mould. The compressive load was applied until failure of the concrete specimen and recorded. The strength in MPa was calculated as shown below:

$f_{s}=\frac{2P}{\pi a^2}$

f_s = Splitting tensile strength [MPa]

P= Compression load at failure [N]

a= cube length [mm]

Figure 3-16: Splitting strength apparatus

The average of three specimens was calculated to the nearest 0.05MPa and the test was considered valid if the highest and lowest recorded strength did not exceed 15% of the average value.

3.7. Specific heat

Specific heat (J/kg °C) was selected to provide an idea of the impact on the concrete mix's thermal properties due to the addition of waste materials. The assessment of thermal performance is usually based on specific heat, thermal conductivity and thermal diffusivity. However testing equipment used for thermal conductivity and diffusivity tests were not available at the time of the study. Considering this, specific heat is still a useful thermal performance indicator as it gives an indication of whether greater amounts of heat energy would be required to cause a change in temperature and hence possibly lower indoor temperatures during hot days which is relevant to the local context (The Concrete Society, 2013). All individual waste samples (2.5%, 5%, 10%, 20%, and 40%) and mixed combinations were tested for specific heat using a non–standard method explained in section 2.3.6.2 as follows.

Apparatus

- Polystyrene cup
- Cotton
- Plastic beaker
- Water
- Thermometer
- Two thermocouples
- Hot-plate

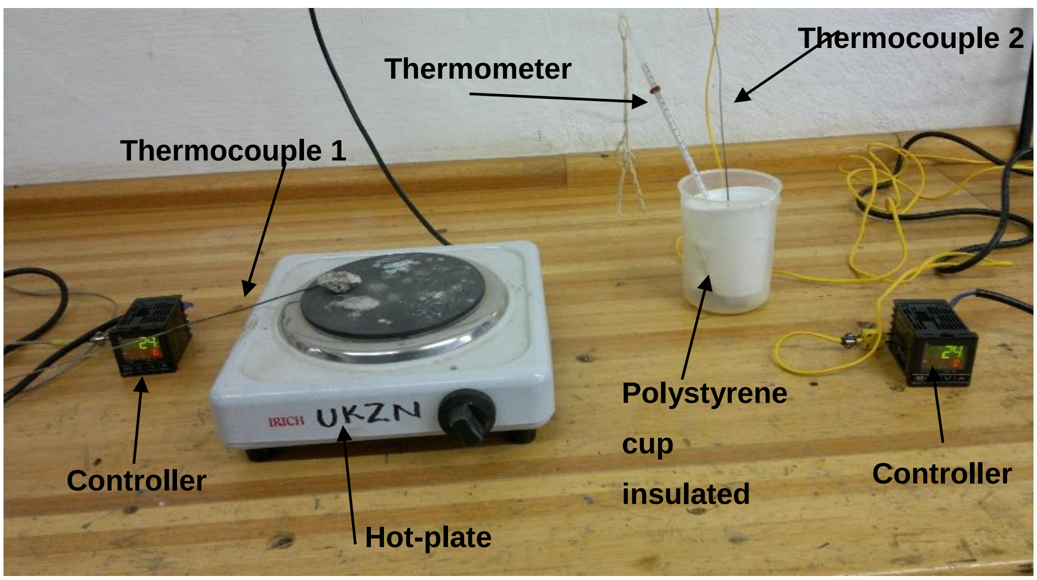


Figure 3-17: Specific heat apparatus setup

Method

Approximately 100g samples of each waste mix were taken from 150mm x 150mm cubes cast especially to evaluate specific heat. To create an insulated vessel to conducted experiment, a polystyrene cup was placed in the beaker and lagged with cotton to minimize heat loss. The beaker was then placed on a mass scale and zeroed. The cup within the beaker was filled with approximately 100g (m_w) of water and the temperature measured (t_i) using a thermometer and thermocouple 2. The ±100g sample material was weighed (m_s) and placed on a hot-plate. The sample was heated and monitored with thermocouple 1 till it reached 100°C. It was then rapidly transferred to the cotton lagged polystyrene cup and the change in heat was monitored with thermocouple 2. The maximum temperature reached was recorded and the formula below was used to calculate the specific heat of the sample, assuming that the polystyrene had a specific heat of zero and heat losses to the environment were limited. The formula below was used to calculate the specific heat of the concrete specimens:

$$m_s c_s (100-t_f) = m_w c_w (t_f-t_i)$$

$$c_s = \frac{m_w c_w (t_f - t_i)}{m_s (100 - t_f)} \text{ [J/kg °C]}$$

m_s = Mass of sample [kg]

m_w = Mass of water [kg]

t_i = Initial temperature of water [°C]

t_f = Final temperature of water and sample [°C]

3.8. Static elastic modulus

Utilizing the available equipment at the time of the study, elastic modulus was tested for all critical volume individual waste samples and mixed waste combinations using the static method (refer to 2.3.7 for literature on elastic modulus). The study followed the non-destructive section of the standard procedure for calculating the static modulus of elasticity as per BS 1881: Part 121 1983. The minimum number of specimens for this test was stated as one as per BS 1881: Part 121 1983. Hence, one specimen was cast per mix and this was confirmed as adequate after consulting Contest Concrete technology Services. However, if more than two out of the four gauges attached gave abnormal results another specimen would have to cast. The static elastic modulus was used to evaluate the effect on stiffness and elasticity of the conventional mix by adding the waste materials.

Apparatus

- 4 x Tokyo Sokki Kenkyujo Co. 90 mm gauge length resistance strain gauges
- 1 x Huggenberger strain reading apparatus
- 150mm (diameter) x 300mm (height) cylinder moulds
- Grout capping
- Compression testing machine

Figure 3-18: Huggenberger apparatus (left), Specimen with strain gauges in compression testing machine (right)

Method

A cylinder (150 x 300) was cast per mix, for all critical volume individual waste samples and mixed combinations and placed in a curing tank with a controlled room temperature of 24°C. The specimens were then tested for elastic modulus after 28 days of curing. The flat surface perpendicular to the direction of vertical load had to be as flat and smooth as possible to ensure equal load distribution. A grout capping was therefore applied by an external concrete testing laboratory (Contest).

After grout capping, four 90 mm gauge length strain gauges imported from Japan were glued longitudinally onto the specimens at equal spacing and at an equal 105 mm distance from both edges of the specimen. The requirement as per the standard is to have at least one pair of gauges attached on opposite sides of the sample, however, under the advice of Prof Everitt (Associate professor in civil engineering of UKZN) four gauges (two pairs) were attached due to the likelihood of some gauges not bonding correctly or giving irregular results. If a result was beyond 15% of the average it was discarded.

To obtain the compressive strength for the static elastic modulus test, the compressive cube strength (f_{cu}) results were converted to cylinder strength (f_{ck}) by multiplying by 0.8 because cylinder strength (f_{ck}) is roughly 80% of cube strength (f_{cu}) (BSI, 1983). This approach has been used in published experimental research such as that by Shafigh, et al. (2012). A design equation provided by Contest Concrete Technology, extracted from BS 8110 was stated as for a specimen with a 2:1 ratio of height to diameter. This equation was used to check the difference in load points compared to using the factor of 0.8.

The specimen was placed centrally in the compression testing machine. The strain reading at rest (approx. zero applied load) was taken. The basic stress of 0.5 N/mm^2 was then applied. The load (N) input for the compression machine corresponding to the basic stress was simply calculated by multiplying the area of the circular face of the cylinder (mm^2) with the basic stress (N/mm^2). The strain reading at basic stress load was taken and the load was increased at a steady rate until the stress reached f_c /3. The stress of f_c /3 was then maintained for 60 seconds and strain readings were taken.

This process was repeated three times per sample. Under the advice of Prof. Everitt (Associate Professor in Civil Engineering of UKZN), to calculate the modulus of elasticity from the data obtained from the Huggenberger apparatus readings, each strain reading was subtracted from the zero reading to give a change in strain. The averages were taken for each set of gauges for each load and the change in strain values were then plotted on the x-axis vs. stress values on the y-axis. The elastic modulus was then calculated by taking the slope of the linear graph formed using the formula from BS1881: part 1221 1983 as follows:

$$E = \frac{\Delta\sigma}{\Delta\varepsilon} = \frac{\sigma_a - \sigma_b}{\varepsilon_a - \varepsilon_b}$$

Where:

E = Static elastic modulus (Pa)

σ_a = Stress at f_{cu}

σ_b = Stress at basic stress (0.5N/mm^2)

ε_a = Strain at f_{cu}

ε_b = Strain at basic stress (0.5N/mm^2)

The modulus results were finally converted to GPa by dividing by 1000000000.

3.9. Durability

For the waste mixes to be potentially viable, they had to meet strength requirements, be economically viable and be durable. Durability tests were carried out for all critical volume individual waste samples and mixed combinations by an external company (Contest), due to the necessity of specialist equipment.

Due to the nature of the tests involving oven-drying and saturation of specimens there was the possibility of laboratory errors occurring as shown by Stanish, *et al*., (2006), due to:

- Incomplete saturation, which would result in a lower conductivity value;
- Improper sealing of the samples, that would result in higher conductivity values, either for the average or for some samples in a set;
- Incomplete oven drying during specimen preparation, which could lead to a decrease in conductivity;
- Not maintaining room temperature, which could decrease chloride conductivity by 2% for every degree Celsius below 25 °C (Thermo Fischer Scientific, 2014).

A brief explanation of the methods used for the Oxygen Permeability Index (OPI), Chloride Conductivity (CC) and Water Sorptivity (WS) tests are shown in sections 3.9.1; 3.9.2 and 3.9.3 respectively.

3.9.1. Oxygen Permeability index (OPI)

The OPI test measures the pressure decay of Oxygen passed through a specimen over time and gives an indication of the permeability of the specimen.

Apparatus

- Oven capable of 50° C (± 2° C)
- Compressible rubber collars
- Permeability cell arrangement as show in Figure 3-19
- Gauges and pressure transducers with at least a 0.5 kPa accuracy
- Vernier calliper (0.02mm accuracy)
- Standard Oxygen supply and regulator capable of 120 kPa pressure regulation
- Desiccator

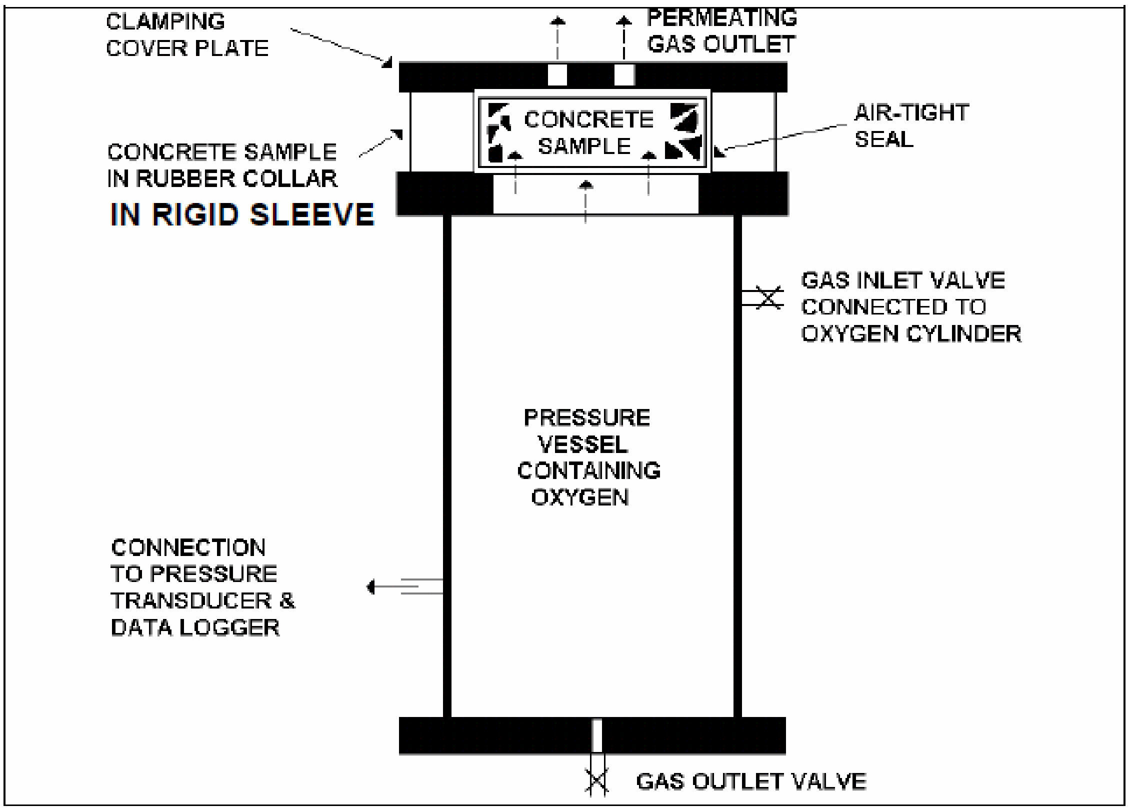


Figure 3-19: Oxygen permeability test apparatus (Comsiru, 2009)

Method

Specimens were cast and cut into 4 disks (70 mm diameter x 30 mm thickness) per specimen. After cutting, specimens were dried for 7 days in an oven at 50° C. Specimens were cooled in a desiccator for 2-4 hours. When testing, each disk was placed in the compressible collar with a rigid sleeve. The sample, collar and rigid sleeve were placed into the test chamber. A solid ring was placed over the rigid sleeve and a cover plate was placed and tightened on top of the solid ring. The test was started 30 min after the sample was removed from the desiccator. The oxygen inlet and outlet were left open and the test chamber was purged of all gases other than oxygen for 5 seconds. The oxygen outlet was closed until pressure built up to 100 kPa then the inlet was closed. The initial pressure P_0 at time t_0 was recorded. The second reading was taken after 5min and subsequent time readings after every 5 kPa drop in pressure were recorded until the pressure reading reached 50kPa. The test was repeated for all four disks and the calculations below were used to calculate the oxygen permeability of the sample.

The time t_0 readings were then plotted against pressure P_0 readings taken and the slope of the line of regression was found (Comsiru, 2009).

The formulas used are shown below:

$$z = \frac{\sum\left[\ln\left(\frac{P_0}{P_t}\right)\right]^2}{\sum\left[\ln\left(\frac{P_0}{P_t}\right)t\right]}$$

z = slope of the regression line

$\mathbf{P_0}$ = pressure at time t_0 [kPa]

$\mathbf{P_t}$ = pressure readings taken at time t after t_0 [kPa]

t = time in seconds

The Darcy coefficient of permeability (k) was found by:

$$k = \frac{\omega V g d z}{R A \theta}$$

$\boldsymbol{\omega}$ = Molar mass of Oxygen = 32 g/mol

V = Volume of Oxygen under pressure in the permeameter [m^3]

g = gravitational acceleration = 9.81 m/s^2

d = Average specimen thickness [mm]

z = slope of the regression line

R = universal gas constant 8.313 Nm/K mol

A = cross-sectional area of the specimen in [m^2]

$\boldsymbol{\theta}$ = absolute temperature [K]

The Oxygen permeability index (OPI) was finally calculated from the negative log of the average k value for the four disks.

OPI = = - $\log_{10} [¼ (k_1 + k_2 + k_3 + k_4)]$

$k_{1, 2, 3, 4}$ = Darcy coefficients of sample disks 1, 2, 3 and 4.

The average of the four concrete disks tested for the OPI for the mixes at critical volume and the mixed waste combinations, were reported by Contest and used for this study.

3.9.2. Chloride conductivity (CC)

The chloride conductivity tests, developed in South Africa, gave an indication of the resistance of the concrete to the ingress of chloride ions by diffusion. Chloride conductivity was measured in milliSiemens per centimetre (mS/cm).

Apparatus

- Oven capable of 50° C (± 2° C)
- Vacuum saturation apparatus as shown by Figure 3-20
- Conduction cell with flexible collars shown by Figure 3-21
- DC power supply (90-12 V) , (0-1 A stabilised)
- Digital voltmeter and ammeter
- Electrical cables and plugs
- Scale accurate to 0.01g
- 10 litre container with lid
- CP grade NaCl (99% purity)
- Desiccator
- 5 M sodium chloride solution

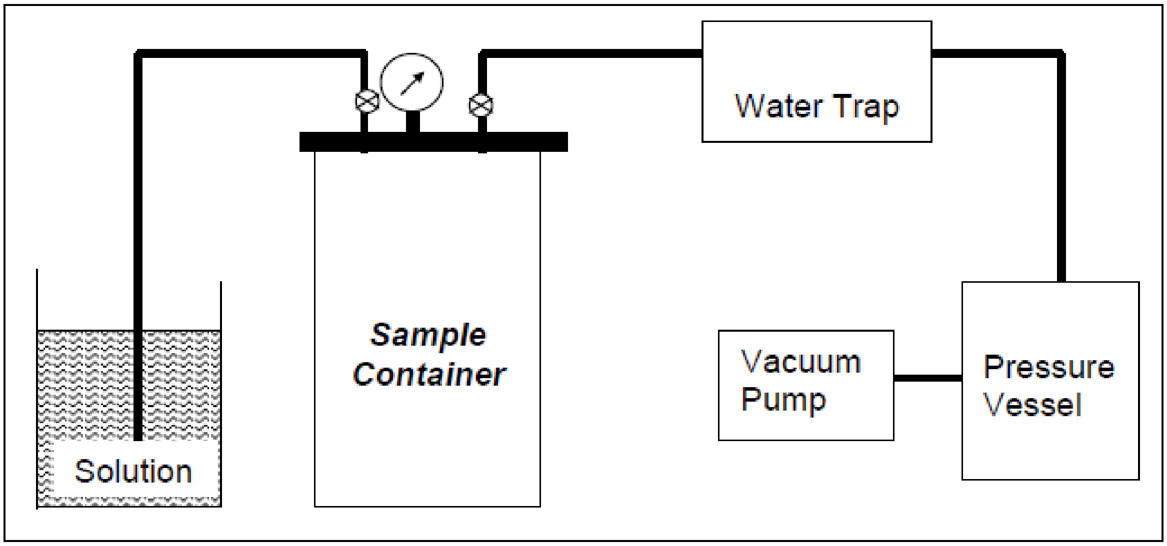


Figure 3-20: Vacuum saturation apparatus (Comsiru, 2009)

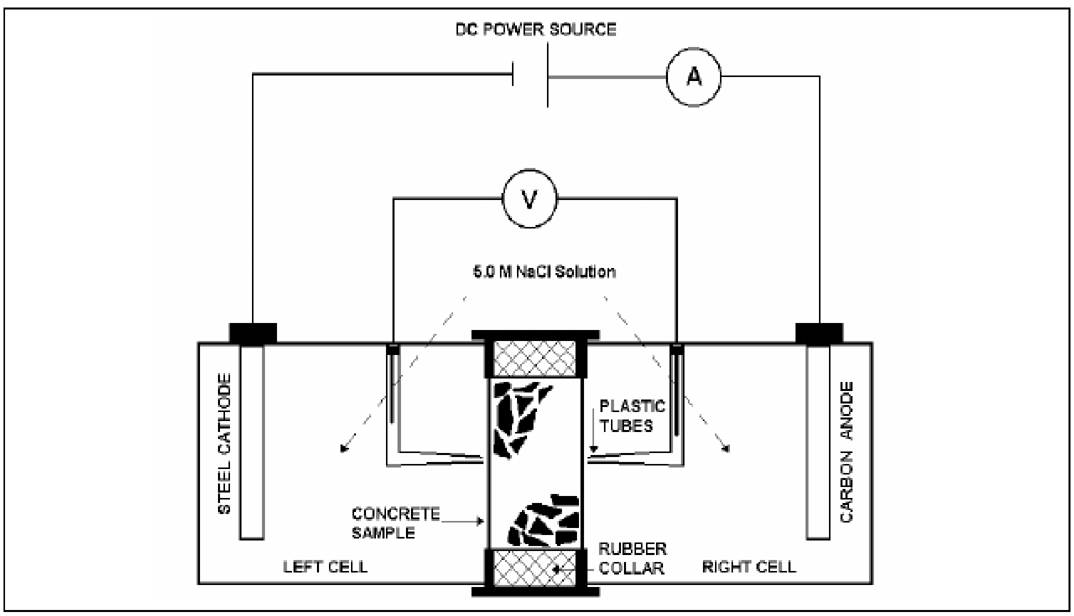


Figure 3-21: Chloride conductivity apparatus (Comsiru, 2009)

Method

Specimens were cast and cut into 4 disks (70 mm diameter x 30 mm thickness) per specimen. After cutting, specimens were dried for 7 days in an oven at 50° C. Specimens were cooled in a desiccator for 2-4 hours. The samples were saturated using the vacuum saturation apparatus in Figure 3-20, for 3 hours under vacuum of -75 kPa to -80 kPa. The vacuum tank was then isolated and NaCl solution was allowed to enter under a constant vacuum of -75 kPa to -80 kPa until the specimens were covered to a depth of 40 mm. After 1 hour the vacuum was released and samples were soaked for 18 hours. After soaking, the specimens were towel dried to saturated surface dry conditions and weighed.

NaCl solution was filled into the plastic tubes shown in Figure 3-21, and the samples were placed in a flexible collar. The use of a highly conductivity (5M) NaCl solution ensured that the test was independent of the concrete pore solution and primarily assessed the physical resistance of the concrete microstructure to chloride penetration (Alexander, et al., 1999). The sample and collar were then inserted into the cell and the voltmeter and ammeter were connected. The DC power supply was then adjusted until there was an approximate voltage of 10 V across the specimen. The voltage and ammeter readings were subsequently taken immediately and after 15 minutes (Comsiru, 2009). The test was carried out at a constant room temperature of 25°C because conductivity increases with increasing temperature.

The chloride conductivity was calculated as follows.

$$\sigma = \frac{\mathrm{id}}{\mathrm{VA}}$$

σ = Chloride conductivity in milliSiemens per centimetre (mS/cm)

i = Electric current (mA)

V = voltage difference (V)

d = average thickness of the specimen (cm)

A = cross-sectional area (cm^2)

The average of the four concrete disks tested for CC for the mixes at critical volume and the mixed waste combinations, were reported by Contest and used for this study.

3.9.3. Water sorptivity (WS)

The water sorptivity test measured the unidirectional ingress of water in a preconditioned concrete sample.

Apparatus

- Oven capable of 50 ° C ± 2° C
- Vacuum saturation
- 20 mm deep tray to hold specimens
- Paper towels
- Measuring scale (0.01g-accuracy)
- Sealant
- Desiccator
- Stopwatch
- Tap water saturated with calcium hydroxide. (5 grams of $Ca(OH)_2$ per 1 litre of water), maintained at 23 ± 2°C.

Method

The disk specimens used for the oxygen permeability test were utilised for the water sorptivity tests.

The WS test was carried out at a room temperature of 23°C .Ten rolls of paper towels were laid out in a tray and were saturated as shown by Figure 3-22 on page 175. Calcium hydroxide solution was poured into the tray with a water layer visible on the top surface. The final water level had to be approx. 2mm up the side of the specimen.

Sealant was applied to the sides of the specimens and the dry mass of the specimens were recorded. Specimens were placed on the paper towels and the stopwatch was started. Specimens were weighed after 3, 5, 7, 9, 12, 16, 20 and 25 minutes. The samples were then removed and placed in a vacuum tank. The samples were saturated using the vacuum saturation apparatus in Figure 3-20 on page 172, for 3 hours under vacuum of -75 kPa to -80 kPa. The vacuum tank was then isolated and NaCl solution was allowed to enter constant vacuum of -75 kPa to -80 kPa until the specimens were covered to a depth of 40 mm. After 1 hour the vacuum was released and samples were soaked for 18 hours. After soaking the specimens were towel dried to statured surface dry and weighed (M_{sv}). The calculations to obtain WS were as follows (Comsiru, 2009):

Line of best-fit (F) by linear aggression analysis.

$$F = \frac{\sum[\sqrt{t_i} - T] - [M_{wti} - \overline{M_{wt}}]}{\sum[\sqrt{t_i} - T]^2}$$

M_{wti} = the mass at any given time in grams

t_i = the time in hours corresponding to the mass gain reading

$$\overline{M_{wt}} = \frac{\sum M_{wti}}{n}$$

$$T = \frac{\sum \sqrt{t_i}}{n}$$

n = number of data points

Water Sorptivity (WS) was calculated as follows.

$$WS = \frac{F\ d}{M_{sv} - M_{s0}}$$

WS = water sorptivity [mm/$\sqrt{h}$]

F = line of best-fit [g/$\sqrt{h}$]

d = average specimen thickness (mm)

M_{sv} = vacuum saturated mass (g)

M_{s0} = initial mass (g)

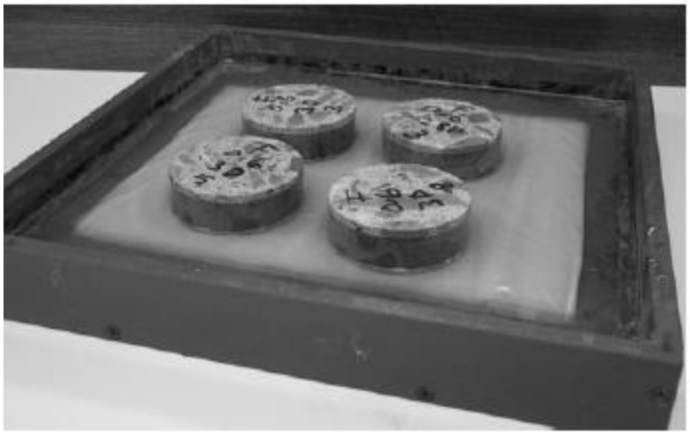


Figure 3-22: Specimens in tank with paper towels (Comsiru, 2009)

The average of the four disk tested for WS for the mixes at critical volume and the mixed waste combinations were reported by Contest and used for this study.

3.10. Scanning Electron Microscope (SEM) imaging analysis

The Zeiss Ultra Plus Field Emission Scanning Gun (FEGSEM) was used for the scanning electron microscope (SEM) analysis. The SEM analysis was conducted at the Westville campus premises of the University of KwaZulu-Natal.

The samples were carefully chipped off from the centres of cast cubes at approximately the same location and were approximately $5mm^2$ in size. The reason for using a small sample was due to the restriction in sample size that could be used in the Zeiss Ultra Plus FEGEM apparatus. This meant that samples would not reflect stones in the mix nor varying proportions of HDPE and SCBF, and would be predominantly cement. Therefore only the aggregate interaction between HDPE and SCBF with the cement mix was investigated with the SEM. The BA samples were also tested using the SEM to investigate BA on a microscopic level. The specimens of approximately 5 mm^2 were placed on stubs with reversible tape, coated in gold using a gold sputter coater and placed in the Zeiss Ultra Plus FEGSEM. The SEM analysis was carried out under vacuum conditions and was done at 5KV.

The SEM imaging was used to enrich the study and provide a visual description of the respective concrete mixes as well as investigate the interface between HDPE, SCBF and cement.

Figure 3-23: Zeiss Ultra plus FEGSEM apparatus

3.11. Moisture effect investigation

As mentioned in section 3.3 the water content of the mixes was kept constant to evaluate the effects on concrete material properties by comparing the waste concrete property results (compression, slump flexure, splitting, elastic modulus, durability, etc.) with the conventional mix. In order to streamline the study, only the critical volume substitutions for each waste and the mixed combinations were tested for the moisture effect investigation. The critical volume wastes were used as these were the best performing waste mixes in compression and the mixed combinations were used to assess the impact of using more than one type of waste in the mix.

Variations to properties such as slump also implied the impact of the waste materials moisture absorption on water demand. For example, if the aggregates were dry, had a high water absorption and if the results subsequently showed a decrease in slump, this would imply that the water demand of the mix was increased.

The effect of moisture state was also compared by testing specimens for compressive cube strengths (f_{cu}) and slump, using mixes with natural moisture state aggregates and mixes with oven dry aggregates after 7 and 28 days of curing. The natural state was selected as this was the state the waste materials would be on site without any special preparation. The oven dried state was selected because in theory this would be when the waste aggregates would have their greatest absorption potential and reflect the biggest impact on the mixes due to moisture properties.

Methodology

The moisture absorption and moisture contents (at natural moisture state) were calculated for each waste material using non-standard tests by Contest and these were related to the tested properties (compression, slump flexure, splitting, elastic modulus, durability etc.)

To further assess the effects of moisture properties on the mix, the waste aggregates were first oven dried at 108° C for 24hrs. The same mix designs as the ones that were done for the mixes at natural state were used but incorporating the oven-dried waste aggregates instead on the waste aggregates at natural state. The slump readings were then taken and the 150mm cubes were cast. The specimens with oven-dry waste aggregates were tested after 7 and 28 days by Contest using the same procedure as discussed in section 3.6.1. The compressive strength and slump results of the specimens with dry waste aggregates were compared to the specimens tested with natural state waste aggregates. The results were discussed, making reference to the moisture absorption and moisture contents of the waste materials as well as any differences in 7 and 28 day strengths.

3.12. Economic analysis

The waste concrete mixes not only had to be strong enough and durable but also economically viable for them to be potentially used in industry. The economic viability of the concrete mixes were assessed in two steps:

- Firstly R/m^3 concrete mix rates were established by costing the mixes based on current market rates using the costing methodology by Popescu, *et al* (2005).
- A scenario analysis was then conducted using the calculated R/m^3 rates for concrete and the costs associated with transport, labour and plant to construct a cast in-situ signalling room and precast platform copings as per the QTO costing method (Dalton, et al., 2011).

3.12.1. Concrete costing in R/m^3

Based on the material cost estimation approach by (Popescu, et al., 2005), local material costs are required to develop a cost/m^3 concrete rate. The local suppliers used, and their rates researched are shown in Table 2-13. Undelivered rates were used to give an idea of purely material cost, with the costs related to the transportation of materials being dealt with separately. The cost of SCBF was not available from a supplier and is explained below.

Cost of SCBF

The rate for SCBF was not publically available due to confidentiality agreements between the Sugar Mill Research Institute (SMRI) and the mills as they are used for power generation feasibility studies. SMRI did however recommend a method to calculate the monetary value of bagasse fibre and this was implemented in this study as follows:

The monetary value of SCBF was established by using a relationship between SCBF and coal, which was used in the boilers as an alternative to burning SCBF as a fuel to generate power for the mill. Under the advice of Sezela Sugar Mill and SMRI, it was decided to take the calorific values in kcal/kg from literature for coal and sugarcane bagasse to obtain a Rand value per kg of sugarcane bagasse based on the market value of coal. The amount of kcal/h to generate 1kW of was found to be 860kcal/h from literature (Unit conversion, 2013).

The amount of kilograms of coal and SCBF to generate a kW of electricity was calculated by dividing 860kcal by the calorific values of respective materials. The R/kW rate of coal was the product of the market R/ton rate and the calculated kg/kW value. This cost was then divided by the kilograms of sugarcane bagasse to generate 1kW of electricity to get the rate in R/kg for SCBF.

SCBF monetary value calculation in summary:

1. Kg of material to generate 1 kW = 860kcal / calorific value

2. R/kW for coal = Kg of coal to generate 1kW x market rate in R.

3. R/kg of sugarcane bagasse = R/kW coal / kg of sugarcane bagasse to generate 1kW

The rates obtained from the suppliers for sand, HDPE, stone and cement (refer to section 2.4) were then used in conjunction with the calculated SCBF rate to cost each mix and obtain R/m^3 waste concrete rates. Refer to section 4.12.1 for the costing of the mixes. In order to truly get an idea of the significance of any cost changes due to the use of waste in mixes, the R/m^3 values had to be applied to real-life scenarios to add purpose to the economic analysis, which was the next step in this study.

3.12.2. Scenario Costing

The QTO method (Dalton, et al., 2011) was used to carry out the cost estimation for the real-life scenario of Pinetown train station works with permission from the contractor. It covered both in-situ and precast concrete applications and was relevant to the local context, due to the current upgrade of passenger rail infrastructure being implemented by PRASA (Passenger Rail Agency of South Africa). The cast-in situ scenario selected was a signalling storage room and the pre-cast scenario was the coping slabs used on platforms. Refer to section 2.4 for further information about the scenarios selected and Appendix K and L for diagrams.

For each scenario the material, plant, transportation costs and labour were all included in the costing.

3.12.2.1. **Material**

For each scenario, a list of materials that would be used and the quantity needed was drawn up as per the QTO method (Dalton, et al., 2011). The items deemed necessary to construct the scenarios are shown in the costing sheet in Appendix J2 and J3. The material costs were then calculated by using rates (R/unit) for each item based on information from a contractor. The concrete material costs in R/m^3 calculated as per section 3.12.1 were applied to the quantities of concrete required for the respective scenarios (Refer to section 4.12.1 for concrete costing). The cost of water was a common cost as all mixtures utilized a similar water content. In practice, the water used would be taken from a metered connection and charges would be based on the eThekwini water tariff guide (eThekwini Municipality, 2014).

3.12.2.2. **Labour**

For the labour cost, the construction duration, labour rate(R/h) and amount of labour required to build the in-situ signalling room and copings were considered as per the QTO method (Dalton, et al., 2011). The labour time was based on an average of 9 working hours a day and the total construction time was based on advice from a civil contractor. The hourly average labour rate in R/h was multiplied by the hours of work for the total construction time and amount of labour required, to obtain the total labour costs. Salary staff considered were the foreman as they were directly involved with the construction of the selected scenario works. The project manager and site agent cost were not considered as the cost estimation was aimed at costs associated with production of each scenario and the site agent and project manager fees would not be included per a single works item. The total labour time considered did not include a 28 day curing time from the casting of the roof as the times stated was the time needed to cast the concrete shell and not the time up until handover.

3.12.2.3. **Plant & Site establishment**

Lists for plant required were drawn up and wet (with fuel) hire rates (R/hour) used by the contractor were applied to the duration that the plant was required for each scenario. Site establishment included the hire and transport costs for a site container.

3.12.2.4. **Transportation**

Transport rates are usually included in deliveries to site, but were separated in this study to show the impact of each aspect of the costing model separately (material, labour, plant, transport). The transport rates for the stone and sand were calculated by taking the difference between delivered (to Pinetown station) and undelivered costs from the suppliers shown in Table 2-18 on page 133. Cement had a fixed delivery charge from the supplier. The transport cost for the HDPE, BA and SCBF were based on the costing methodology by Carter & Troyano-Cuturi (2009) and was calculated taking the rate to hire a 10 m^3 tipper truck or bakkie, depending on quantity of the materials required. The volume of each material was calculated based on the scenario quantities and this indicated how many loads would be required using the load capacity of the tipper or bakkie. The rates for hire were given in R/hour, excluded fuel cost (dry rates) and required a minimum of 9 hours hire. The fuel required was calculated by multiplying the quotient of the distance to the mill and recycling plants over 100, by the average fuel consumption (l/100km) of the 10m^3 tipper or bakkie. The fuel cost was then calculated by multiplying the fuel required by the current fuel cost per litre.

3.12.2.5. **Evaluation**

The economic analysis was assessed in terms of the difference in cost for each of the scenarios using the selected waste concrete mixes versus conventional concrete. Comments were firstly made on the difference in concrete material cost between the waste mixes and the conventional concrete.

The scenario analysis was then used to comment on the percentage that each aspect of the construction (material, labour, plant, transport) contributed to the total cost to construct the scenario. Transport costs were especially significant, considering the distances that needed to be travelled with regard to the waste materials.

3.13. Limitations of this study

The limitations of this study are as follows:

- Tests were mostly standard, however the moisture absorption and moisture content tests on Ash, HDPE and SCBF were non-standard tests, as stated by the concrete testing facility used (Contest) due to the material not being typical concrete materials.

- Assumptions that had to be made for the specific heat analysis were based on laws of thermal behaviour due to the lack of costly specialized machinery and software. For greater accuracy, thermal measuring machinery such as heat flow meters and a Differential Scanning Calorimeter with specific heat and thermal conductivity software should be utilised, neither of which were available for this study.

- It is understood that toughness is a significant property investigated with regards to fibre reinforcement, as fibres act as crack resistors generally increasing the energy absorption capacity *(toughness)* of the mix. However this study looked to compare waste modified mixes with a conventional concrete using the main properties generally associated with conventional concrete specification and not just fibre reinforced concrete, such as strength, durability, workability and cost. Toughness is recommended as a topic for future research in conjunction with research into crack behaviour.

- The equipment used for SEM had a limit on sample size and hence imaging could only be done on small areas that were of interest as opposed to a large area reflecting all the concrete constituents.

- NPC original black 42.5N was used based on the NPC products available to the university. It is known that this does contain extenders, however a blend with the least extender content from products available was selected and this was used as a control (no waste material added) to evaluate the effects of adding waste materials, which in essence is the aim of this research.

- The study did not evaluate the change in W/C ratio and it was kept as a fixed variable. This was because the study focused on evaluating the change in properties due to waste material and not the effect of varying W/C ratios. The method of keeping W/C constant and investigating changes in slump, etc. was also used by Gul, *et al*. (2014). Tests related to varying the W/C ratio would be used more for developing a specification for the material or develop an empirical strength W/C relationship, which could be a topic for future research.

- The elastic modulus test was carried out to provide general additional information on the waste concrete mixes. It did not have a bearing on the primary aim of this study which was to access the viability of the waste mixes in terms of strength, durability, workability and cost. The minimum single sample was used for the elastic modulus test only, however because it was a non-destructive test the specimens for each mix where still tested under three loading and unloading cycles per test. This was confirmed as adequate by Contest Concrete Technology Services Durban and BS 1881: part 121:1983 (BSI, 1983). Four strain gauges were used as opposed to two, to factor in possible errors due to bonding or malfunctioning gauges. The approximate factor of 0.8 was used to convert from cube to cylinder strengths and hence is listed as a limitation to the study. This was however compared to a formula from BS 8110 (BSI, 1983), to evaluate the variance in load points

- The water demand was not changed, as the moisture effects of adding the waste materials was assessed by investigating the change in concrete material properties, and any variation in water demand could be implied from these results. This methodology was used by Rao (2010). The effect of moisture absorption and water content was further evaluated by measuring the slump and compressive cube strength of samples using oven-dry waste (maximum absorption potential) and comparing the results to the results of samples using waste at natural moisture state.

- In terms of sustainability, the embodied CO_2 was not considered in this study. For it to provide meaningful results a full life cycle analysis would be required and this validates an entire research topic on its own.

- An in-depth investigation of shrinkage, cracking and creep behaviour was not part of this study. Due to their complex nature, an investigation of these properties would constitute a separate research topic at MSc level, as carried out by Gaylard (2011). Elastic modulus was however investigated in this study, and the relationship of elastic modulus to shrinkage and cracking was discussed where possible.

- The durability tests on the SCBF mixes did not factor in the effect or rate of potential natural fibre deterioration which may affect long-term concrete durability.

- The cost analysis was just the production cost for the scenarios and not the total projects cost. The duration analysed was just the construction duration and not the duration up until handover which would include curing time. The cost analysis did not take into account the project life costs as this detail would constitute a separate study.

3.14. Chapter summary

The methodology implemented for the study was structured to meet all objectives stipulated in the introductory chapter and achieve the research aim of investigating the potential viability of the selected waste materials used for concrete in the local context.

The study was predominantly experimental based, but included a cost based scenario analysis and SEM imaging analysis. Knowledge gaps found in the literature review were also addressed where possible thus expanding knowledge on the topic for future research.

Testing procedure were mostly standard apart from a few exceptions such has specific heat which was non-standard due to equipment limitations. The research approach was also clear and focused more on the effect on concrete properties of adding the waste materials in volumetric proportions, as opposed to the effect of varying W/C ratios or cement types. The effect of moisture on factors such as compressive strength, slump and subsequently the W/C ratio were however investigated as stipulated in section 3.11, but again this was the implied effect on W/C not the effect of changing the W/C ratio.

By following the methodology, all research objectives stipulated for the study were met, such as finding the critical volume for each waste, investigating the effect on properties such as strength durability and elastic modulus and establishing whether the waste worked better independently or in combinations. The scenario analysis carried out also provided the reader with an indication if there was any reasonable economic impact achieved by using the waste materials.

Considering the results of the analyses stated in the methodology, a conclusion on the "potential" viability of the mixes could be made if the mixes met the required strength and durability requirements whilst still being economically feasible. The next chapter will provide the reader with the results of the study by following the methodology stipulated in this chapter and include a subsequent critical discussion of the findings.

Chapter 4 **Results and Discussion**

4.1. Introduction

The study consisted predominantly of experimental analyses and included an economic analysis to conclude the viability assessment of the selected waste materials in concrete. The results of the methodology described for this study in chapter 3 will be explained in Chapter 4 as follows:

- Constituent material properties (fineness modulus, relative density, particle size, moisture absorption, moisture content) used for the mix design and related to the concrete properties will be reported and evaluated.
- The calculations used for the mix design explained in section 3.3 will be shown, to indicate the process taken based on the C & CI design method.
- Workability of all mixes will be discussed to indicate to the reader the effects on fresh properties.
- Strength properties (Compressive, Tensile, and Splitting) of all mixes will be discussed considering densities and workability. Critical volumes between 2,5% - 40% substitution will be established for each waste material based on compressive strength results.
- Density of the all mixes will be discussed in terms of mass/volume as well as the theoretical increase/decrease in void area due to hydration. This will be related to concrete properties such as durability and strength.
- Elastic modulus of the critical volume and mixed combination mixes will be discussed.
- Effects of moisture properties on the critical volume and mixed combination mixes will be reported by comparing the compressive strength and workability tests carried out using oven-dry waste materials, with mixes using wastes at natural moisture state.
- Concrete specific heat results for all mixes will be discussed.
- Durability of the critical volume and mixed combination mixes (Oxygen permeability, water sorptivity, chloride conductivity) will be discussed.
- SEM analysis will be carried out to investigate samples on a microscopic level.

- The economic analysis for critical volume and mixed combination mixes will be discussed firstly showing the calculations carried out for the R/m^3 rate and secondly the in-situ and precast scenario analyses.
- Conclusions will be made on the viability of the mixes at critical volume and mixed combinations.

4.2. Constituent material properties

Constituent material properties were obtained from either experimental testing or from literature. These material properties were used for the mix design and to discuss effects on resulting concrete properties. The aggregate properties that were calculated were the compacted bulk density (CBD) for stone, fineness modulus for sand, moisture absorption for the waste aggregates, moisture content for the waste aggregates and the relative densities for all the aggregates.

4.2.1. 13.2mm stone compacted bulk density (CBD)

Table 4.1 below shows data for the CBD in kg/m^3 of 13.2 mm stone after 10 trials. This was obtained using the method explained in sub-section 3.2.1. The CBD was applied to the stone content formula in the C & CI mix design method.

Table 4-1: 13.2mm Stone CBD

COMPACTED BULK DENSITY OF 13.2MM STONE			
Volume of cylinder	0,002758	m^3	
TEST NUMBER	Mass (g)	Mass (kg)	Density (kg/m^3)
1	4069	4,069	1475,25
2	3982	3,982	1443,71
3	4164	4,164	1509,7
4	4053	4,053	1469,45
5	4149	4,149	1504,26
6	4162	4,162	1508,97
7	4156	4,156	1506,8
8	4118	4,118	1493,02
9	4100	4,1	1486,49
10	4202	4,202	1523,48
Average			1492,11

4.2.2. Sand fineness modulus

The fineness modulus (FM) was applied to the stone content formula in the C & CI mix design

FM= $(\sum of\ culmintive\ percentages\ of\ retained\ material\ on\ each\ sieve\ 0.15mm\ and\ coarser\ ) \div 10$

The average masses retained are shown in Table 4-2. Please refer to Appendix A for full data.

Table 4-2: Average percentage retained

SAND FINENESS MODULUS	
Sieve Size	Average percentage retained
4,75	0,96
2,36	6,92
1,18	46,52
0,6	87,88
0,3	98,52
0,15	99,64
FM	3,4044

The results showed that the sand had an average FM of 3.4. This value was high due to the presence of more coarse particles. The range quoted in the U.S army standard CRD-C 104-80 (U.S Army, 1980), states that for fine aggregates the FM should be between 2-4. So although the sand was closer to the upper bound of that range, it was accepted. The stone content in the mix would however be lower than usual, as per the C & CI stone content formula, to reduce the likelihood of the mix becoming harsh.

4.2.3. Relative densities

The purpose of finding relative densities was not only for converting from mass to volumes in the mix design but also when substituting materials where a mass had to be weighed out for a volumetric percentage based substitution. Two different methods were applied as the aggregate sizes varied.

4.2.3.1. **HDPE and Stone**

For the HDPE and stone aggregates Method B 14 from TMH 1 (CSIR, 1986) was used to calculate the densities. The following formula was used and the experiment was conducted in triplicate:

$$\text{Relative density} = \frac{Oven\ dry\ mass}{Oven\ dry\ mass - (submerged\ mass - Mass\ of\ holder)}$$

Table 4.2 shows the data applied to the relative density equation to calculate the relative density of HDPE.

Table 4.2: Relative density of HDPE 4-3

RELATIVE DENSITY OF HDPE PLASTIC		
Mass of holder	185,1	g
Submerged mass (g)	**Oven-dry mass (g)**	**Relative density**
168,8	197,7	0,924
170,5	200,5	0,932
168,6	195,4	0,922
	Average	0,926

The average of three trials was found to be 0.926. This was compared to the literature value of 0.95 (Dynalab, 2013). The percentage difference to the literature value was 2.52%. Taking into account the possibility of human error and HDPE composition, the difference of 2.52% was satisfactory. The RD of HDPE was less than water and hence it would have the tendency to float. Remedial measures as stated in section 2.2.3.3 were therefore taken to mitigate particle segregation.

Table 4-4 below shows data for the relative density of 13.2 mm stone after 5 trials.

Table 4-4 Relative density of 13.2mm stone

RELATIVE DENSITY OF 13.2mm STONE		
Mass of holder	185,6	g
Submerged mass (g)	Oven-dry mass (g)	Relative density
816	987,5	2,77
821,4	991,1	2,79
821	988,8	2,8
846,3	1030,5	2,79
844,2	1024	2,8
	Average	2,79
Relative density	2788,36	kg/m^3

The average of five trials was found to be 2788 kg/m^3. This was compared to the literature value of between 2600 kg/m^3 and 2950 kg/m^3 which averages approximately 2775 kg/m^3 (Addis, 2008). The difference between literature and the measured value was 0.47%, which was acceptable considering that stone composition can vary slightly from sample to sample (Lewis, 1995).

4.2.3.2. **Sand**

For both the sand and the ash, the process was according to the standard concrete design method B15 from TMH1 (CSIR, 1986), because they were both too fine in particle size for method B14. Table 4-5 below shows the data obtained from the testing.

Table 4-5 : Sand density

SAND DENSITY		
Mass of empty flask	254,3	g
Mass of oven-dry sample + flask	554,1	g
Mass of sample + water + flask	1436	g
Mass of water + flask	1251,5	g
Relative Density of sand	2,6	

The density of the sand was calculated by using the formula below.

$$Realtive\ density\ of\ sand = \frac{(mass\ of\ oven\ dry\ sample\ and\ flask - mass\ of\ empty\ flask)}{(mass\ of\ flask\ filled\ with\ water - mass\ of\ empty\ flask) - (Mass\ of\ flask\ with\ water\ and\ sample - (mass\ of\ oven\ dry\ sample\ )}$$

$$Realtive\ density\ of\ sand = \frac{(554.1 - 254.3)}{(1251.5 - 254.3) - (1436 - 554.1)} = 2{,}6$$

This was the same as the literature value of 2, 6 (Reade Advanced Materials, 2006).

4.2.3.3. **Coal bottom ash**

The coal bottom ash was sieved through a 1400 μm aperture sieve of 210 mm diameter to remove foreign particles such as stones. Table 4-6 shows the data obtained from testing.

Table 4-6 : Density of coal ash

DENSITY OF COAL ASH		
Mass of empty flask	254,9	g
Mass of flask + sample	554	g
Mass of sample + water	1405	g
Mass flask + water	1251,8	g
Relative density of coal ash	2,05	

The relative density of the coal bottom-ash was calculated by using the same formula as with sand.

$$\text{Relative density} = \frac{(554 - 254.9)}{(1251.8 - 254.9) - (1405 - 554)} = 2.05$$

This was compared to the literature value of 2.4 (Federal Highway Administration, 2012). The relative density value measured differed from the literature value by 15 % because the bottom ash used was sieved, altering it from the typical bottom ash literature value.

4.2.3.4. **Sugarcane bagasse density**

Due to the nature of the SCBF, the relative density method used for HDPE and ash was impractical for use with SCBF and a literature value of 1.25 (C&CI, 2013) was used for this study.

4.2.3.5. **Cement**

Cement was obtained from the supplier data sheet as 3.7 (NPC, 2013).

4.2.4. Moisture properties

The moisture absorption properties tested by an external company using the formulae for moisture absorption and moisture content from section 3.2.4 are shown in Table 4-7.

Table 4-7: Waste material moisture properties

Material moisture properties		
	Tested Moisture Absorption	Moisture content
HDPE	0,05%	0,01%
SCBF	181,80%	62,60%
BA	1,8 %	0,90%

In comparison to the literature values for the respective waste materials, HDPE varied by 0,02% when compared to the value stated in Table 2-5 on page 73. SCBF was within the range stated in Table 2-4 on page 57, and BA was within the range stated in Table 2-8 on page 85.

The moisture absorption and moisture contents of HDPE were low as HDPE is not an absorbent material and is hydrophobic. The relatively high moisture values for SCBF indicated that it is a highly absorbent material and even though it was quite moist at natural state it still had an absorption potential of 119,2%. This absorbency would draw water from the mix. This could result in reduced strength if too much water is absorbed and there is insufficient water available for cement hydration. Alternatively, due to the W/C being lowered this could enhance compressive strength if cement hydration was not hindered. The workability of the mix would be expected to decrease independent of the effect on strength due to moisture loss. The high absorption of SCBF was due to the high cellulose content. The BA, due to its porous nature, would in theory exhibit similar behaviour to SCBF but to a much lesser extent.

4.2.5. Material sizing

SCBF had fibre lengths from 45mm to 100mm for the rind fibres and between 1-5mm for the short pith fibres as shown in Figure 4-1. This may cause a problem with workability but also may improve tensile strength as it provide a reasonably long bond surface to transfer load and bridge macro-cracks, as per research carried out by Wang, *et al*. (2000) and Vandewalle (2006).

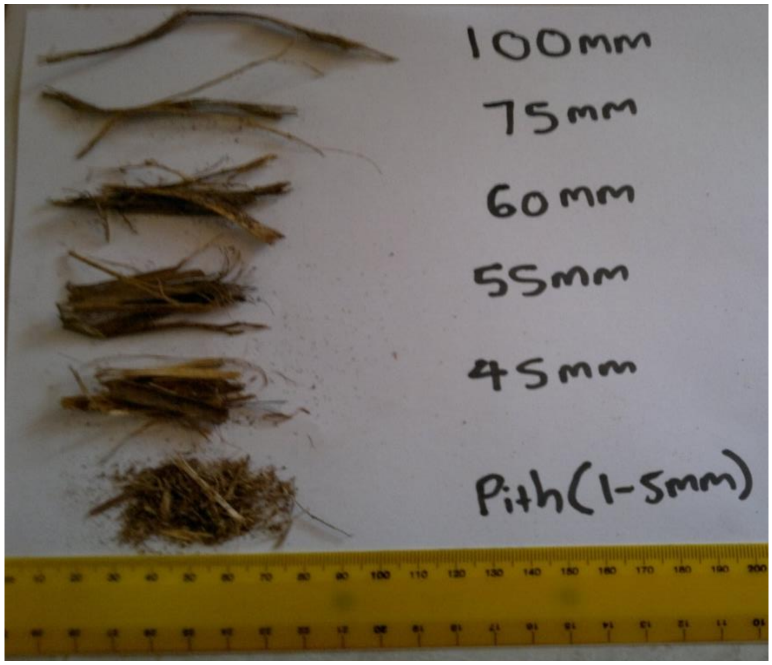


Figure 4-1: SCBF length measurements

HDPE was measured to be 2mm x 2mm cylindrical pellets, which means it fits in between the stone and sand aggregate sizes and hence may improve the particle packing of the concrete mix.

Figure 4-2: HDPE pellets

The particle size of the sieved BA was measured to be approximately between 0,76 μm and 1,5 μm from the SEM image shown in Figure 4-4. This is similar to fly-ash which can be around 1μm-20μm (University of Memphis, 2014). In theory, this means the sieved BA in this study should be just as reactive as fly-ash based on comment made by Magistri, *et al* (2011) relating reactivity to particle size.

Figure 4-3: BA sample

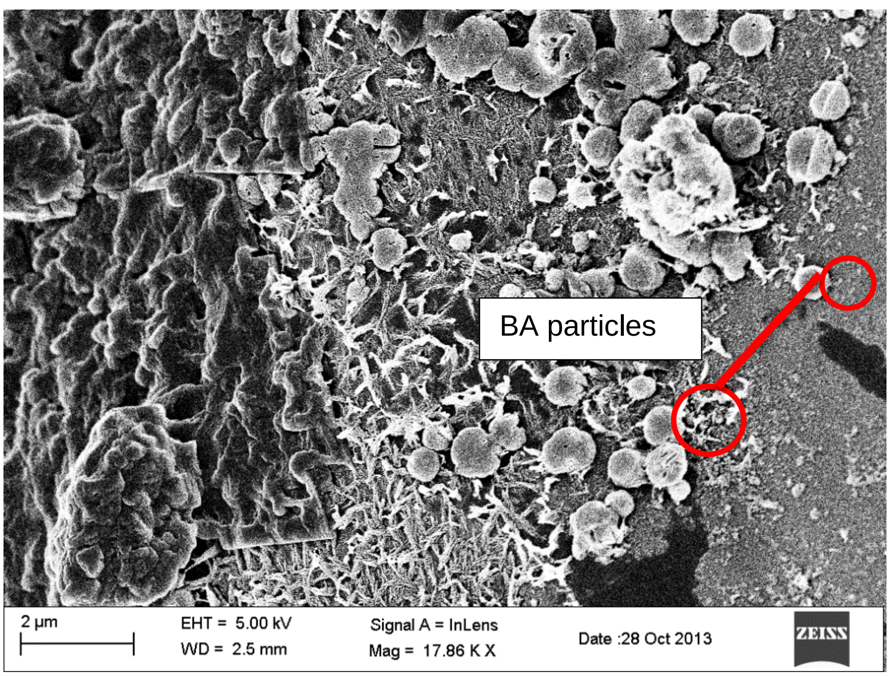


Figure 4-4: SEM of BA specimen

4.3. Mix design

The mix design followed the C&CI method explained in chapter 3.

The W/C ratio of 0,57 was applied to Figure 3-7 on page 151, to obtain a design compressive strength of 35 MPa at 28 days. The water requirement was specified as 225ml for 13.2 mm stone. The mass of water content was divided by the W/C ratio to obtain a cement mass of 394kg.The K-factor was selected as 0.9 for moderate vibration and stone content was then calculated as shown below:

St = CBDst (K-0.1FM) = 1492*(0.9 - 0.1*3.4) = 835 kg

Where;

CBDst = Compacted Bulk Density of stone [kg/m^3]

K = Factor depending on workability and nominal stone size

FM = Fineness Modulus of sand

The masses of stone and cement was converted to a volume using relative densities and the sum total of cement, water and stone was subtracted from 1000 litres to obtain the volume of sand.

Cement = 394 kg / 3.07 = 128.58 litres

Stone = 835 kg / 2.788 = 299.45 litres

Water = 225 litres

Sand = 1000 – 128.58 – 299.45 -225 = 346.97 litres, converted to mass = 346.97 x 2.6 = 902.12 kg

The above calculations gave the mix proportions for the control mix. For each waste modification, the percentage of waste substitution was multiplied by the volume of material it replaced in the control mix (cement or stone). To calculate the mass of the waste material, the calculated waste volume was multiplied by its relative density. The following volumetric substitutions were considered for bottom ash, HDPE and SCBF: 2.5%, 5%, 10%, 20% and 40%. Ash + HDPE, Ash + SCBF and Ash + HDPE +SCBF mixes were also considered by using the same volumetric substitution method described above but with combinations of waste , taking the critical volume percentages for each waste obtained from the compressive strength tests instead of the entire range of substitutions. Refer to Appendices C1-4 for mix designs in terms of proportions per a cubic metre of concrete. From this the required material quantities were batched by dividing each quantity of constituent material per a cubic metre of concrete by 1/ (quantity of concrete required).

For example, if 0.05m^3 was required then all quantities to make a cubic metre of the concrete would be dividing by 1 / 0.05.

4.4. Workability of waste concrete

4.4.1. Individual waste materials

The fresh properties (workability) of the waste mixes were assessed using the slump test as described in section 3.4. The mixes containing individual wastes materials were first tested in varying proportions and the average workability results are shown graphically in Figure 4-5. The average workability results for the mixed waste combinations are shown in Figure 4-6 of section 4.4.2, using the critical volumes identified from the compressive strength tests in section 4.6.1.

Table 4-8 shows the average slump results (refer to Appendix D for full data) for the control (0%) and waste mixes (HDPE, SCBF, BA) in proportions of 2.5%, 5%, 10%, 20% and 40%, which are plotted in Figure 4-5.

Table 4-8: Average slump results

Average slump values for waste mixes (mm)			
Control = 74 mm	HDPE	SCBF	BA
2,50%	81	76	74
5%	91	65	73
10%	117	63	70
20%	125	57	68
40%	140	55	65

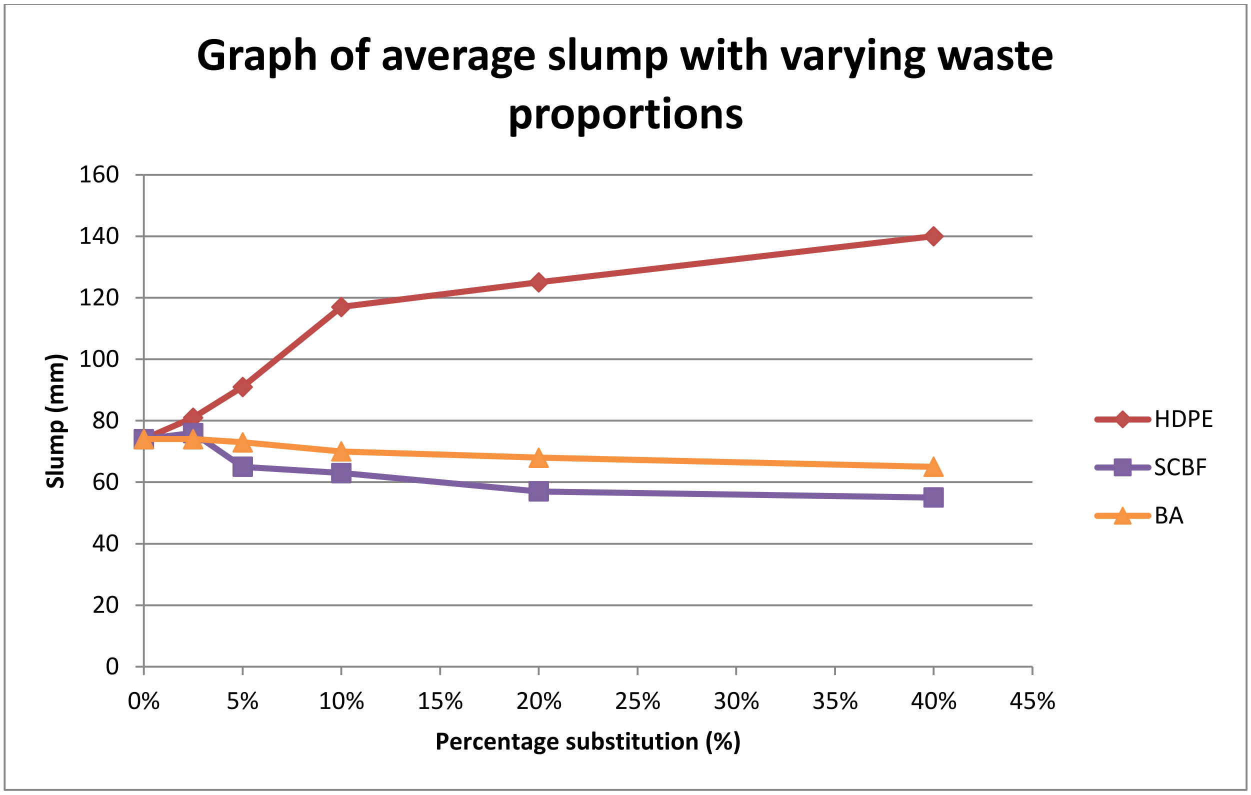


Figure 4-5: Workability (slump) of Individual waste with varying proportions

As stated in section 2.3.3, material properties and mix proportions have a significant effect on the workability of a mix. This was evident in the workability results of the waste aggregate mixes which showed fluctuating performance with varying proportions of waste and waste material types.

The conventional mix achieved a slump of 74 mm, which was within an acceptable tolerance from the 75 mm target slump and hence the control mix proportions were accepted for the study. The conventional mix is show in Figure 4-5 as 0% substitution. The waste mixes were compared to the control slump to gauge the effects of adding the wastes to a conventional mix.

An inverse relationship between SCBF fibre content and slump was noticed as the samples containing SCBF showed a general decrease in workability with increasing proportions of fibres. These findings confirm similar findings on the effect of fibres on workability by Jorillo & Shimizu (1992), Silva & Suely (1984) and Vandewalle (2006). The SCBF mix showed a general decline in slump when compared to the conventional mix. At 5% SCBF substitution the slump of 65mm was 12% less than the conventional mix. This further decreased to 55 mm at 40 % SCBF substitution, which was 25% less than the conventional mix. A slump of 50-100 mm is generally used for general applications as stipulated in section 2.3.3 and the slump at 40% still fell within this range (The Constructor, 2014). The general decrease in workability could have been attributed to the moisture absorption of the fibres which reduced the fluidity of the mix (literature Table 2-4 on page 57 & section 4.2.4) as well as the interlocking of long fibres within the mix, as observed by Sivarja, *et al* (2010). There was a slight increase of 2.7% in workability relative to the control mix, noticed at 2.5% substitution. This could have been attributed to minor variations in the sand moisture and the lack of absorption effect from the SCBF due to the low-volume substitution. It can therefore be said that the effect of SCBF is significant at substitutions above 2.5%.

The slump test results indicated that the effect of HDPE on workability was significant at both low and high volume substitutions. An almost linear increase was noticed between 2.5% and 10%, substitutions, after which the rate of change decreased. This indicated the possible effect of particle interference at high volume substitutions. The HDPE pellets increased the workability of the mix from 81 mm at 2.5% HDPE substitution, to 140 mm at 40% HDPE substitution, which meant a maximum slump increase of 89 % relative to the conventional mix. Figure 4-5, shows a direct relationship between volumetric HDPE content and workability, as the workability increased with increasing volumes of HDPE in the mix. This could have been attributed to the hydrophobic nature and smooth surface of the HDPE pellets having poor adhesion with the cement paste thus reducing interlocking resistance (Weymouth, 1933). This confirms findings by Ahmad, *et al* (2008) and Ferreira, *et al* (2012) on the effect of smooth surface and cylindrical shapes on workability. The poor adhesion may impact on other concrete properties such as compressive strength and elastic modulus which are dependent on the transfer of load from the cement paste to the aggregates.

The bottom ash (BA) samples did show a relatively constant decrease in workability with increasing BA substitution, however not as much as the SCBF which was a more absorbent material. The slump of BA mixes compared to the conventional mix was reduced by a maximum of 12 %, to 65 mm at 40% volumetric substitution. The relatively low deviation from the control mix indicated that BA behaved similarly to the cement it replaced in terms of its effect on workability. This was possibly because the BA was sieved before use to reduce the average particle size. The porous nature of BA compared to cement was possibly why there was a general decrease in slump.

The workability of all waste specimens were within a reasonable range of 40 mm to 150 mm for general concrete use. However, the SCBF and BA mixes at higher volume substitutions would not be recommended for thin sections or members with closely packed reinforcing.

4.4.2. Mixed substitutions

Figure 4-6 shows the results of the workability tests on the mixed waste combinations and critical individual volume substitutions. The mixed waste combinations were combinations of the selected wastes using the respective critical/optimum volume substitutions from the individual waste compressive strength results, as stated in section 3.1.

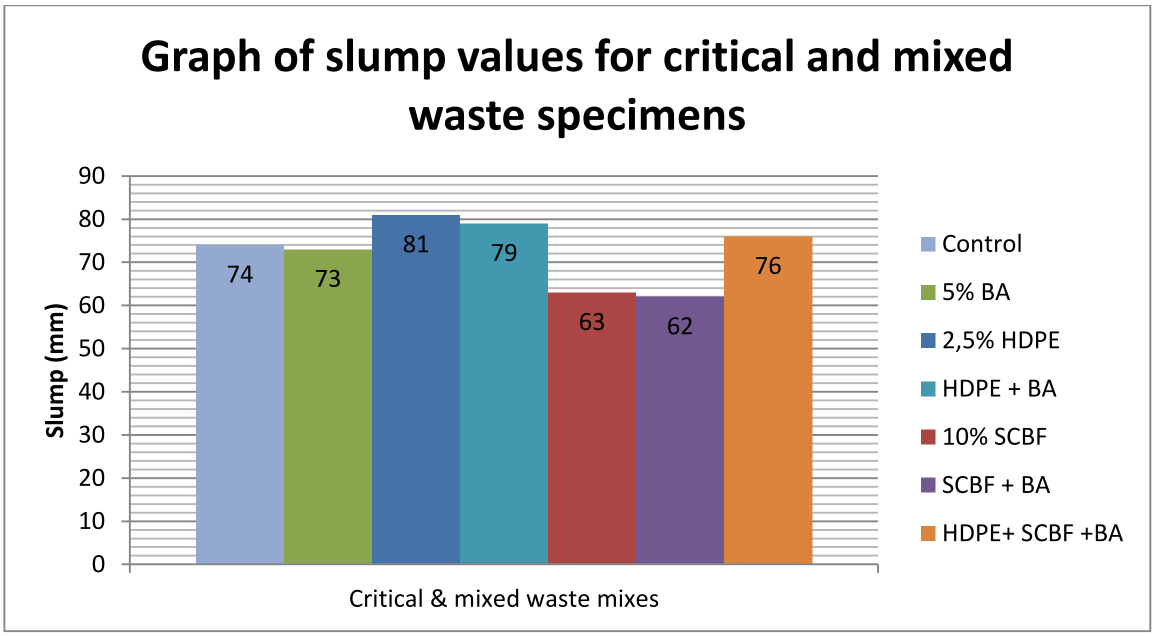


Figure 4-6: Workability of mixed waste combinations

When partially substituting cement in the critical volume SCBF and HDPE waste mixes with 5% of BA by volume, the mixes remained relatively workable (between 50-150mm). However, slight reductions of 1.6% and 2.5% from the respective SCBF and HDPE individual waste mix slumps were noticed. This indicated that HDPE was more sensitive to the inclusion of BA than SCBF. When all three wastes were added together using the respective critical volume substitutions, the resulting slump increased by 2.7% when compared to the conventional mix. This was possibly due to the poor bond characteristic of the HDPE pellets. The increase in slump was however less than the HDPE mix on its own due to the absorbency properties of BA and SCBF.

To conclude on workability, it was noticed that proportions of waste, as well as the material properties (absorbency, surface texture) of the waste do have an impact on the workability of the concrete, which confirms findings by Weymouth (1933). Workability is generally task specific so this may not affect the viability of the mixes as long as the concrete's workability is suitable for the task it is intended for. The SCBF and BA mixes were still within the S2 class (refer to Table 2-13 on page 102) generally applied to most concrete applications. HDPE ranged from a slump class of S2 to S3. S3 is generally used for thin concrete sections or trench foundations.

4.5. Hardened saturated concrete density

Hardened saturated density was tested in accordance with section 3.6. The individual `waste and mixed waste specimens were tested at saturated state prior to compression testing after 7 and 28 days. This allowed for a discussion not only on the effect of varying waste proportions, but also the changes in density from 7 days to 28 days due to curing. The discussion was split into two parts:

- A section on the individual waste mixes focused on the effects of varying waste proportions on the hardened concrete density.
- A section on mixed waste combinations discussing the effect on hardened concrete density by adding bottom ash (BA) to the High-density polyethylene (HDPE) and sugarcane bagasse fibre (SCBF) critical volume mixes.

4.5.1. Individual waste mixes

The average density results for the individual waste materials are shown in Table 4-9 (refer to Appendix B for full data). The control mix yielded a density of 2386 kg/m^3 which differed from the literature value of 2400 kg/m^3 by 0.58% and was deemed accurate for this study (Elert, 2001). The results showed that each waste material decreased the density of concrete but to a different extent. There was also a change in density noticed when comparing 7 day densities (Figure 4-7) to 28 day densities (Figure 4-8) as shown graphically in Figure 4-9.

Table 4-9: Table of Saturated hardened densities

Hardened Saturated density average values								
Control density (kg/m^3)		**Material**						
7 day	28 day	HDPE (kg/m^3)		SCBF (kg/m^3)		BA (kg/m^3)		
2378,07	2387,41	7 day	28 day	7 day	28 day	7 day	28 day	
Volumetric substitutions	2,50%	2336,00	2385,78	2345,78	2384,2	2372,05	2386,67	
	5%	2335,70	2339,26	2338,67	2373,83	2371,56	2379,85	
	10%	2329,38	2335,41	2319,01	2369,68	2362,86	2365,33	
	20%	2224,1	2232,00	2272,89	2330,67	2324,94	2330,07	
	40%	2119,9	2139,16	2266,27	2274,57	2290,86	2313,48	

Figure 4-7 shows the variations in densities of the individual waste mixes (HDPE, SCBF, BA) after 7 days of curing.

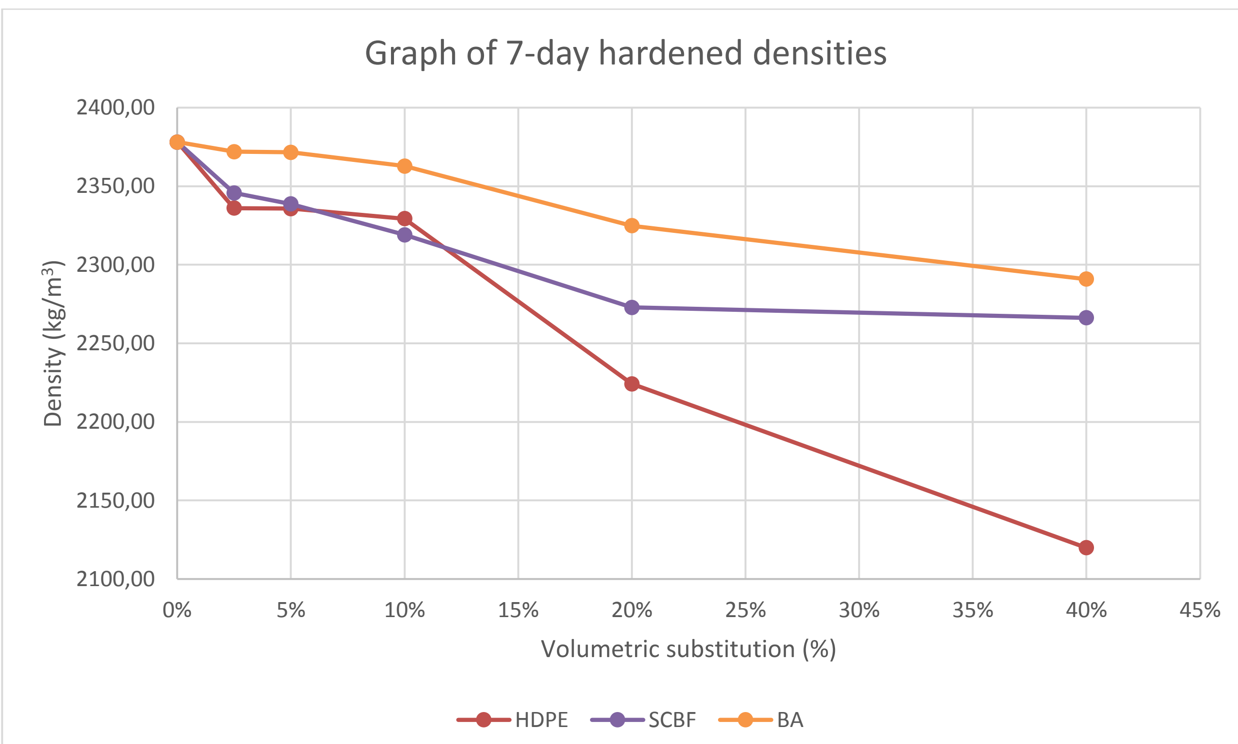


Figure 4-7: Graph of 7 day hardened densities

Bottom ash

The 7-day densities shown in Figure 4-7, for the BA mixes were the least sensitive to the increasing volumetric substitutions and decreased gradually from the 2378,07 kg/m^3 control mix density to 2290,86 kg/m^3 at 40% BA substitution.

The extent of reductions in 7-day densities varied from 0.26 % at 2,5% volumetric substitution, to 3.67 % at 40% volumetric substitution compared to the control mix. Variation in density remained below 1% up to a volumetric substitution of 10%. This indicated that below 10% substitution, the effect on 7 day density is almost negligible and could even be attributed to slight variations in cement composition, which can vary from bag to bag as stated by Ultratech concrete (2014).

The density could have also decreased due to the less reactive BA not forming C-S-H after 7 days. This statement was based on a literature investigation by Singh & Siddique (2013), who stated that the pozzolanic reaction only takes effect at curing ages from 56-90 days. Therefore the BA possibly did not close off pores in a similar manner to the cement particles it replaced, thus lowering density relative to the conventional mix.

SCBF

The densities of the mixes containing SCBF shown in Figure 4-7, also indicated a decline in density as with BA, but decreased more sharply from the 2378,07 kg/m^3 control mix density to 2266,27 kg/m^3 at 40% volumetric substitution

The extent of reductions in 7-day densities varied from 1,36 % at 2,5% volumetric substitution, to 4,7 % at 40% volumetric substitution compared to the conventional concrete. The changes in density were due to the SCBF having a lower density than the stone aggregates it replaced.

HDPE

The densities of the HDPE mixes shown in Figure 4-7, also declined as with the other waste mixes and showed similar behaviour to SCBF. However, there was a rapid decrease in density of 8% relative to the conventional mix from 10% volumetric substitution to 40% volumetric substitution.

The decrease in 7-day density was mainly due to the density of the HDPE being less than stone. This is similar to the reasoning for the sugarcane bagasse fibres but, unlike SCBF, HDPE is also hydrophobic. Above 10% substitution, the effect of reduced bonding with cement particles and increased void content decreased the density of the mix to a greater extent than the SCBF mixes. The possible effect of improved particle packing as stated by Shen (2007), was also not noticed as densities did not increase and could have been negated by the effect of poor aggregate-cement bond properties as mentioned. Further confirmation will be reached after the investigation into durability in section 4.9.

Figure 4-8 shows the variations in density after 28 days of curing, for the three individual waste mixes.

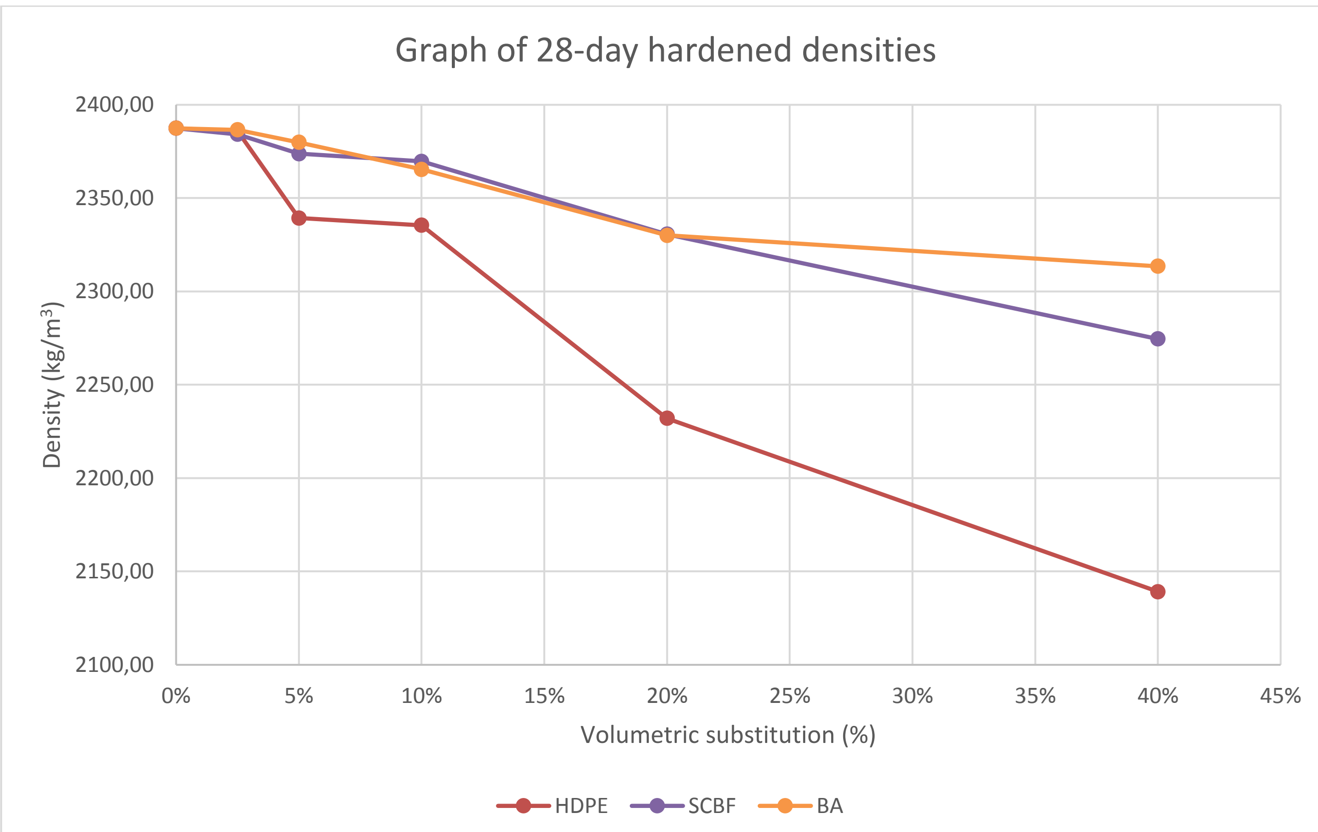


Figure 4-8: Graph of 28 day hardened densities

The results for the 28-day density tests showed increases from the 7-day tests due to the hydration reaction blocking pores and hence reducing the densities of the mixes (Refer to Figure 4-8). All wastes still showed a decline in density with increasing volumetric substitution however, not as significant as the 7 day results in the cases of SCBF and BA.

The SCBF mix decreased by 0.13% at 2.5% substitution and 4.73% at 40% substitution relative to the control mix. The BA mix decreased by 0.03% at 2.5% substitution and 3.1% at 40% substitution relative to the control mix. This indicated that at substitutions below 10% the effect of SCBF and BA on density is negligible. The minimal reductions in BA mix density may have been attributed to variations in the cement composition used, or the lack of pozzolanic reaction products to replace the hydrated particles from the cement substituted out of the mix. The reductions in density for the SCBF mixes could have been attributed to the lighter mass of the SCBF than the stone it replaced, variations in the cement composition used or additional voids been introduced to the mix. HDPE, however, still showed a significant decrease in density from 0,07% at 2,5% substitution to 10,4% at 40% substitution. This indicated that the hydrophobic nature of the material had a significant effect on the concrete matrix and hence the density. The findings on the effect of HDPE on concrete density supports past research by Al-Manaseer & Dalal (1997), who showed similar sharp decreases in density above 10% substitution. The effect on compressive strength and permeability will be investigated in section 4.6.1 and 4.9 respectively, to confirm further findings by Girard (2011), Al-Manaseer & Dalal (1997) that stated that compressive strength and permeability also decrease with reductions in density.

Figure 4-9 shows the percentage changes in density from 7 to 28 days. The increases in density relative to the 7 day density, support findings by Lamond & Pielert (2006), who stated that as the hydration reaction proceeds through curing, hydration products grow and occupy void spaces, therefore increasing density. BA showed less sensitivity to curing age than the aggregate substitutions which indicate the importance of the interfacial zone between the aggregate and cement. BA and HDPE generally increased by a similar percentage to the control indicating that changes to density were mainly due to hydration. The higher increase in density of the SCBF specimen could be due to SBCF absorbing free water and decreasing the W/C ratio in the interfacial zone which may have reduced porosity after 28 days. Although the HDPE mix showed a higher percentage change in density from 7 to 28 day curing times at 2,5% substitution, it must be considered that this change was just above 2% and as with the other mixes which reflected even lower changes (<1%), was most likely attributed to slight variations in composition of the cement used.

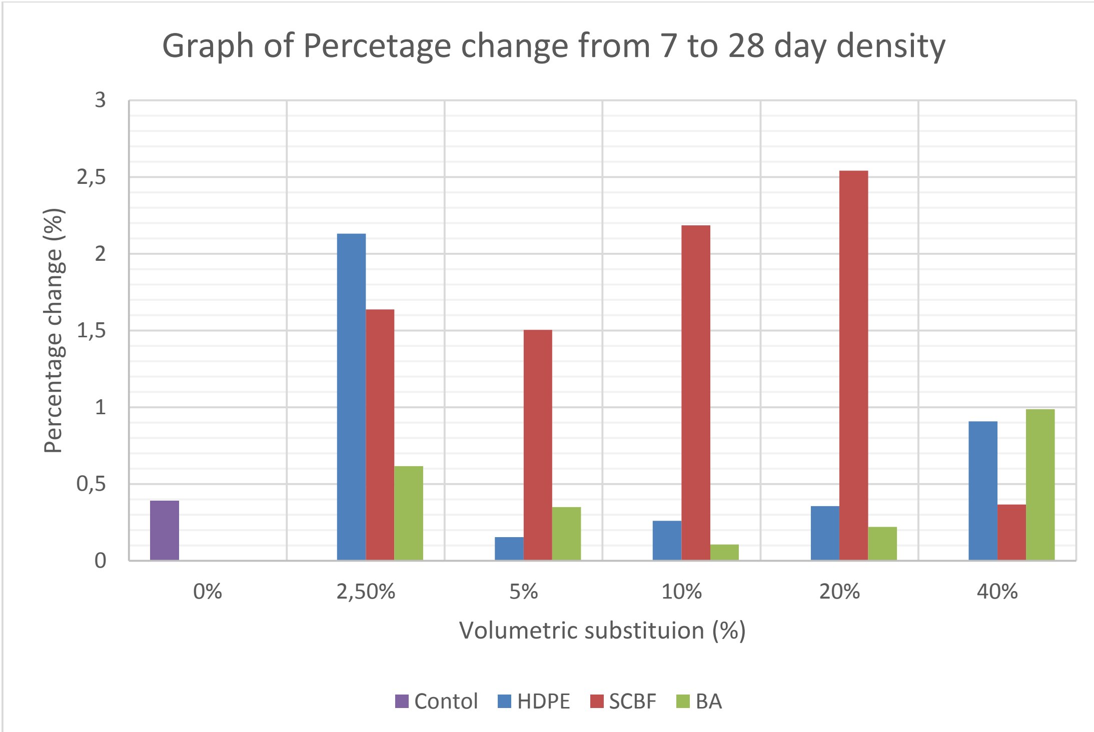


Figure 4-9: Percentage change in density from 7 to 28 days

4.5.2. Mixed waste combinations

Figure 4-10 shows the densities at 7 and 28 days for the critical volume individual waste mixes and mixed waste specimens. The critical volumes of 2.5% for HDPE, 10% for SCBF and 5% for BA, were based on the volumetric substitutions that yielded the optimum/critical volume compressive strengths for the individual waste mixes (See section 4.6.1). As stated in section 3.1, the critical volumetric substitutions were also used for the mixed waste specimens.

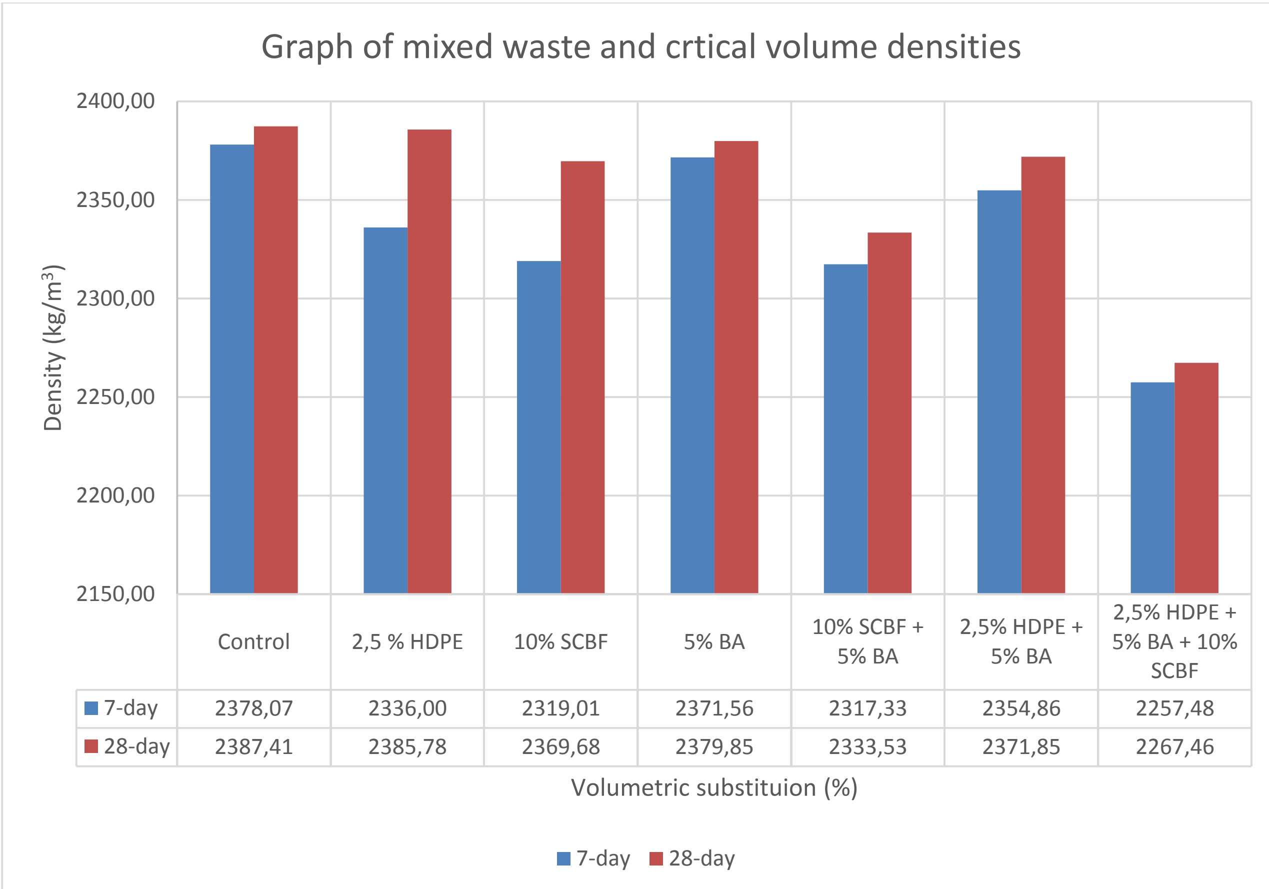


Figure 4-10 Density of mixed waste and critical volume substitution mixes at 7 and 28 days

The addition of BA to the individual waste samples decreased the density of the mixes by 1.5% for SCBF and 0.58% for HDPE at 28 days. The highest reduction in density of 5% relative to the conventional mix for both 7 and 28 days, was noticed when all three wastes where mixed together. The HDPE at critical volume showed the highest density of the mixed and critical volume specimens because it had the least volumetric substitution and behaved similar to the control mix at 28 days. The BA at critical volume also behaved similar to the control mix indicating that, at 5% BA substitution, the effect of reduced C-S-H forming cement is negligible. HDPE increased in density at 7days, when comparing the individual waste mix with the HDPE mix combined with ash, but the minimal change of 0.8% could be attributed to variations in cement composition.

The findings on the HDPE mixes confirmed findings by Al-Manaseer & Dalal (1997) and expanded on their findings by indicating that the decrease in density also applied to low–volume substitutions, but to a lesser extent than high-volume substitutions. As shown in Figure 4-10, the effect of adding ash to another waste also generally reduced the density of the concrete mix further, possibly because of a combination of individual material properties and poor bond interface which introduced additional voids to the mix.

The effect on concrete density due to aggregate variations was more significant than cement substitution. This supported the idea that aggregates have a greater influence on concrete density as they constitute 65-75% (MAST, 2014) of the concrete mix. It was also noticed that the effect on concrete density due to the degree of volumetric substitution and the material type had a greater effect than the effect of hydration from 7 to 28 days. The impacts on the viability of waste mixes regarding density results alone were inconsequential, because density requirements are based on the application. The changes in density relative to the conventional mix using waste materials were also minimal in this study (0.03% to 10%).

The effect of waste materials on strength (compressive, flexural and splitting) will be investigated in section 4.6 and from that, comparisons can be drawn with density and workability behaviour.

4.6. Hardened concrete strength properties of waste concrete

Strength, durability and workability are fundamental properties of concrete that have an influence on its viability. This section discusses the results of the waste mixes in terms of compressive strength, flexural strength and splitting strength.

4.6.1. Compressive cube strength (f_{cu})

Compressive strength is one of the primary specification properties for designers because if the target strength is not met, the concrete mix is not fit for purpose. It is for this reason that the optimum/critical volumetric substitutions used in this study were based on the results of the individual waste compressive strength tests. The optimum/critical volumetric substitutions were the maximum substitutions prior to a significant decline in compressive strength.

This study did not aim to create a specification for waste concrete to achieve a pre-determined target strength, but rather investigated the potential viability of wastes in concrete by comparing the effects on the conventional mix. Therefore in saying this, if a mix did not meet the target strength, it could not be stated as being viable for structural applications due to the fact it did not meet a predetermined strength requirement. However, if it did not meet the target strength but still gave an adequate strength reading of above 30 MPa (general structural applications), then there was still potential for its use in structural applications pending further empirical research. Examples of such research would be a repeatable set of guidelines to achieve a predetermined target strength using a waste mix, or the development of a W/C ratio–strength relationship graph related to waste concrete.

In South Africa, cube strength is used as stated in section 2.3.5.1. The average cube strength results from the tests on waste mixes at 7 and 28 days carried out as per section 3.6.1, are shown in Table 4-10 (refer to Appendix E for full data).

Table 4-10: Table of 7 and 28 day compressive strengths

Average compressive strength (MPa)							
Control mix							
7 Day	**28 Day**	**HDPE**		**SCBF**		**BA**	
25,32	42,10	**7 Day**	**28 Day**	**7 Day**	**28 Day**	**7 Day**	**28 Day**
2,50%		18,64	29,95	18,94	31,64	22,10	30,55
5%		17,03	28,01	19,40	31,72	22,09	30,39
10%		16,86	27,98	20,23	32,16	15,85	23,96
20%		16,76	27,22	13,55	23,53	13,86	19,33
40%		16,20	21,30	16,26	22,62	7,49	10,20

4.6.1.1. **Compressive strength of SCBF**

Figure 4-11 shows the 7 and 28 day compressive cube strength (f_{cu}) results for the SCBF specimens. The conventional mix (control) results are stated as 0% volumetric substitution.

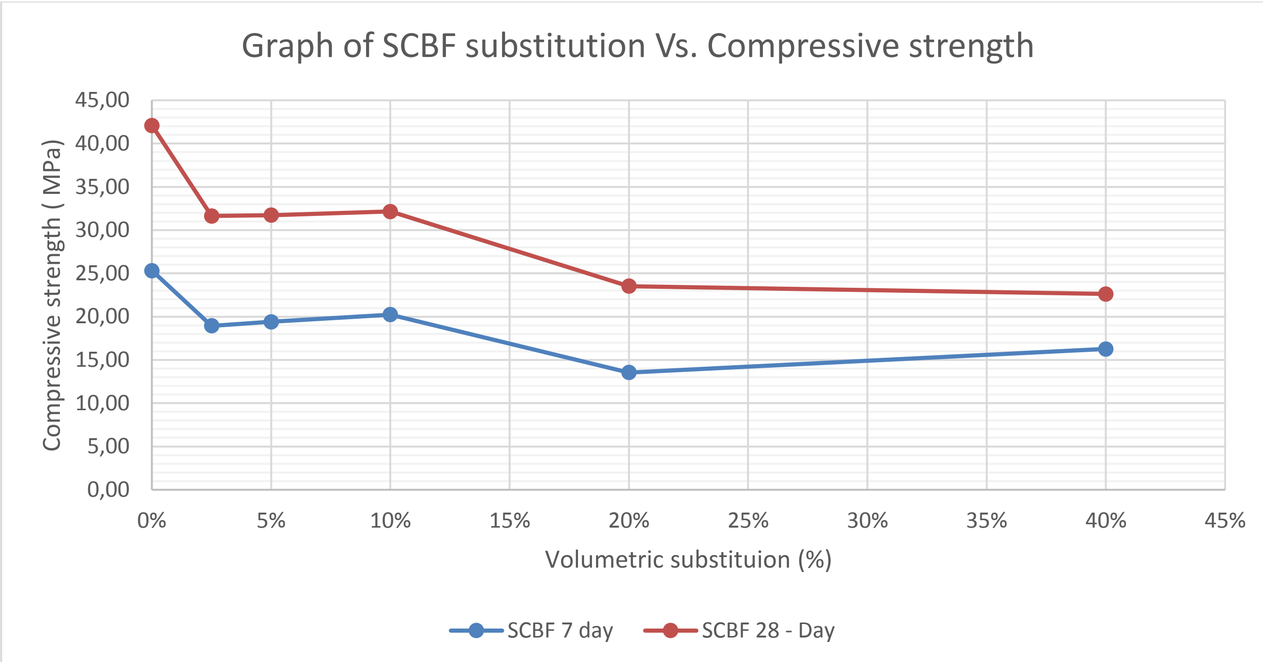


Figure 4-11: Graph of 7 and 28 day compressive strengths for SCBF mixes

The 7-day and 28 day compressive strength behaviour showed similar patterns of strength fluctuations with regards to variations in percentage substitutions. For substitutions between 2.5% and 10% at 28 days, compressive strength varied by not more than 1.4% between consecutive substitutions. This indicated that within this range of volumetric substitutions, the effect of SCBF on compressive strength was negligible and variations in compressive strength which yielded the unusual behaviour that created a ”peak” value at 10%, was probably due to variations in cement composition. The 10% volumetric substitution yielded the best results relative to the other volumetric substitutions, of 20.23 MPa and 32.16 MPa for 7 and 28 days respectively, although this was still 25% less than the conventional mix. This meant that for this study, SCBF at natural moisture state could not be used for structural applications as it did not meet the target strength of 35 MPa. However, considering a strength of 32.16 MPa was achieved, there is future potential for it to be used for structural purposes with further research into developing a specification guideline for SCBF waste concrete mix design.

The compressive strength at 40% volumetric substitution was 29% less than at the lowest volume substitution of 2,5% .The steep decrease in compressive strength shown above 10% volumetric substitution in Figure 4-11, confirms findings by Racines & Pama (1978), who showed that at higher volume substitutions of 20% and 30% volumetric substitution, there are sharp decreases in compressive strength.

In terms of the rate strength development for substitution of 2,5% to 20%, Figure 4-11 shows an almost parallel relationship between 7 and 28 day results indicating a constant rate of strength development of approximately 60% of 28 day strength being realised at 7 days. This behaviour was similar to that exhibited by the conventional mix .The volumetric substitution at 40% did show a slightly higher rate of strength development but this is probably due to the fact that the 28 day strength was significantly reduced due to the high volume of SCBF, balling or inhibiting hydration, thus resulting in a smaller gap between 7 and 28 day strengths. Although the rate of strength development in terms of percentage of 28 day strength realised at 7 days was not adversely affected by the possible sucrose in SCBF when compared to the conventional mix, the overall strength declined compared to the conventional mix. This meant that the possible residual sucrose may have retarded the initial setting of concrete and hence lower strengths were realised. This statement can be supported by the findings on the retarding effects of sucrose done by Bilba, *et al*.(2003). It can therefore be said, that the effect of sucrose was not sufficient to adversely affect the rate of strength development between 7 and 28 days which showed similar rates of strength gains to the conventional mix. However, lower strengths were obtained due to the initial retardation of the mix.

In relation to workability which decreased, it can be said that the effect of moisture absorption on the mix did not have a significant impact on the W/C ratio as the compressive strengths declined relative to the conventional mix. Therefore, one can say that the decline in workability was more attributed to the length of fibres than the absorption properties of the SCBF seeing as the initial moisture content was also quite high at 63%, therefore resulting in a lower absorption potential as per section 2.3.10.

There was a directly proportional relationship between compressive strength and the reduced density mentioned in section 4.5.1. The reasoning however, would need to be confirmed with durability testing in section 4.9, to establish if density decreased due to increased voids or just the lighter weight of fibres. If density decreased due to increased voids this further explains the decline in compressive strength.

4.6.1.2. **Compressive strength of HDPE**

Figure 4-12 shows the 7 and 28 day compressive cube strength (f_{cu}) results for the HDPE specimens. The conventional mix (control) results are stated as 0% volumetric substitution.

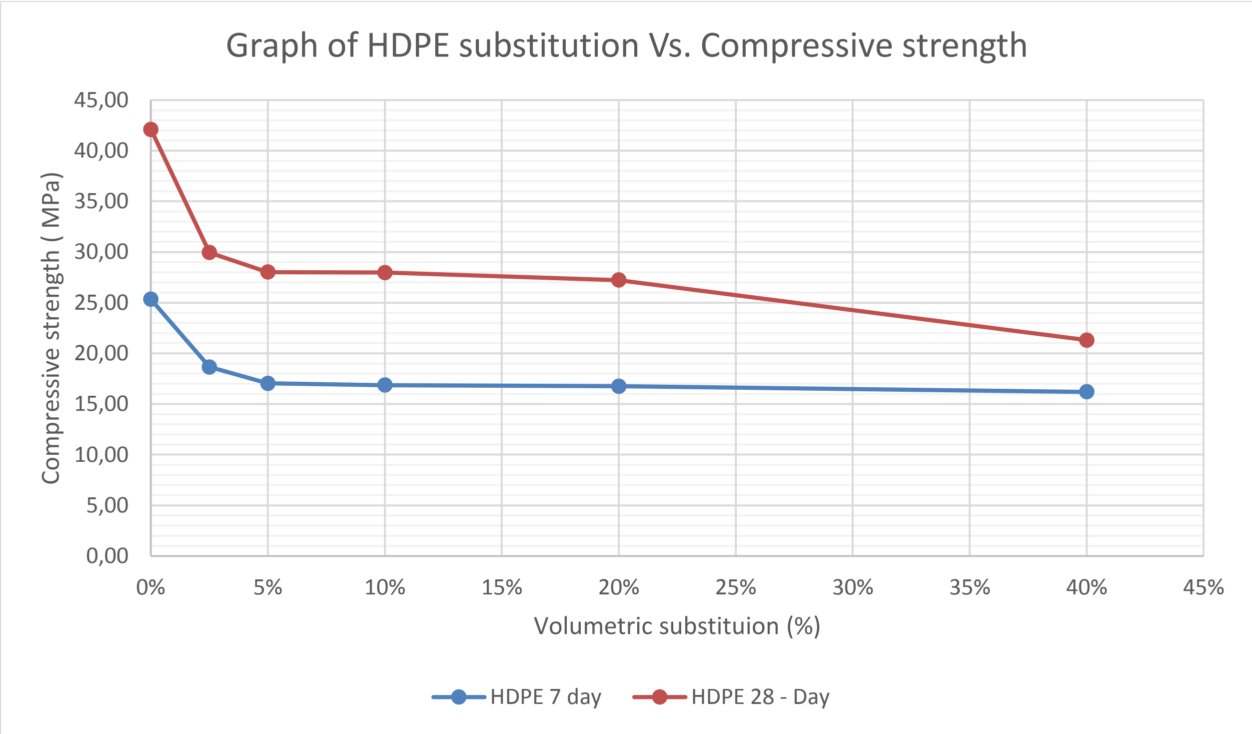


Figure 4-12: Graph of 7 and 28 day compressive strengths for HDPE mixes

Compressive strength decreased for all HDPE mixes relative to the control mix. The 7-day and 28 day compressive strengths of HDPE followed a similar trend with regards to changes in compressive strength for varying percentage substitutions. A steep decrease in compressive strength was noticed at both 7 and 28 day strengths with the addition of 2.5% HDPE to the conventional mix. The 2.5% volumetric substitution yielded a 26% decrease in 7 day strength to 18,64 MPa and 29% decrease in 28 day strength to 29,95 MPa relative to the conventional (control) mix. The 2,5% volumetric substitution was chosen as the critical/optimum substitution because it resulted in the least loss in strength relative to the other proportions of HDPE. The 40% substitution yielded the highest decrease in strength of 49% from the conventional mix to 21.3 MPa at 28 days and 36% from the conventional mix to 16.2 MPa at 7 days. The variation in compressive strength between the low-volume

(2,5%) and high-volume (40%) substitution was 29%. The decrease in compressive strength supports findings by Rahman *et al.* (2012), and to a lesser extent Suganthy, *et al.* (2013) and Al-Manaseer & Dalal (1997), who showed decreases in compressive strength at high volume substitutions of HDPE but with higher decreases in strength than those found in this study. Either way, the fact that there was a significant decrease at higher volumes emphasize that HDPE is best used at low–volume substitutions as per Ferreira, *et al* (2012), where it yields the least reduction in compressive strength.

The rate of strength development remained relatively constant between 2,5% and 20% substitutions, with approximately 61% of 28 day strength being realised after 7 days. This was similar to the conventional mix. This indicated that the HDPE pellets did not adversely affect the "rate" of strength development. However, the introduction of pellets did affect the overall strength of the mix due to the smooth, hydrophobic surface of the pellets that had poor bond characteristics with the cement mix. This was evident in the inverse relationship with workability which increased with greater proportions of HDPE due to the increased fluidity around the HDPE pellets and lack of bonding with the cement matrix. These findings supported those by Choi, *et al.* (2009) and Ahmad, et al. (2008).

Compressive strength was found to be directly proportional to density with increasing HDPE content in the mix. The effect of the weakened transition zone due to poor bond characteristics with the HDPE pellets may have increased porosity and contributed to the lower density and strength. This will be confirmed from the durability results (see section 4.9).

The use of HDPE at natural moisture state could not be used for structural applications as the target strength was not met, however a compressive strength of 29.95 MPa was achieved. This was technically medium-high strength concrete as stated in section 2.3.5.1.

4.6.1.3. **Compressive strength of BA**

Figure 4-13 shows the 7 and 28 day compressive cube strength (f_{cu}) results for the BA specimens. The conventional mix (control) results are stated as 0% volumetric substitution.

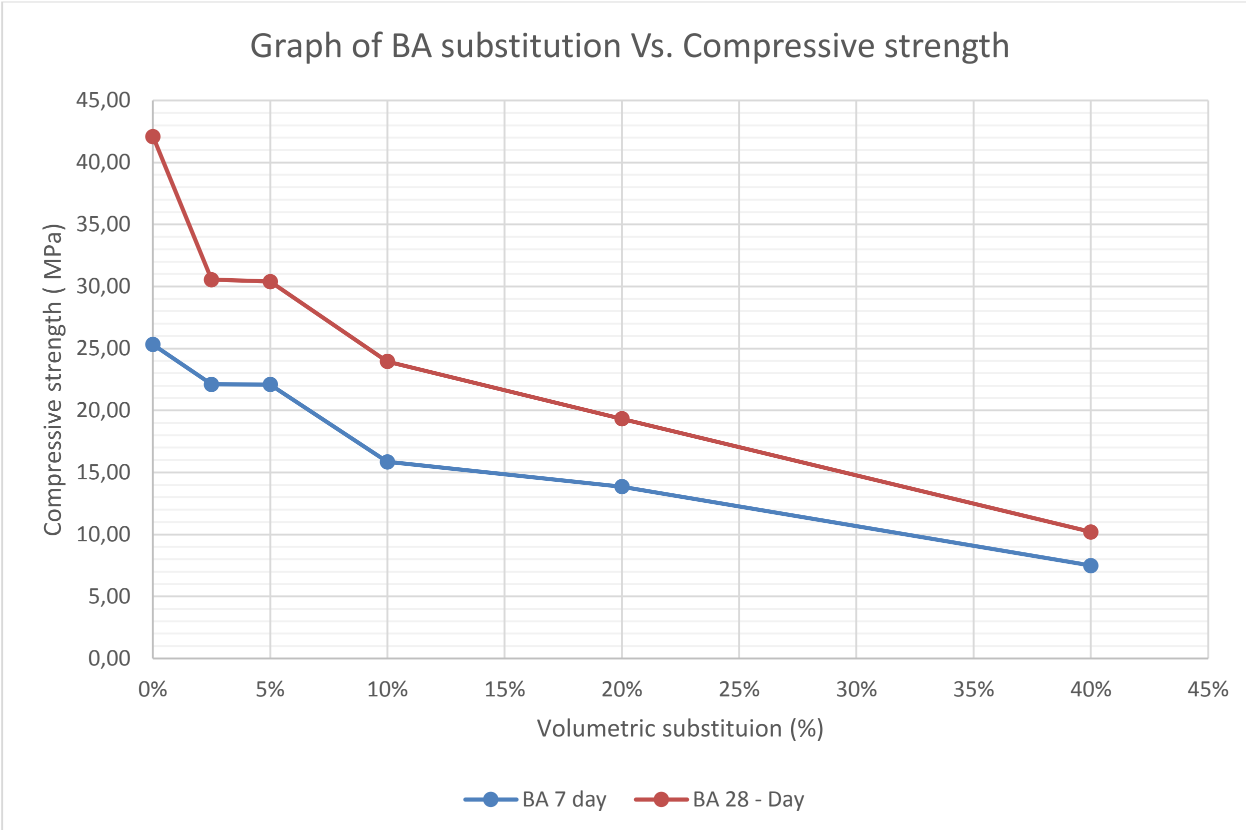


Figure 4-13: Graph of 7 and 28 day compressive strengths for BA mixes

The BA samples showed a general decrease in compressive strength with increasing volumetric substitutions. This was shown by the loss in 28 day compressive strength of 67% from the lowest volume (2,5%) to the highest volume (40%) substitution. Passing the particles through a sieve did not improve reactivity and compressive strength, which was theorised by Cheriaf, *et al* (1999) to improve due to the finer particle size.

For both the 7-Day and 28-day strengths, the strength behaviour between 2.5% and 5% was similar. The 2,5% sample yielded 7 and 28 day strengths of 22.1 MPa and 30.55 MPa respectively compared to the 5% substitution which yielded 22.09 MPa and 30.39 MPa after 7 and 28 days respectively. The minimal differences (0.04% - 7 days and 0.52% - 28 days) between these substitutions could have been attributed to variations in cement composition. It was decided that because the difference in strength between 2, 5 % and 5 % was minimal, the 5% substitution would be selected as the optimum/critical volumetric substitution as it reduced the carbon footprint of the concrete mix further by increasing the reduction in cement content.

In comparison to the conventional mix, at the selected optimum/critical volume 5% substitution, there was a 13% decrease in 7-day day-strength and a 27% reduction in 28-day strength. The difference in reduction from 7 to 28 days could be because at 7 days the hydration reaction was still relativity young with mainly tricalcium silicate reacting, and the impact of having less cement particles to partake in the hydration reaction was less evident. However, as the hydration reaction proceeded and more compounds such as dicalcium silicate became active, the lower cement content of the BA mix relative to the conventional mix was noticeable.

The near parallel line between 7 and 28 days showed a similar compressive strength response to volumetric substitution changes in BA for both curing ages. The minimum strengths of 7.59 MPa (7-day) and 10.2 MPa (28-day) were both reached at 40% substitution and were 70% and 76% less than the conventional mix respectively.

In relation to the density results for the BA samples, the reductions in density were minimal (max of 3%) relative to the conventional mix and hence it is reiterated that the reason for the loss in strength was due to the lower cement content and unreactive BA.

The general decrease in compressive strength with increasing BA content was due to the pozzolanic reaction not taking place within 28 day tests period and hence not providing any C-S-H via the pozzolanic reaction, which the cement would have produced. This reasoning is supported by past research carried out by Kurama & Kaya (2007) who showed that bottom ash in concrete only showed significant improvements for curing ages beyond 28 days.

Based on the compressive strength results of BA samples, concrete with coal bottom ash performed better in low-volume substitutions compared to high volume substitutions. However, all mixes with BA at natural moisture state could not be used for structural purposes because the target strength was not met based on findings from this study.

4.6.1.4. **Mixed combinations**

Figure 4-14 shows the 7 and 28 day compressive cube strengths for the critical volume individual waste mixes and mixed waste specimens. The critical volumes of 2.5% for HDPE, 10% for SCBF and 5% for BA, were based on the volumetric substitutions that yielded the optimum/critical volume compressive strengths for the individual waste mixes (See section

4.6.1). As stated in section 3.1, the critical volumetric substitutions were also used for the mixed waste specimens.

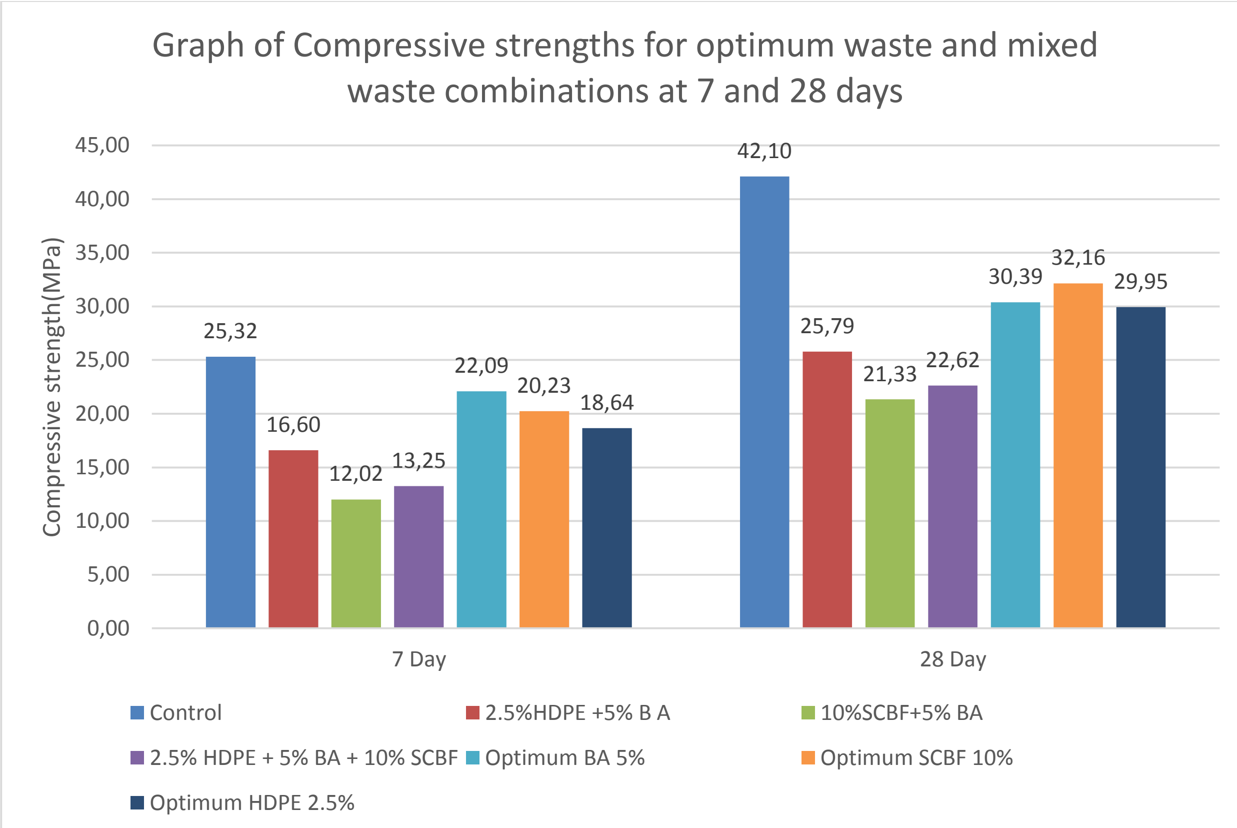


Figure 4-14: Graph of mixed waste and optimum waste substitution compressive strengths at 7 and 28 days

In reference to Figure 4-14, the 7-day and 28-day compressive strengths show that the mixed combinations do not perform as well as the concrete with individual optimum/critical volume material proportions. Therefore, findings by Ahmad, *et al*. (2008) on the use of HDPE with ash and Onésippe, *et al*. (2010) who researched SCBF and ash, did not correlate with findings from this study. Bottom ash varies in composition and it is possible that the ash used by Ahmad, *et al.* (2008) and Onésippe, et al, (2010) was more reactive than that which was used for this study.

The 2.5% HDPE mixed with 5% BA performed the best out of the three mixed combinations attaining a 7-day strength of 16.6 MPa and 28 day strength of 25.7 MPa. This was a 34% and 39% decrease from the control strength respectively. However, all three optimum/critical volume mix percentages showed higher 7-day and 28-day strengths than the mixed waste substitutions. The optimum/critical volume independent mixes also achieved larger percentages of 28-day strengths after 7 days, which indicates quicker strength generation compared to the mixed waste specimens. The possible reason for this was that BA did not form a pozzolanic reaction before 28 days. This coupled with the reduction in tricalcium silicate (early strength gain compound) due to cement substitution, resulted in a slower strength gain for the mix.

In terms of compressive strength it can be said that the proposed waste materials are best used independently and not in conjunction with each other. However, neither the optimum/critical volume nor mixed waste specimens reached the target strength of 35 MPa and were all less than the control/conventional mix sample. Therefore, none of the mixes using waste aggregates at natural moisture state would be viable for structural applications based on findings from this study. However, because the compressive strengths were generally around 30 MPa (general structural use), with further research into developing a waste concrete design specification, there was still the future possibility for medium to high strength applications.

4.6.2. Flexural strength

Flexural strength is a measure of tensile strength under a bending load. Beams were tested in flexure for this study as per section 3.6.2. The average results are shown in Table 4-11 (see Appendix E for full data).

Table 4-11: Table of 7 and 28 day flexural strengths

Average flexural strength (MPa)							
Control mix							
7 Day	**28 Day**	**HDPE**		**SCBF**		**BA**	
4,27	5,43	**7 Day**	**28 Day**	**7 Day**	**28 Day**	**7 Day**	**28 Day**
2,50%		3,95	5,06	3,92	4,68	3,26	4,59
5%		3,45	4,86	3,92	4,55	3,21	4,38
10%		3,35	4,57	3,67	4,43	3,18	3,92
20%		3,23	4,47	3,29	3,77	2,72	3,95
40%		2,87	3,98	3,18	3,65	2,68	2,93

4.6.2.1. Flexural strength of SCBF

Figure 4-15 shows the 7 and 28 day flexural strength results for the SCBF specimens. The conventional mix (control) results are stated as 0% volumetric substitution.

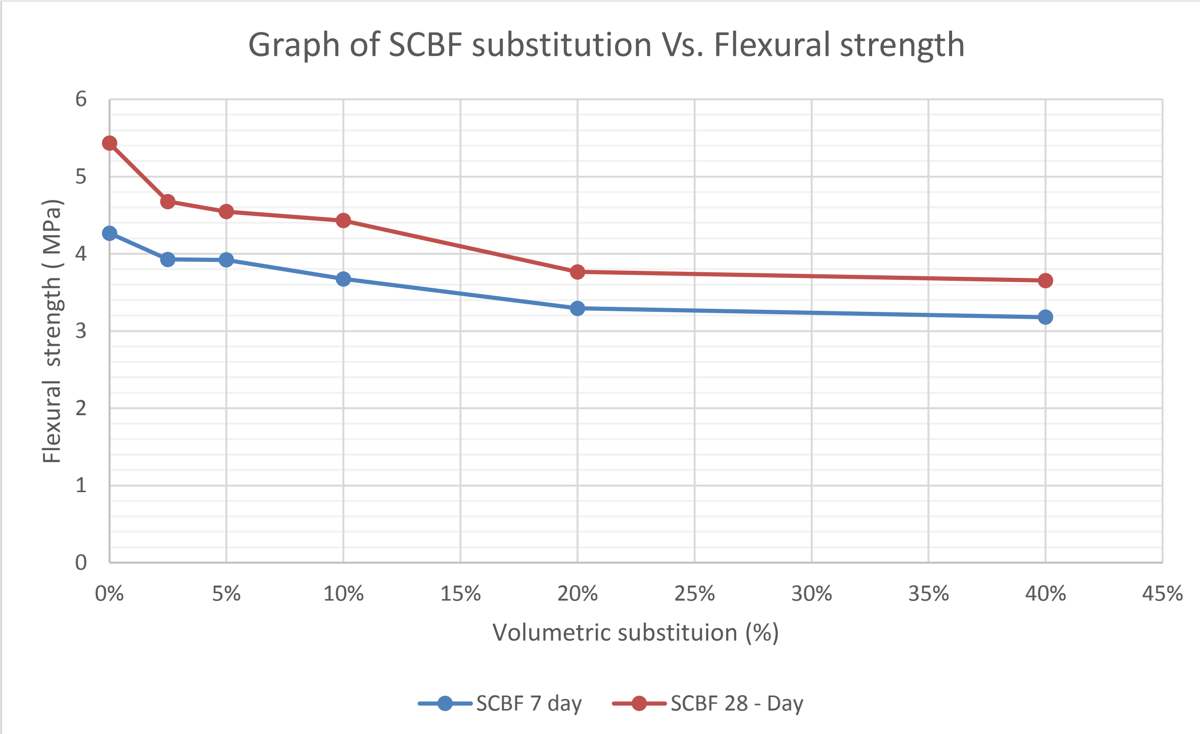


Figure 4-15: Graph of 7 and 28 day SCBF flexural strength results

All mixes showed a decline in flexural strength relative to the conventional mix with substitution's beyond 5% showing more significant decreases. This indicated that the low-modulus and relatively low tensile strength of SCBF stated in Table 2-4 on page 57, did not offer much resistance to bending. The softening of SCBF under alkaline conditions as per Daniel (2002), may have also contributed to the reduced stiffness and hence lowered the concrete flexural strength. This supports the concept stated in section 2.3.5.2, that generally, for a fibre to perform well as a crack arrestor, it needs to have a high modulus of elasticity and tensile strength. The highest performing percentage substitution was 2,5% which yielded flexural strengths of 3.97 MPa (7 day) and 4,68 MPa (28 day). These were however 8.04% and 13.9% reductions in 7 and 28 day flexural strengths respectively, relative to the conventional mix. The highest reductions in flexural strength of 25% and 33%, for 7 and 28 days respectively were at 40% volumetric substitution.

The 7 day flexural strength showed a similar behaviour to the 28 day strength in terms of flexural strength variations with increasing volumetric substitutions, indicated by the near parallel flexural strength plot lines. The difference between the 28 day flexural strength at 2,5% substitution and 40% substitution was 22%. In general, approximately 83% of flexural strength was realised at 7days between 2,5% and 10% volumetric substitution. This rate increased to 87% between 20% and 40% substitutions probably due to larger proportions of SCBF which softened after 28 days and lowered the gap with 7 day strength.

The decrease in tensile strength even at low volume substitutions was in contrast to findings by Sivarja *et al*. (2010), who showed improvements to tensile strength at low-volume substitutions. Sivarja *et al*. did however investigate a volumetric substitution of 1.5% and this was lower than the 2.5% tested in this study, therefore, the 1.5% used by Sivarja *et al*. Ramirez-Coretti (1992) also showed increases in tensile strength between 3-7%, albeit minimal. Findings in this study also were in contrast to findings by Wang, *et al*. (2000), however, the reason for this is that they based their study on short natural fibre whereas the SCBF used in this study were a mix of short and long fibres. Therefore, one can state that SCBF is best used at low-volume substitutions. However, the behaviour varies depending on the properties of the bagasse such as the average fibre length and moisture content.

When comparing the 28 day SCBF flexural strength and compressive strength results, flexural strength was on average 15% of compressive strength, which is within the typical range stated in section 2.3.5.2 on page 108. Therefore, an approximation relationship shown below can be made:

Flexural strength (SCBF) = 0.15 f_{cu}

4.6.2.2. **Flexural strength of HDPE**

Figure 4-16 shows the 7 and 28 day flexural strength results for the HDPE specimens. The conventional mix (control) results are stated as 0% volumetric substitution.

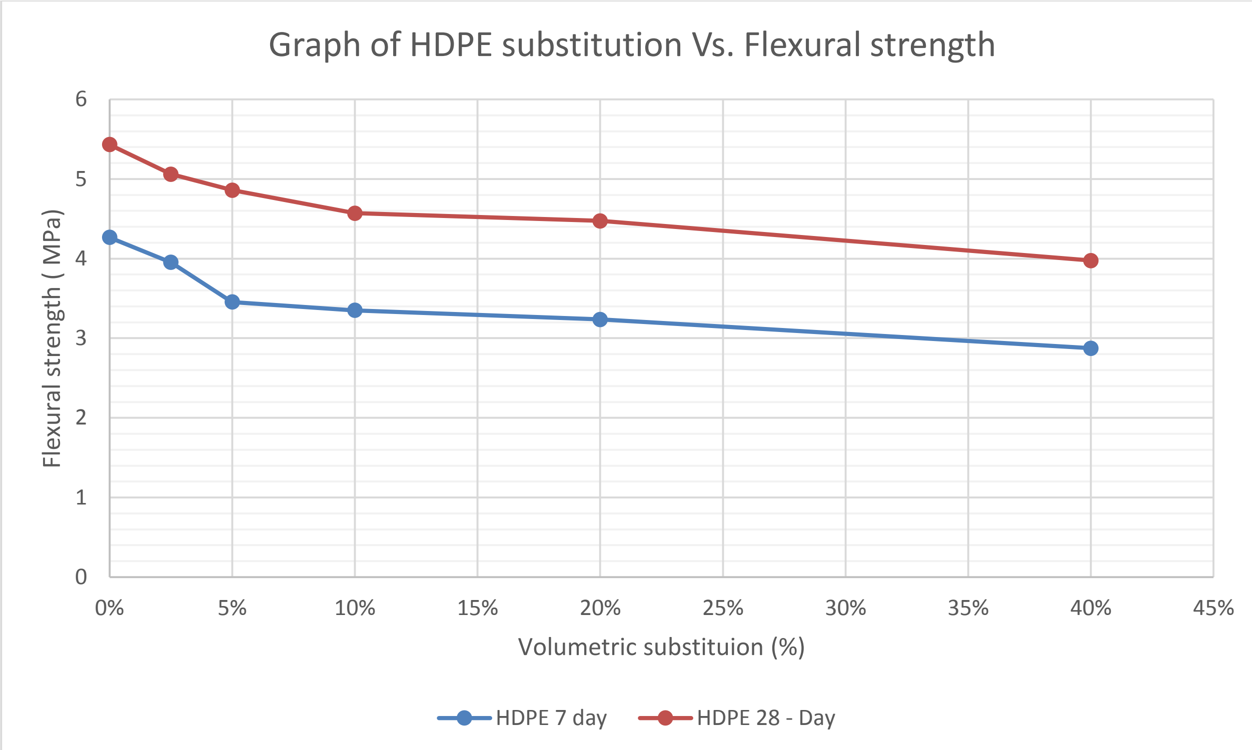


Figure 4-16: Graph of 7 and 28 day HDPE flexural strength results

The flexural strength of HDPE showed similar trends for 7 and 28 day strengths with regards to changes in flexural strength for varying percentage substitutions. All mixes showed a decrease in flexural strength with the least decline for both 7 and 28 days being noticed at 2.5 % aggregate substitution. These were 3.95 MPa and 5.06 MPa which were a 7.3% and 6.8% decrease in 7 and 28 day flexural strength respectively, compared to the conventional concrete mix. The lowest 7 day and 28 day flexural strengths were 2.87 MPa and 3.98 MPa respectively, at 40% substitution. The difference in flexural strength from 2,5% to 40% volumetric substitution was 21%

The percentage of 28 day strengths realised at 7 days was on average 75%. The decline in flexural strength for all HDPE mixes relative to the conventional mix, indicated that the HDPE pellets did not transfer tensile load effectively. This was possibly due the poor bond interface between cement creating weak points for cracks to propagate, thus reducing the tensile strength with increasing volumetric substitution. These findings confirm those of Ismail & AL-Hashmi (2007) who showed that with high volume substitutions of 10%, 15% and 20% there was a drop in tensile strength. This study expanded on the findings of Ismail & AL-Hashmi (2007) by showing that the decline in tensile strength also applied to lower volume substitutions of 2,5% and 5%.

When comparing the 28 day HDPE flexural strength and compressive strength results, flexural strength was on average 17% of compressive strength. This was within the typical range stated in section 2.3.5.2 on page 108 . Therefore, an approximation relationship shown below could be made:

Flexural strength (HDPE) = 0.17 f_{cu}

4.6.2.3. **Flexural strength of BA**

Figure 4-17 shows the 7 and 28 day flexural strength results for the BA specimens. The conventional mix (control) results are stated as 0% volumetric substitution.

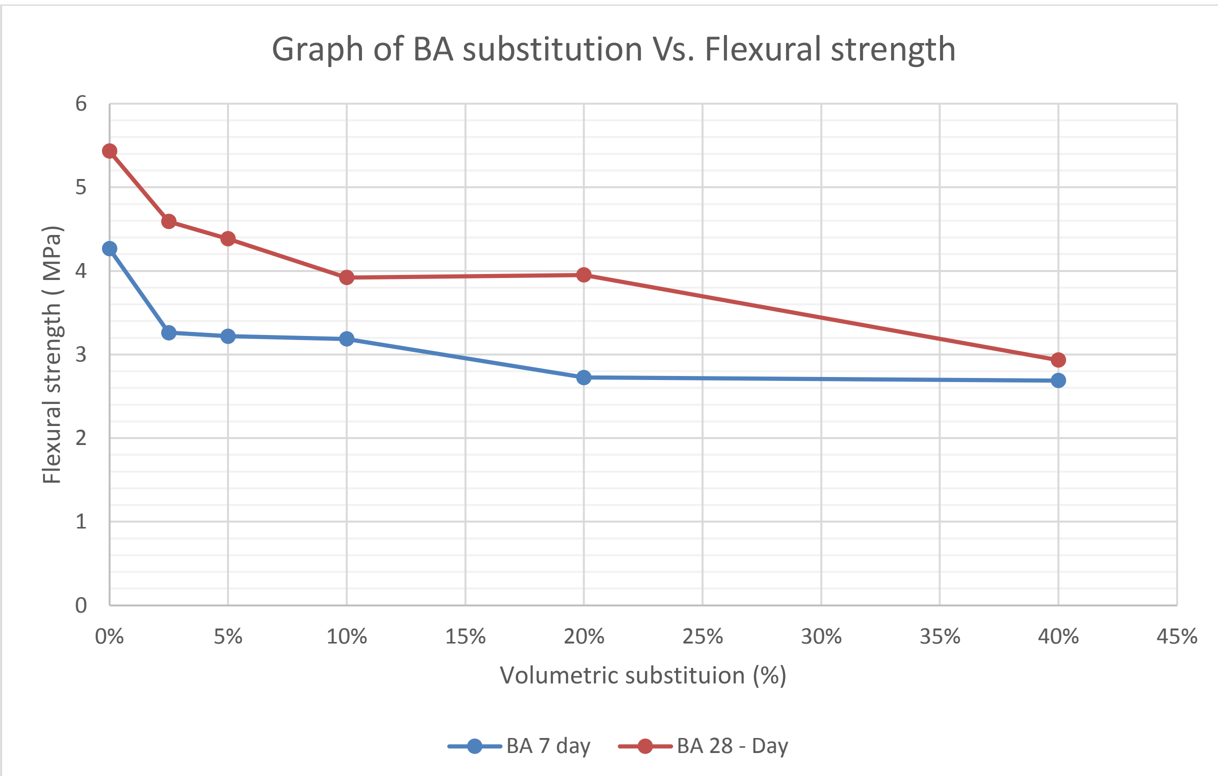


Figure 4-17: Graph of 7 and 28 day BA flexural strength results

The 28-day flexural strength showed an inverse relationship to the degree of cement substitution. It is due to this behaviour that the highest 7 day and 28-day BA mix flexural strengths were reached at 2.5% substitution. The flexural strengths of 3.26 MPa and 4.59 MPa attained at this substitution were a 24% and 16% decrease from the control mix respectively. The lowest 28 day flexural strength of 2.93 MPa, which was a 46% drop in flexural strength relative to the control mix, was realised at 40% volumetric substitution.

At 40% substitution, 91.5 % of the mix's 28-day strength was developed after 7 days. This was due to the low 28 day strength at the 40% substitution because BA had little effect on improving tensile properties. It was also interesting to note that between 10% and 20% volumetric substitution there was only a minimal change in flexural strength.

When compared to the control sample, the decreased flexural strength shown by all mixes supported findings by Aggarwal *et al*. (2007), who stated that the tensile strength decreased, relative to a conventional mix, regardless of percentage substitution of BA or curing age. Kurama & Kaya (2007) however stated that at substitutions of 5%, flexure improved by 3% and 2% after 7 and 28 days respectively. The changes noticed in this study were relatively small and could have been due to variations in cement composition used.

When comparing the 28 day BA flexural strengths and compressive strength results, excluding the 20% and 40% substitutions which were outliers, flexural strength was on average 15% of compressive strength. This was within the typical range stated in section 2.3.5.2 on page 108. Therefore, an approximation relationship shown below can be made:

Flexural strength (HDPE) = 0.15 f_{cu} for substitutions between 2.5% and 10%.

4.6.2.4. **Mixed Combinations**

Figure 4-18 illustrates the 7 and 28 day flexural strengths for the critical volume individual waste mixes and mixed waste specimens. The critical volumes of 2.5% for HDPE, 10% for SCBF and 5% for BA, were based on the volumetric substitutions that yielded the optimum/critical volume compressive strengths for the individual waste mixes (See section 4.6.1). As stated in section 3.1, the critical volumetric substitutions were also used for the mixed waste specimens.

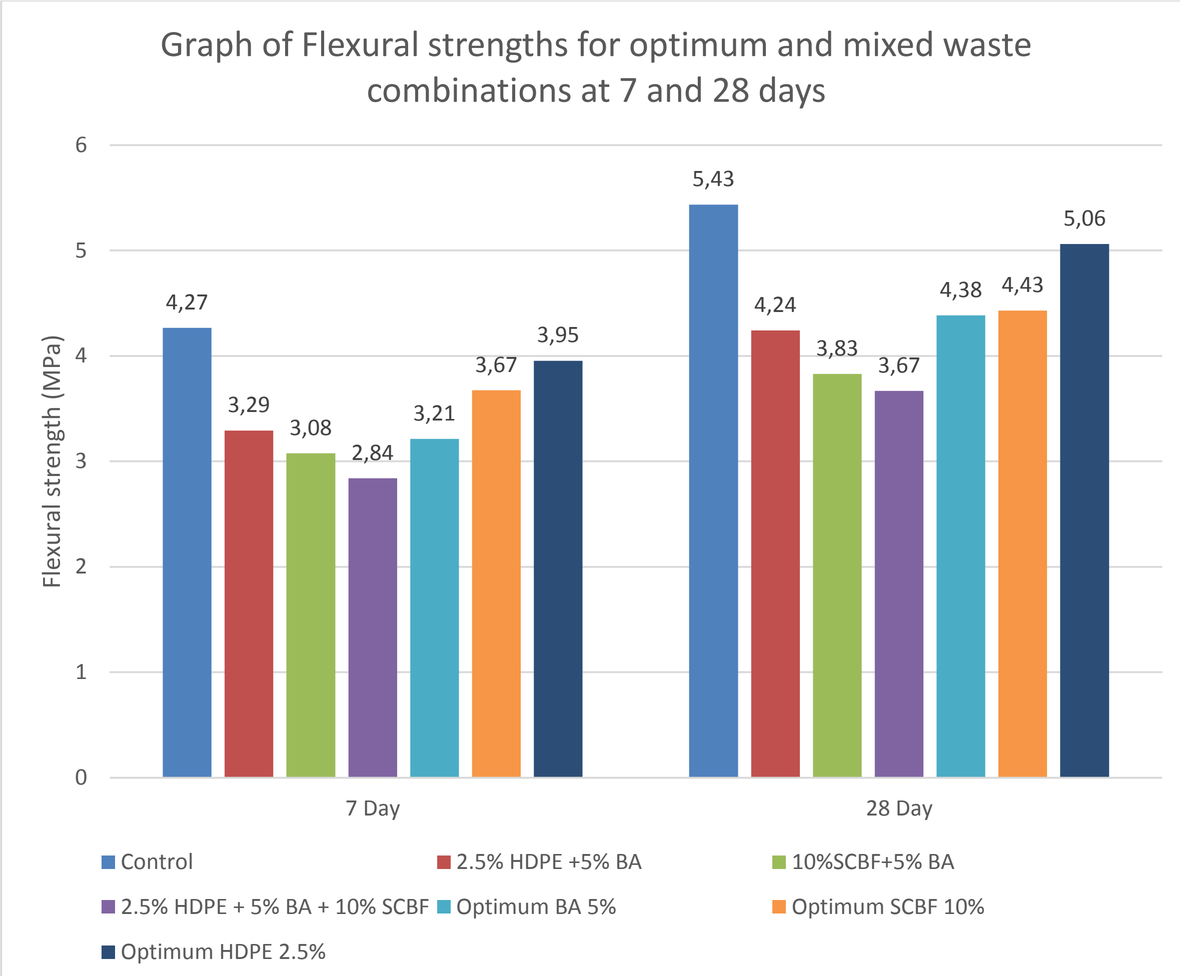


Figure 4-18: Flexural strengths of mixed waste and optimum mix combinations at 7 and 28 days

The flexural strength of the mixed waste combinations decreased in both 7-day and 28-day flexural strengths when compared to the control sample. This was because the pozzolanic reaction did not take place before 28 days and hence the BA did not have any positive effect on the concrete mixes. SCBF with the introduction of BA, showed a reduction in 28 day flexural strength of 13.5% relative to the individual SCBF mix. The introduction of BA also reduced the 28 day flexural strength of the individual HDPE mix by 16%. The mix with all the wastes however performed the worst and showed a decline of 32.5% in 28 day flexural strength relative to the control mix.

4.6.3. Splitting strength

Splitting strength is a measure of tensile strength under direct compression. Cubes were tested at 28 days for splitting strength in this study in accordance with section 3.6.3 and the average results are shown in Table 4-12 (refer to Appendix E for full data).

Table 4-12 Table of individual waste mix splitting strengths

Control	Average splitting strength (MPa)		
3,38	**HDPE**	**SCBF**	**BA**
2,50%	3,04	3,90	2,83
5%	2,74	3,90	2,91
10%	2,74	3,43	2,85
20%	2,60	2,59	2,46
40%	2,60	2,33	1,06

4.6.3.1. Splitting strength of Individual mixes

Figure 4-19 illustrates the changes in splitting strength for the waste mixes at 28 days. The control mix was shown as 0% substitution,

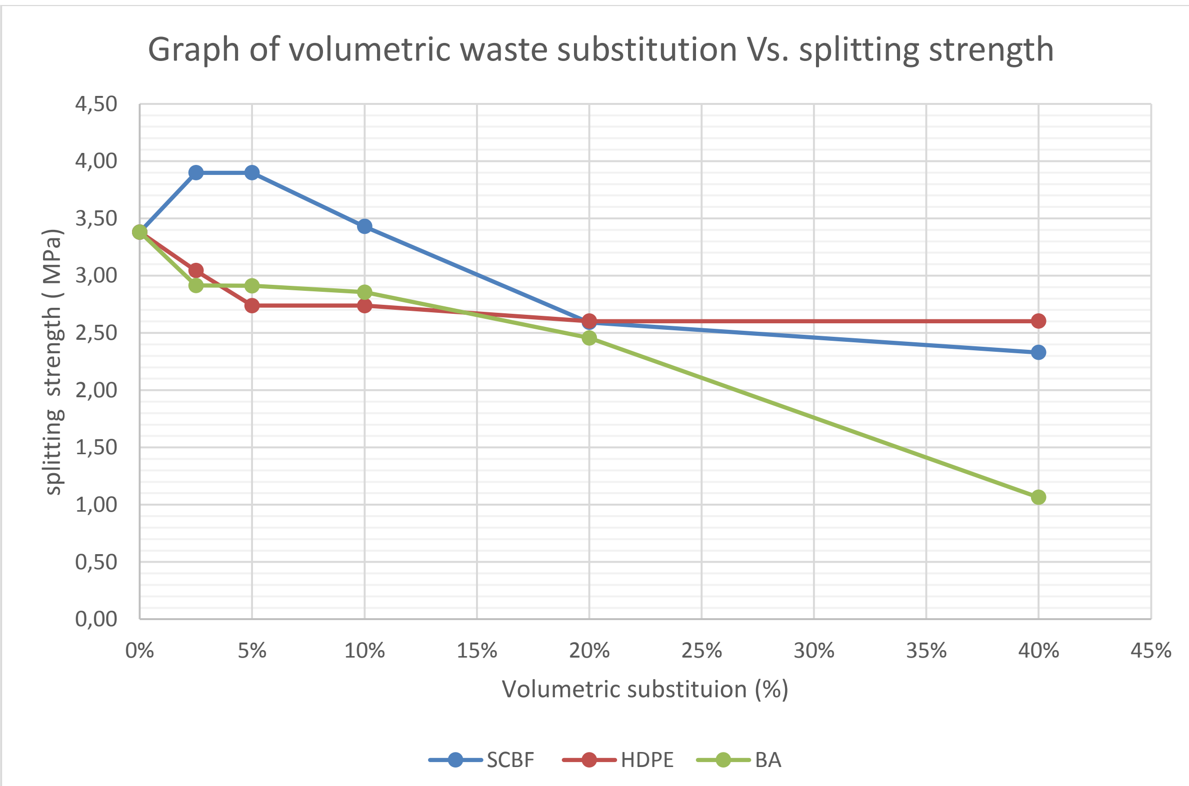


Figure 4-19 : Graph of Splitting strength vs. volumetric waste substitution

SCBF

The splitting strength of the SCBF mix showed an increase relative to the control, to 3.9 MPa at 2.5% and this remained constant until 5% substitution. This indicated that splitting strength was either not sensitive to changes in volumetric substitutions of SCBF between

2,5% and 5% or the SCBF in the 5% mix was not uniformly dispersed and hence did not provide the maximum benefit in terms of improvements to splitting strength. The peak value at 5% volumetric substitution yielded an increase of 15% from the conventional mix, thus showing that SBCF can benefit in terms of improving shear resistance. The increase in splitting strength expanded on past research carried out by Sivarja *et al*. (2010), whose findings showed an increase in splitting strength by 37% for a lower volumetric substitution of 1.5%.

Splitting strength was also slightly increased at 10% substitution by 1.6% relative to the control, however, from 20-40% substitutions, there were decreases of 23% and 31% respectively relative to the conventional mix. The lowest strength of 2.33 MPa was found at 40% substitution and the difference in strength between the 2,5% and 40% substitutions was 40%. This indicated that beyond 20% the SCBF may have been susceptible to balling/clumping and hence did not perform well as a crack arrestor.

When splitting and flexural strengths were compared, it was found that on average, splitting strength equated to 70% of the flexural strength. The substitutions at 2,5% and 5% were considered as outliers.

HDPE

Splitting strength showed a gradual decrease with increases in percentage volume substitution, with the least reduction being noticed at the lowest volumetric substitution of 2.5%. The decrease in splitting strength relative to the control mix with increasing volumetric substitution was supported by past research carried out by Ferreira, *et al*. (2012), which showed decreases of 6,8% and 20% for 7,5% and 15% volumetric substitutions respectively. The reductions at 5% and 10% in this study were 36% less than the control mix, which was slightly more than the results of Ferreira, *et al*. (2012), however, this could be due to variations in BA properties.

The reasoning for the reductions in splitting strength was similar to that of flexure strength. The increased HDPE pellet content resulted in an increase in weak points in the mix due to poor bond characteristics. It is these points of weakness where cracks could propagate thus decreasing the splitting strength of the mix. The effect on splitting strength due to poor bond characterises had less of an impact than flexural strength because from 5-40% substitution splitting strength varied by only 5.1%. The least reduction was found at 2.5% substitution, which had a strength of 3.04 MPa and yielded a decrease of 10% compared to the conventional mix.

When the HDPE mix splitting strength and flexural strength were compared, it was found that on average, splitting strength equated to 61% of the flexural strength

Bottom Ash (BA)

The splitting strength of BA specimens showed a general decrease in strength as the percentage of volumetric substitution increased. Splitting strength varied by 2.82% between 2,5% and 5% volumetric substitution. This could have been attributed to variation in cement composition.

The least reduction in splitting strength of 2.91 MPa was achieved at 5% volumetric substitution, which was 13.8% less than the control specimen. The lowest BA mix splitting strength was 1.06 MPa at 40% substitution which was a 68,5% decrease relative to the control mix. The difference in strength between the 2,5% and 40% substitutions was 62.5%.

The decrease in splitting noticed for this study confirmed past research by Aggarwal *et al. (*2007), which stated that splitting strength decreases for all percentage of substitution for ages up to and including 28 days. This was mainly because the pozzolanic reaction had not taken place prior to 28 days of curing.

When the BA mix splitting and flexural strengths were compared, it was found that on average, splitting strength equated to 57% of the flexural strength.

Based on the data from this study, one can say that BA substitution does not yield any benefit in terms of 28-day splitting strength when compared to conventional concrete.

4.6.3.2. **Splitting strength of mixed combinations**

Figure 4-20 illustrates the splitting strength results for the mixed waste combinations compared to the optimum/critical volume waste (based on compressive results) and control mixes.

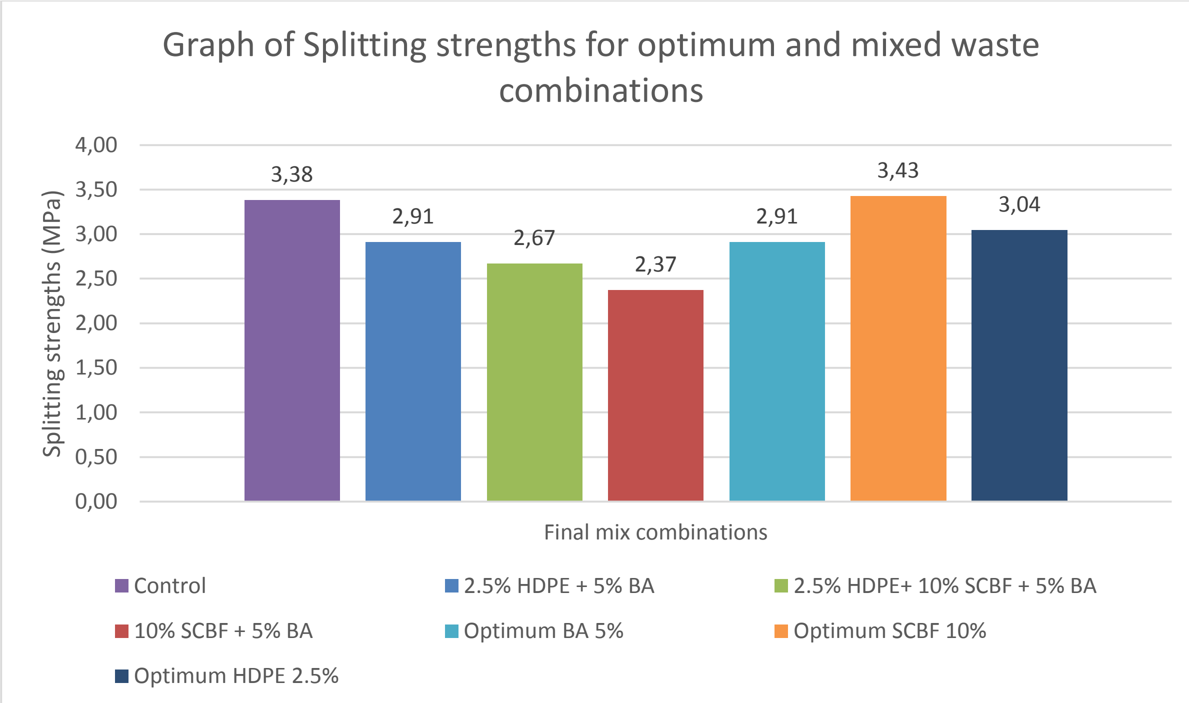


Figure 4-20: Graph of splitting strengths for mixed waste substitutions and optimum waste mixes

In terms of the splitting strengths, mix combinations with BA all showed reductions. The introduction of BA, deceased the splitting strength of the individual HDPE mix by 4%. The optimum/critical volume SCBF mix performed best on its own achieving an increase in strength of 1.4% over the control however, it performed the worst with BA added yielding a 30% decrease in strength compared to the control.

Based on the above it can be said that SCBF at the optimum/critical volume mix proportion without BA performs best in splitting strength from the waste mixes. The optimum/critical volume SCBF mix also improved in splitting strength relative to the control mix. If a reduction in cement was required for economic or environmental reasons, then HDPE would be the better waste aggregate to use with BA as it lost the least strength out of the three mixed waste combinations.

4.6.4. Summary of strength properties

Compressive strength

When assessing the three waste materials (HDPE, SCBF & BA) in compression, SCBF in general, had the highest compressive strengths of the three material and HDPE the lowest for low-volume substitutions. BA had the lowest strength at high volume substitutions due to the lack of cement to form C-S-H and lack of pozzolanic reaction taking place at 28 days. This had a larger negative impact on compressive strength than the hydrophobic nature of HDPE. The lack of pozzolanic reaction was also evident when BA was mixed with the individual wastes and resulted in decreases in compressive strength. The reasoning for the SCBF and HDPE strength results could be related to the absorption properties of the respective materials. SCBF may have decreased W/C due to the absorption of water from the mix thus increasing compressive strength, whereas HDPE had a hydrophobic nature which caused a film of water to form in the interfacial zone, thus increasing the W/C in the interfacial zone and decreasing compressive strength. This meant that out of the three waste materials SCBF was the most viable in terms of compressive strength.

When considering the varying proportions of waste material in the mix, all waste proportions for all three waste materials showed decreases in compressive strength when compared to the conventional mix. This confirmed findings by Okajima (1972) that stated, if the aggregate has a modulus of elasticity lower than the matrix, compressive strength is decreased due to lowered resistance to micro-cracking. Therefore the optimum/critical volumetric substitutions were not “peak” values where performance benefits were realised relative to the conventional mix before a decline, but rather, were the substitutions at which the least reductions in strength was yielded relative to the conventional mix. These optimum/critical volume percentage substitution based on compressive strength results were, 2.5% for HDPE, 5% for BA and 10% for SCBF, which yielded 28 day compressive strengths of 29.95 MPa, 30,39 MPa and 32.16 MPa respectively. Neither of these met the target strength of 35 MPa, therefore concrete with waste aggregates (HDPE, SCBF, BA) at natural moisture state, were not viable for structural applications. The reason was because of the unpredictability in the strength yielded, therefore with further research into developing a specification guideline for waste concrete mix design to yield a predetermined concrete strength based on a calculated mix proportions, there was future potential for the waste mixes to be used for structural applications as they could be classified as High strength (30 MPa) concrete based on the compressive strength results from this study.

In terms of the relationship with workability, the compressive strengths of SCBF and BA did not show a typical inverse relationship with workability as was the case with HDPE. Instead, the strength of the SCBF and BA decreased with decreasing workability relative to the conventional mix. This meant that the possible viable applications were reduced as the mixes containing SCBF or BA at natural moisture state were not only weaker, but also less workable than conventional concrete.

Mixing waste materials was not viable in terms of compressive strength as the HDPE and SCBF mixes showed 13.9% and 33.67% declines in 28 day compressive strength respectively when BA was added. Moreover, when all three mixes were added together, the reduction in 28 day strength relative to the conventional mix was 46%.

In relation to past research, findings on the effect of SCBF on compressive strength confirmed research by Racines & Pama (1978), who showed sharp decreases in compressive strength for high volumetric substitutions. This study also expanded on research by Racines & Pama (1978), by showing that low-volume substitutions exhibited a similar behaviour to high-volume substitutions. The HDPE compressive strength results confirmed and expanded on research by Rahman *et al.* (2012), Suganthy, *et al*. (2013) and Al-Manaseer & Dalal (1997) by broadening their findings on compressive strength properties to include low-volume substitutions. Findings on the compressive strength of BA mixes used in this study confirmed findings by Kurama & Kaya (2007), which stated that significant strength benefits are generally not realised at curing ages prior to and including 28 days.

Flexure

In terms of flexural strength, HDPE generally resulted in the highest flexural strength properties yielding the highest 7 day and 28 day flexural strengths out of the three waste materials. BA mixes were the worst performing of the three wastes, yielding lowest 7 and 28 day flexural strengths.

The highest strengths for each waste were 5.06 MPa (HDPE), 4.68 MPa (SCBF) and 4.59 MPa (BA). These were yielded at the lowest volumetric substitution of 2,5% and were within the general flexural strength range of 3-5 MPa as stated in section 2.3.5.2. All mixes however decreased in flexural strength relative to the conventional mix. Therefore, the 2,5% volumetric substitution was not a “peak” performance value but rather the volumetric substitution that yielded the least reduction in flexural strength relative to the control mix. The extent of the reductions in flexural strength at 28 days, relative to the conventional mix at 2,5% volumetric substitution were 6,8%, 13.85% and 15.15% for HDPE, SCBF and BA respectively. Therefore, in terms of flexural performance, the use of waste mixes yield no benefit relative to the conventional mix.

It was noticed that the 7 day strength of SCBF was higher than that of HDPE and BA. This meant that at 7 days the bagasse fibres provided a better resistance to flexure than the other waste materials due to effective crack arresting, as it had a higher tensile strength and modulus of elasticity than the HDPE pellets. However, as the bagasse fibres were possibly softened after being exposed to the alkaline cement environment for up to 28 days, the modulus elasticity would have decreased and hence offered less resistance to flexure than the HDPE waste mixes. The flexural results for the BA mixes were the lowest due to the lack of pozzolanic reaction taking place.

The waste mixes developed a greater percentage of 28 day strength at 7 days with regards to flexural strength in comparison to compressive strength. The 28 day flexural strength was generally 15% of 28 day compressive strength for HDPE and BA, and 17% for SCBF.

Mixing waste material was not advised in terms of flexural strength, as results showed further reductions in 28 day flexural strength relative to the conventional mix when BA was added to the HDPE and SCBF optimum mixes respectively. Moreover, when all three wastes were added together in a concrete mix, 28 day flexural strength decreased by 32.5% when compared to the conventional mix.

In relation to past research, the findings from this study were in contrast to those by Sivarja *et al.* (2010), indicating that a possible peak may exist at substitutions below 2,5% and hence the percentages in this study did not cover the substitution range that benefits may have been realised for SCBF. The variations in concrete performance with Wang*, et al.* (2000) and Ramirez-Correti (1992), also indicated that varying SCBF properties such as average fibre length and inherent tensile properties, can cause variances in strength results amongst studies. The variability in material properties was also evident by the contrast in results when comparing the findings from this study and Aggarwal, *et al*. (2007), which showed decrease in flexural performance at low-volume substitutions, with those by Kurama & Kaya (2007) which showed potential benefits at 5% substitutions. Findings on HDPE in terms of flexure confirmed and expanded on past research by Ismail & AL-Hashmi (2007) for substitions of 2,5% and 5%. Based on findings from this study, there was no benefit to using the waste materials selected in terms of flexural strength.

Splitting

The splitting strengths of the SCBF mixes were generally higher than the HDPE and BA mixes. This was also the case with the compression tests, indicating that SCBF reacted better under direct compressive load than flexural load.

The HDPE and BA mixes decreased at all substitution tested in this study. Therefore, there was no “peak” value noticed for these waste materials in respect to splitting strengths. The least reduction in splitting strength for HDPE of 9.8% was at 2,5% substitution. The least reduction for BA was at 5% and was 13.8% less than the control. The decreases in splitting strengths for the HDPE and BA mixes may be attributed to the weakened aggregate-cement interfaces falling within the applied tension plane. SCBF differed from the other waste materials in a sense that it increased splitting strength for the 2.5% to 5% volumetric substitutions. This indicated that SCBF offered more resistance to shearing than flexural load by improving splitting strength by 15.5% relative to the control mix.

In terms of the relationships between strength properties, the splitting strengths were generally approximately 10-11% of 28 day compressive strength and ±60% of 28 day flexural strength. The flexural strengths were higher than the splitting strength results because as mentioned in section 2.3.5.3, the beam offers some compressive resistance when under flexural load as per Oregon State University (2014).

Mixing waste materials was not advised as with flexural and compressive strength, because the results yielded further reductions in 28 day flexural strength. These reductions were 14% and 30% relative to the conventional mix when BA was added to HDPE and SCBF mixes respectively. Moreover, when all three waste were added together in a concrete mix, 28 day splitting strength was decreased by 21% relative to the control, to 2.67 MPa.

In relation to past research, the BA findings confirmed findings by Aggarwal, *et al.* (2007). The findings on splitting strength for HDPE and SCBF correlated with past research by Ferreira, *et al.* (2012) and expanded on research by Sivarja *et al.* (2010) respectively by stating that splitting strength benefits for SCBF are not only limited to substitutions of 1,5%, but continue until 10% volumetric substitutions. Apart from SCBF, there was no benefit in splitting strength by using the waste materials tested.

4.7. Specific heat of waste concrete

Specific heat (J/kg°C) was tested in this study to provide an idea of the impact on thermal properties of the concrete mixes due to the addition of waste materials. As stated in section 3.7, thermal performance is usually assessed based on specific heat, thermal conductivity and thermal diffusivity, however, due to equipment constraints only specific heats was evaluated in this study. Therefore, the results gave an idea of the implied impacts on thermal performance as opposed to a definitive verdict on thermal performance.

The type of material was stated by Al Faruq, *et al*. (2013) to have an effect on the specific heat of the concrete mix because, as stated in section 2.3.6, the specific heat of concrete is the sum effect of the specific heat values of the individual constituents. This was confirmed in this study as shown by Figure 4-21, which indicates the varying specific heat results for the different types of individual waste mix specimens with the control shown as 0% substitution. Refer to Appendix I1-2 for data.

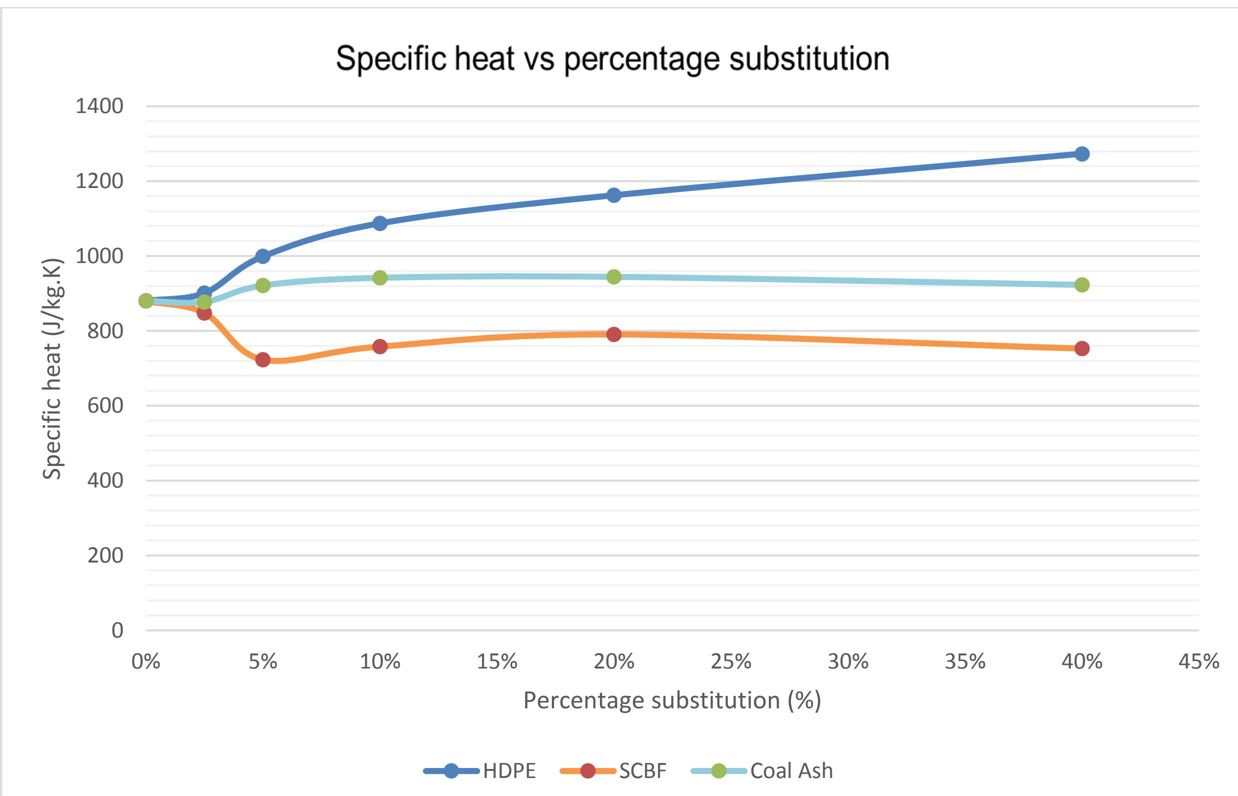


Figure 4-21: Graph of specific heat results for individual waste mixes

The specific heat of the control concrete was tested to be 880 J/kg°C. This was identical to the literature value of 880 J/kg°C (The Physics hypertextbook, 2013), thus rendering the test methodology plausible.

Figure 4-21 indicated a common divergence point at 2.5% for all materials. The reason for this is that at 2.5 % the proportions of waste in the respective mixes were too low to induce any notable change in specific heat, so the specific heat properties behaved similar to the control (0%).

After this divergence point, the HDPE showed an increasing trend, the BA produced an almost linear plot and the SCBF showed a decreasing trend. The specific heat behaviours shown therefore corresponded with the specific heat properties of the waste materials shown in sections 2.2.2.3, 2.2.3.3 and 2.2.4.3 for SCBF, HDPE and BA respectively.

4.7.1. Comments on the SCBF concrete specific heat results

The literature value of specific heat for sugarcane bagasse was found to be 460J/ kg°C (Shrivastav & Hussain, 2013). This was lower than the stone it replaced, so in theory the specific heat of the concrete should decrease. This was confirmed by the results of the specific heat tests on SCBF which ranged from 847.83 9J/kg°C at 2.5% substitution to 722.9J/kg°C at 5% substitution. This was a 4% and 18% decrease relative to the control mix respectively. The specific heat tests on SCBF showed an average decline with an increase in the proportion of sugarcane bagasse. The substitutions at 10% and 20% also showed decreases relative to the control but did not conform to the declining trend. This could have been attributed to human error or mild fluctuations in climatic conditions.

The specific heat sample containing SCBF and respective density results were directly proportional because as density decreased relative to the control sample, so did the specific heat of the bagasse samples. This confirmed findings by Holiday (2013), on the proportional relationship between density and specific heat.

The lower specific heat from the SCBF mixes would however not be of any benefit in terms of improving passive heating/cooling measures.

4.7.2. Comments on the BA concrete specific heat results

The literature value of the coal ash was 730J/kg°C (Torrenti, et al., 2013). This value was close to the cement that it replaces so in theory the specific heat should remain similar. This was confirmed with the findings from this study as the specific heat did remain close to the control mix despite the relatively steep gradient from 2.5% to 5% substitution, which was still within a variation range of 0.3% to 7,3%. The maximum increase of 7,3% relative to the control mix was found at 20% substitution and the minimum specific heat was found at 2.5% volumetric substitution differed from the control by less than a percent.

The close specific heat readings of the BA concrete to the control meant that BA had little impact on the specific heat of concrete. This meant that cement could be partially replaced with BA and the effect on the thermal mass of the building would be almost negligible. Differences in specific heat may have been attributed to chemical variations in the cement or the porous and unreactive nature of the bagasse used, which allowed voids to form thus introducing a greater degree of permeated water into the sample during testing and hence increasing specific heat. The specific heat results of BA also correlated to the decrease in density of BA samples shown in section 4.5.

As with the SCBF mix, the lower specific heat would not be of any benefit in terms of improving passive heating/cooling measures.

4.7.3. Comments on the HDPE concrete specific heat results

The specific heat of concrete with HPDE pellets showed a general increase, this was because the specific heat of the HDPE is significantly higher than that of the stone it replaced. It was noticed that as the proportions of HDPE doubled, the specific heat of the concrete increased by an average of 10%. This showed the dependence of specific heat on the proportions of material used. This confirmed findings by Elzafraney *et al.(*2005) who showed an increase in thermal mass, which is realted to the specific heat of concrete, by adding plastic aggregates.

The density of HDPE had an inverse relationship to specific heat but the decrease in HDPE sample density may have been due to the material being physically lighter than the stone it partially replaced.

The peak value of 1272.88 J/kg°C was found at 40% substitution which yielded a 44% increase in specific heat relative to the control mix. The greater the specific heat, the more heat energy was required to raise the heat of the material. Therefore, considering the implied effect on thermal mass, on a hot day, it would take longer before the interior of a room increased in temperature and during the colder nights the heat would be retained longer due to the higher capacity. Therefore, based on the findings from this study, the use of HDPE in concrete would be beneficial in terms of passive heating and cooling.

4.7.4. Comments on specific heat results for mixed waste combinations

Figure 4-22 indicates the specific heat results for the mixes at optimum/critical volume percentage already tested (based on compressive strength) and the mixed waste combinations.

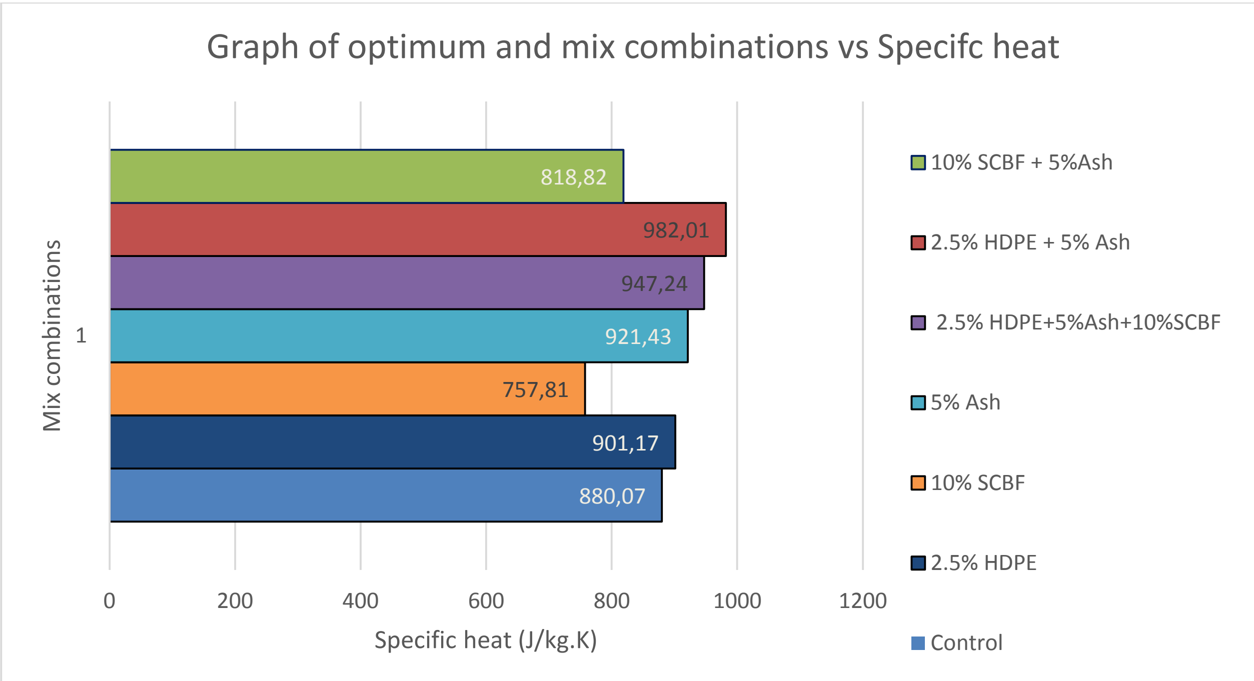


Figure 4-22: Graph of mixed waste specific heat results

The HDPE mix was slightly more sensitive to the addition of BA than SCBF, showing an 8,9% increase versus 8,05 % for SCBF relative to the mix without BA. The close percentage could imply that the effect of BA on a concrete mix is independent of the waste aggregate used. The specific heat of the HDPE + SCBF +BA mix was less than the HDPE + BA by 3.5%, more than the SCBF + BA mix by 15,6% and more than the control by 7,6%. The behaviour of the mix with all three waste materials indicates the net effect of the individual specific heat impact brought about by the waste materials.

Although specific heat would not have a large bearing on the viability of the waste mixes, it can be said that based on purely specific heat, BA would be the best material for applications similar to conventional concrete. HDPE would suit applications that required concrete that can absorb heat effectively, such as housing panels used for passive heating and cooling measures. SCBF could be applied to scenarios that require a material which is sensitive to temperature variations, such as a wood-fire oven. Therefore, HDPE and BA had the most relevance to this study in terms of specific heat properties. Findings also confirmed those by Al Faruq, *et al*. (2013), on the influence on concrete specific heat using varying types of materials

4.8. Static elastic modulus of waste concrete

The elastic modulus tests were carried out for the optimum/critical volumetric percentages samples (as per compressive strength results) as well as the mixed waste combinations.

The factor used to convert cube strength to cylinder strength to obtain load points at which to take strain readings was 0.8 as per BS 8110 Part 120:1983. This was acknowledged however as a general equation and was compared to another equation extracted from BS 8110-120 shown below:

Cube-cylinder conversion factor = 0,76+0,2*LOG10(fcu/19,58) (BSI, 1983)

Table 4-13: Comparison of cube strength to cylinder strength conversion factors

Comparison between 0.8 and BS 8110 formula						
Mix number	Cube strength fcu (MPa)	Ratio from BS 8110 - 120 equation	Cylinder strength as per BS 8110-120: fck (MPa)	Cylinder strength using 0.8 fck (MPa)	Variance (MPa)	% Difference
control	42,1	0,83	34,80	33,68	1,12	3,31
2,5% HDPE	29,95	0,80	23,87	23,96	-0,09	-0,39
10% SCBF	32,16	0,80	25,83	25,728	0,10	0,39
5% BA	30,39	0,80	24,26	24,312	-0,06	-0,28
HDPE +BA	25,79	0,78	20,22	20,632	-0,41	-2,01
SCBF + BA	21,33	0,77	16,37	17,064	-0,69	-4,07
HDPE+ SCBF + BA	22,62	0,77	17,47	18,096	-0,62	-3,43
Average		**0,79**				

The average factor using the formulae from BS 8110 was 0.79. This was a 1,26% difference to the 0.8 used for the purpose of obtaining load points. The maximum percentage variance in strength was 4%.

Table 4-14 shows the cumulative strain at selected stresses which was plotted in Figure 4-23 (see Appendix F for full data).

Table 4-14 : Average cumulative strain and stress results

Average cumulative strain							
Stress	Control	2.5 %HDPE	10% SCBF	5 % BA	10% SCBF + 5% BA	2,5% HDPE + 10% SCBF	2,5% HDPE + 10% SCBF + 5% BA
0,5 MPa	0,01	0,02	0,01	0,01	0,02	0,02	0,02
fc/3	0,28	0,18	0,29	0,27	0,22	0,25	0,25
0,4 fc	0,34	0,20	0,34	0,31	0,26	0,30	0,30

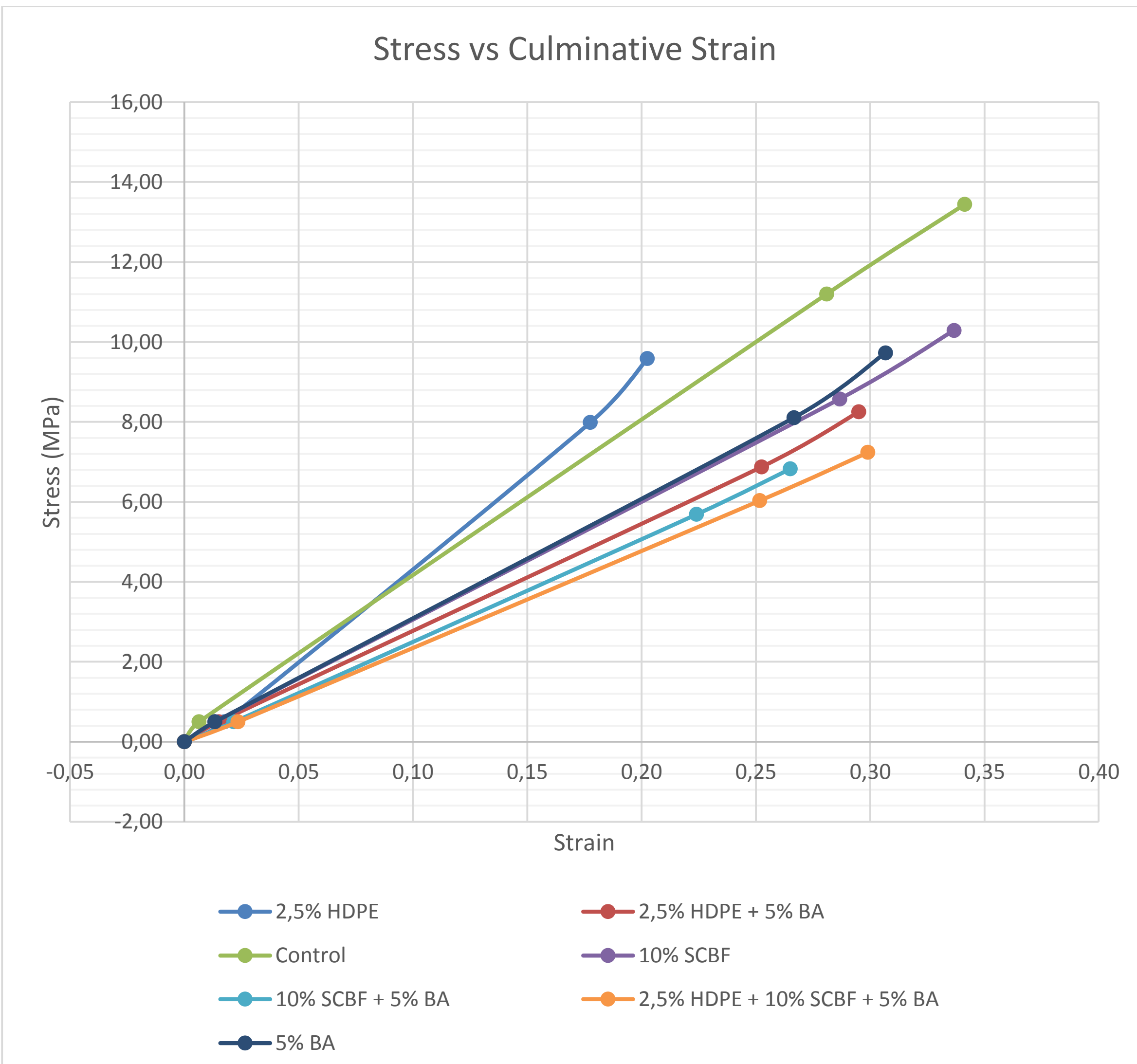


Figure 4-23 : Graph of cumulative strain vs. stress for waste mixes

Figure 4-23 & Figure 4-24 on page 237, show that the HDPE mix was the stiffest (steepest gradient-49.1 GPa) and the mix comprised of all wastes (HDPE + SCBF + BA – 24.45 GPa) was the most flexible (flattest gradient). When compared to the control mix, the following changes in stiffness were noticed based on the gradient of the stress-strain plot:

- HDPE – Increased elastic modulus by 27,1%
- SCBF – Decreased elastic modulus by 21,6%
- BA - Decreased elastic modulus by 18,5%
- HDPE + BA - Decreased elastic modulus by 27,7%
- SCBF + BA - Decreased elastic modulus by 32,7%
- HDPE + SCBF + BA - Decreased elastic modulus by 36,7%

The HDPE mix may have yielded a high static elastic modulus because the 2,5% volumetric waste substitution may not have been sufficient to impose any negative effects on the concrete elastic modulus, but was enough to bridge cracks and stiffen the mix. The latter statement, although a possibility, is unlikely because of the poor bond properties associated with HDPE pellets, which dictates load transfer between the cement matrix and the aggregate and hence the ability of the aggregate to resist deformation under load. This indicated the possibility that at 2,5% substitution, the negative bond characteristics and low elastic modulus of HDPE may be negligible but at higher substitutions such as 7,5% and 15 % as tested by Ferreira, *et al*. (2012), reductions in elastic modulus due to poor bond properties and low material elastic modulus, are noticeable.

As per Kaplan (1959) the varying properties of aggregates, in this case, SCBF and BA wastes, did affect the concrete matrix differently compared to the control mix. The SCBF mixes may have softened under exposure to the alkaline cement matrix thus decreasing the stiffness of the fibres and hence the stiffness of the concrete mix. This confirmed findings by Lamond & Pielert (2006) and Bentur & Mindess (1990) which stated that low-modulus aggregates were likely to reduce the elastic modulus of the concrete mix. However, the findings on SCBF concrete, from this study contrasted with those by Sivarja, *et al*. (2010), who showed increases in the elastic modulus at 1,5% volume substitution. This indicated that 1,5% may be a limiting volumetric substitution with regards to SCBF. This was where the introduction of SCBF was low enough to not compromise the stability of the matrix and still impart a degree of crack arresting properties to improve the elastic modulus of the concrete matrix.

The BA mix by itself at optimum/critical volume substitution (5%), reduced the elastic modulus relative to the control mix. This was because the BA used was shown to be unreactive based on the lack of beneficial strength development, and hence reduced load transfer between the aggregates and the paste matrix due to poor bond characteristics. When BA was added to the other waste mixes the stiffness of the respective mixes subsequently decreased further.

It was noticed that elastic modulus behaviour mimicked that of compressive strength behaviour in a sense that if compressive strength was reduced then so did elastic modulus. This followed findings as per Davis & Alexander (1989) and Ferreira, *et al.* (2012), who showed a direct relationship between elastic modulus and compressive strength. To further discuss the relationship between compressive strength and elastic modulus, the compressive strength results from section 2.3.5.1 were applied to the formula below, as per BS 8110 part 2 (BSI, 1985) ; and the subsequent theoretical results were compared to the actual results achieved.

Predicted $E_{28} = K_o + 0.2\ f_{cu28}$

Where, $K_0 = 19$ for Coedmore quartzite (used in this study)

The results of the predicted static modulus tests compared to the actual results carried out in this study are shown in Figure 4-24.

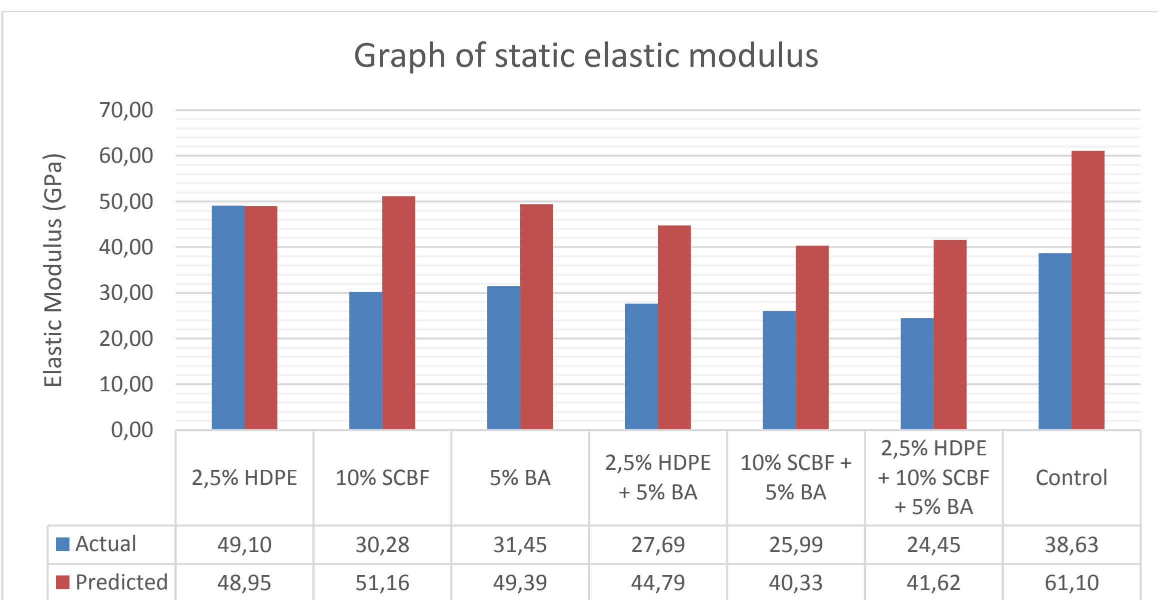


Figure 4-24: Graph of predicted static elastic modulii vs actual static elastic modluii

The HDPE actual results were within 0.3% of the predicted value. This further emphasizes that, at 2,5%, the volume of HDPE in the mix was too little to affect the mix, as the formula used for predicting the elastic modulus is based on the behaviour of conventional concrete. The SCBF and BA mixes showed 69% and 57% variation from the predicted results, whereas the mixed waste substitutions showed variances of 61,7%, 55,1% and 70,2%, for the HDPE +BA , SCBF + BA and HDPE + SCBF + BA mixes respectively. The variations to the predicted values may have been attributed to the properties of the waste material used altering the compressive strength–elastic modulus behaviour when compared to conventional concrete. However, the predicted elastic modulus for conventional concrete varied by 58%. This raised a question as to the validity of the prediction equation and created an opportunity for future research into developing the equation to consider waste mixes. Due to the nature of the static elastic modulus test, small variations in strain gauge alignment, flatness of the load surfaces or the bonding of the strain gauges with the surface can drastically affect the results and this may have been the case when the control sample and the HDPE sample were tested.

Generally, tensile strength is proportional to elastic modulus (The Constructor, 2014) and this was investigated for this study. The percentage changes in comparison with the control in terms of flexure, splitting and elastic modulus for the optimum/critical volume mix proportions (based on compressive strength) and the mixed waste specimens, are shown in Table 4-15.

Table 4-15: Table of percentage changes relative to the control mix (Flexure, Spitting, and Elastic modulus)

	Flexure change	**Splitting change**	**Elastic modulus change**
2.5 % HDPE	6,8 % ↓	9,8 % ↓	27,1 % ↑
10 % SCBF	18,5 % ↓	1,6 % ↑	21,6 % ↓
5 % BA	19,3 % ↓	13,7 % ↓	18.6 % ↓
2,5% HDPE + 5% BA	21,9 % ↓	13,7 % ↓	28,32 % ↓
10% SCBF + 5% BA	29,5 % ↓	29,8% ↓	32,7 % ↓
2,5% HDPE + 10% SCBF + 5% BA	32,5 % ↓	20,9% ↓	36,7 % ↓

Table 4-15 indicates that changes in elastic modulus are more sensitive to variations to constituent material properties than flexure and splitting strength. Generally, the flexural and splitting strength showed a directly proportional relationship to the elastic modulus which confirms the theory that tensile strength is directly proportional to elastic modulus. The reasons why elastic modulus decreased was because of the poor bond properties with regard to HDPE, softened fibres with regard to SCBF and lack of pozzolanic reaction taking place with regards to BA. SCBF at 10% substitution was one of the exceptions, as it transferred load more effectively under splitting load than flexural load and showed that flexural strength would be a better gauge to predict elastic modulus than splitting strength. HDPE at 2,5% was the other exception, which showed that at low-volumes, the impact of elastic modulus was negligible. Improvements to elastic modulus were either due to a variations in cement properties or errors due to testing anomalies such as strain gauge bonding with the sample surface or flatness variations of the specimen.

To conclude, each waste affected the mix differently, as per Kaplan (1959). Findings by Lamond & Pielert (2006) and Bentur & Mindess (1990) on the behaviour of low-modulus aggregates in a matrix were confirmed and findings by Sivarja, *et al*. (2010) showed that SCBF is better used at 1,5% with regard to elastic modulus. Compressive properties were found to be proportional to elastic modulus, as per Davis & Alexander (1989) and Ferreira, *et al*. (2012), with the exception of HDPE. However, behaviour differed from the predicted values due to variations in waste properties in comparison to the conventional mix constituents. The tensile properties were shown to have a relatively direct proportional relationship with the elastic modulus properties, with the exceptions of the optimum/critical volume HDPE and SCBF mixes, for reasons explained. In terms of application, the static elastic modulus gave an indication of the deformation resistance of the hardened concrete, therefore, all mixes except for the optimum/critical volume HDPE mix were more susceptible to increased deformation and cracking when compared to the control mix which may affect serviceability conditions and therefore reduce the viability of the mixes.

4.9. Durability of waste concrete

The durability of the optimum individual waste mixes and mixed waste specimens were evaluated by carrying out tests related to the three transport mechanisms which lead to the deterioration of concrete. As stated in section 2.3.8 concrete deteriorates when harmful agents are transported into the concrete mix through permeation, diffusion or absorption. In order to test these respective properties, the Oxygen permeability, Chloride conductivity and Water Sorptivity tests were used. Due to the specialist apparatus required, an external company (Contest) carried out the tests.

4.9.1. Oxygen permeability index (OPI)

The OPI is a measure of the pressure decay of oxygen over time, which gives an indication of the permeability of the specimen. Figure 4-25 illustrates the OPI index results for the optimum/critical volumetric substitutions (based on compressive strength test), mixed waste substitutions and the lower bounds for the OPI ranges stated in Table 2-17 page 121 .

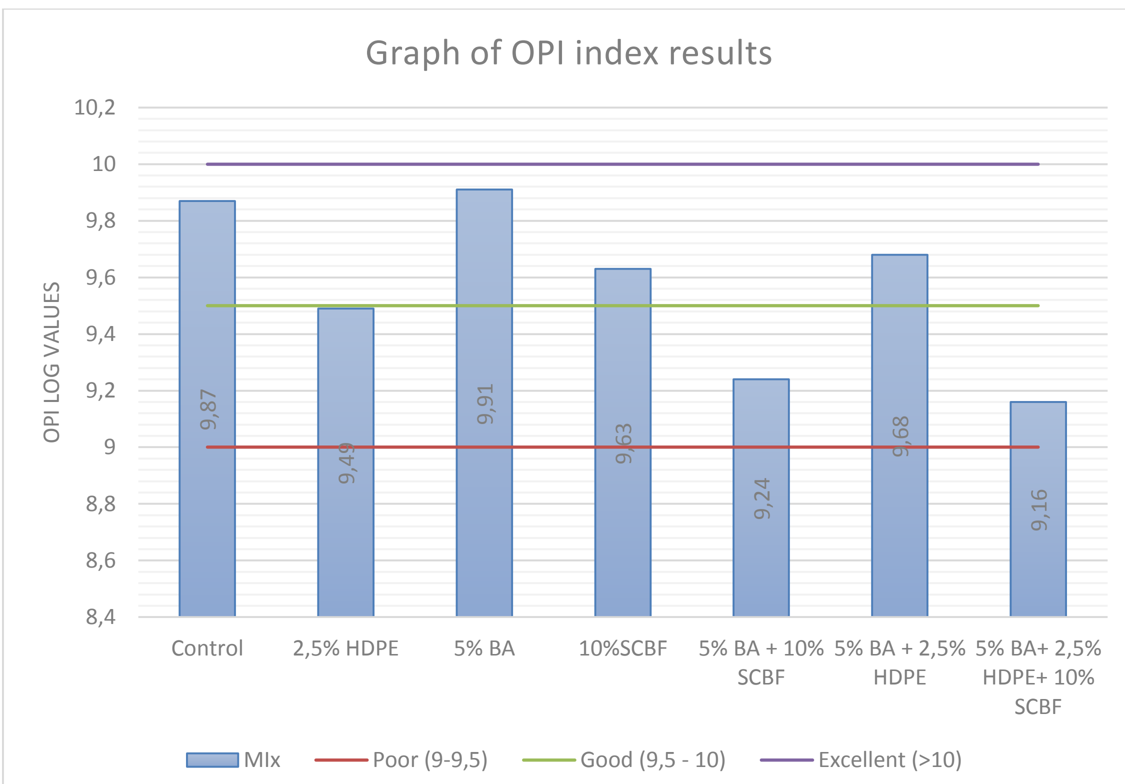


Figure 4-25: Graph of OPI results for mixed waste combinations and optimum volume waste mixes

The oxygen permeability index logarithmic values generally range from 8 to 11, the higher the reading, the better (less permeable). All mixes fell within this range. SCBF and HDPE increased the permeability of the control (conventional mix) mix by 2,43% and 3,85% respectively, whereas the mix containing BA reduced the permeability by 0.4% relative to the control mix.

Table 4-16: Percentage changes in OPI relative to control mix

2.5 % HDPE	3,85 %	↓
10 % SCBF	2,43 %	↓
5 % BA	0,41 %	↑
2,5% HDPE + 5% BA	1,93 %	↓
10% SCBF + 5% BA	6,38 %	↓
2,5% HDPE + 10% SCBF + 5% BA	7,19 %	↓

The increase in permeability for the mixes containing SCBF may have been due to the long fibres balling and hence forming additional voids. This reasoning correlated to the findings by Omoniyi & Akinyemi (2013), who showed that for substitutions above 3%, permeability increased because of the balling of fibres at higher volumes which clumped together leaving voids in the mix. However, the 556% increase in permeability at 3% substitution showed by Omoniyi & Akinyemi (2013), was 222 times more than the increase of 2,5% increase shown in this study. This indicates the variability of concrete properties when SCBF is added. This may have been because the bagasse samples used by Omoniyi & Akinyemi (2013) may have had a larger proportion of long fibres than the bagasse used in this study, hence a higher degree of balling and clumping may have occurred thus increasing permeability to greater extent for substitutions beyond 3%. Although Omoniyi & Akinyemi (2013) evaluated water permeability and used a different testing procedure, results were still comparable because the changes in permeability were made relative to a control mix and hence this allowed for comments on the general effect of bagasse on the permeability of the concrete mix.

The increase in permeability yielded by the SCBF mixes in this study, may have also been attributed to the softening of SCBF fibres which may have affected the bond interface with the cement matrix. This statement was substantiated by reductions in the elastic modulus, 28 day flexural and compressive strengths, which indicated the possible softening of SCBF. These findings correlated with findings by Gram (1986) and Daniel (2002) who stated that the increase in permeability was due to the softening of SCBF in an alkaline environment.

The permeability of the HDPE mixes may have increased due to the poor bond properties of HDPE with the cement matrix which could have caused the film of water to form in the aggregate- cement interface. This subsequently increased void content and thus increased the permeability of the mix. The statement regarding the film of water forming in the aggregate-cement interface, corresponded to the increase in workability relative to the conventional mix as shown in section 4.4.

The HDPE and SCBF mixes also showed decreases in density relative to the conventional mix. This indicated that in addition to the materials being lighter than the stone they replaced, there was also the possibility of additional voids being introduced to the mix, hence decreasing density and subsequently reducing the respective compressive strengths and elastic moduli. This confirmed findings by Bird & Callaghan (1977) and Al-Manaseer & Dalal (1997), which showed that as the concrete mix becomes more permeable the compressive strength decreases.

With regard to the BA mix, the minimal 0.4% reduction in OPI relative to the control mix, may have indicated a pozzolanic pore filler effect taking place but because there were no significant improvements in strength properties (section 4.6), or reductions in density (section 4.5), this effect was unlikely. The more probable reason was slight variations in the cement composition, or errors in test procedure such as incomplete oven drying samples which would lower conductivity as per Stanish, *et al*. (2006). This study which indicated the permeability behaviour of BA mixes for the curing age of 28 days, expanded on findings by Nunuz-Jaquez, *et al*.(2012), who showed significant further reductions in permeability and improvements to strength between 56- 90 days, once the pozzolanic reaction was more active. This emphasized that the introduction of BA was more viable at curing ages above 56- 90 days if improved permeability was a critical application requirement.

By adding HDPE to BA, the BA mix was made 2% less permeable than the mix with just HDPE. As with the reasoning for the BA mix, this may have been due to a very slight pozzolanic effect taking place as there were no increases to the mixes respective strength (section 4.6) and density (section 4.5) results relative to the control mix. However, as with the BA mix, it is was likely that there were slight variations in the cement composition, or errors in test procedure as mentioned.

When SCBF was added to the BA mix the permeability decreased by a percentage of 4,05% relative to the BA mix without SCBF. This meant that the balling effect of SCBF may have been more evident and this coupled with the lack of substantial pozzolanic reaction from the BA, increased permeability. The increase in permeability was also in contradiction to findings by Gram (1986),but this was because the BA used in this study did not show any substantial pozzolanic behaviour which would have reduced permeability significantly in theory had it taken place. When all three wastes were mixed together the permeability increased by 7,19% relative to the conventional mix due to the compounding effects on permeability of the waste materials mentioned.

In terms of application, permeability affects the extent of entry and rate of movement of liquids or gases through concrete (Mays, 1992). Based on the Oxygen permeability test from Table 2-17 on page 121, all mixes except for the HDPE, BA + SCBF and the mix with all three wastes, were classified as having “good” permeability according the OPI characterisation ranges done by Alexander, *et al*. (2009). This indicated that increases in permeability for the SCBF and SCBF + BA mixes were not significant enough to negatively impact on durability. The HDPE mix however was 0.01 below the lower bound of “good” classification and still may have durability issues in environments where there was a significant external pressure head, such hydrostatic pressure for submerged structures. This meant that despite the permeability of all mixes except the BA mix being more permeable than conventional concrete ,deterioration due to harmful agents permeating through the concrete mix would be unlikely for all mixes except the HDPE, BA + SCBF and the BA + SCBF + HDPE mixes, based on findings from this study.

4.9.2. Chloride conductivity (CC)

Diffusion is the primary means of chloride ingress in concrete. The chloride conductivity test gave an indication of the resistance of the concrete specimens to the ingress of chloride ions by diffusion.

Figure 4-26 illustrates the CC index results for the optimum/critical volumetric substitutions (based on compressive strength test), mixed waste specimens and the lower bounds for the CC ranges as stated in Table 2-17 page 121 .

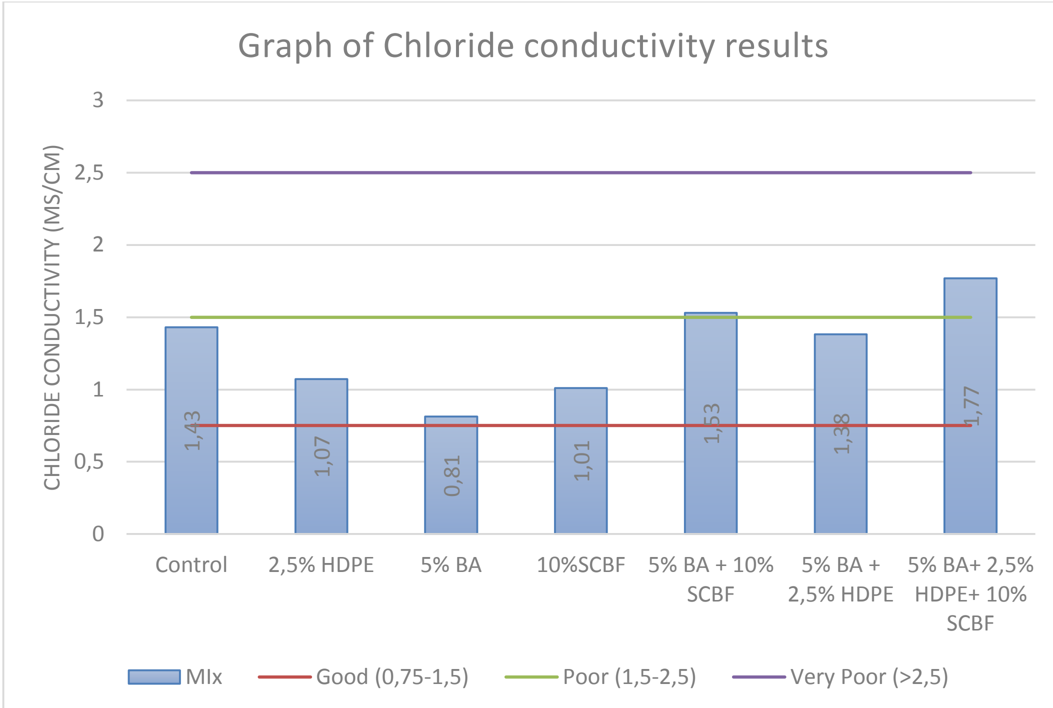


Figure 4-26 : Graph of CC results for mixed waste combinations and optimum volume waste mixes

The lower the CC results, the more resistant the specimens were to the ingress of chloride ions by diffusion, because a lower conductivity value implied a slower rate of chloride ion travel through a concrete specimen.

Table 4-17: Percentage changes in CC relative to the control mix

2.5 % HDPE	25%	↓
10 % SCBF	29.37%	↓
5 % BA	43.18%	↓
2,5% HDPE + 5% BA	3.32%	↓
10% SCBF + 5% BA	6,99%	↑
2,5% HDPE + 10% SCBF + 5% BA	23,78%	↑

As shown in Figure 4-26, all mixes except for the SCBF + BA and the HDPE + SCBF + BA mixes were more resistant to chloride diffusion than the control as they showed decreases in chloride conductivity relative to the control mix.

With regards to the HDPE mix, the OPI and WS results pointed to an increase in porosity due to the poor aggregate bond qualities of HDPE. However, the CC results showed an improvement in CC by 25% relative to the control. This may have been because of the HDPE was a good electrical insulator (Polyplastics, 2001) and hence reduced conductivity results. This indicated that the electrical resistivity of HDPE may have been more significant than changes to pore structure (based on OPI) in terms of CC.

For SCBF, the improved CC indicated that the electrical insulating properties (Singh & Pathania, 2009) of SCBF restricted conductivity. This showed that the increased voids in the mix, as shown by the OPI results, were less significant in terms of the effect on CC, than the electrical resistivity of the SCBF.

The SCBF and HDPE results indicated the possible influence of aggregate electrical properties on CC, which followed findings by Alexander, *et al*. (2009) which also indicated that electrical resistivity may be more significant than pore structure in terms of CC. However, there could have also been the probability of errors in testing procedure such as incomplete saturation and oven drying of samples, which would result in a lower conductivity value (Stanish, et al., 2006) or variations in room temperature which could decrease conductivity by 2% for every degree Celsius below 25 °C (Thermo Fischer Scientific, 2014) .

The reduced CC for the BA mixes relative to the control mix, contrasted findings by Basheer & Bai (2003), which showed that the introduction of BA increased CC readings due to the porosity of BA. The fact that BA has poor electric resistivity which would decrease CC, removed the possibility that there was influence on conductivity due to the BA's electrical properties. This meant that a possible explanation for the increase in CC was that there was some degree of pozzolanic reaction that took place within the specimen sampled for the CC test and this created a pore filling effect that reduced conductivity, as per (Alexander, et al., 1999). This would also confirm one of the explanations for the reduction in OPI shown in section 4.9.1. However, because the decrease in conductivity relative to the control was so substantial (43,18%), as opposed to the OPI (0,41%), it could have been attributed to errors in carrying out the experiment by the external company such as incomplete saturation or oven drying of specimens (Stanish, et al., 2006) or variations in room temperature which could decrease conductivity by 2% for every degree Celsius below 25 °C (Thermo Fischer Scientific, 2014) .

The HDPE+ BA mix reduced conductivity by 3,32% relative to the control sample, due to the electrical resistivity of the HDPE pellets. However, the addition of BA to the HDPE did make it more conductive than the HDPE mix alone. This was because of the poor interaction between BA and HDPE which resulted in more voids and hence higher conductivity. The reductions in CC for the SCBF + BA and the HDPE + SCBF + BA could be attributed to the poor interaction between the aggregate interface and cement matrix when BA was added. This increased conductivity by 6,99% and 23,78% respectively, relative to the control. These findings confirmed those by Ganesan, *et al*. (2007) and Yuksel & Demirtas (2013) on the influence of pore structure and interfacial interaction on CC.

Based on the results from this study, the mixes with HDPE yielded the least conductivity and according to the durability characterisation developed by Alexander, *et al*. (2009), all specimens besides the SCBF + BA and the HDPE + SCBF + BA mixes, were classified as having “good” chloride conductivity. This meant that these mixes would provide improved resistance to chloride diffusion relative to the control mix and hence improve corrosion resistance. The combination of mixes was generally not advised as it increased conductivity. It was also noticed that based on the HDPE and SCBF results, the electrical properties of the aggregates may have a more significant effect on CC than the pore structure of the mixes, which contrasted findings by Ganesan, *et al*. (2007) and Yuksel & Demirtas (2013). They stated that the interfacial zone interaction and pore structure have the greatest influence on CC results, which was noticed in this study for the BA and mixed waste combinations. However, there may have been the possibility of errors in testing procedures for the HDPE and SCBF mixes, which showed no correlation to changes in pore structure shown by the OPI tests. Also, for the BA mix, the significant differences in changes in relation to the control, for the OPI and CC results, indicated that there may have also been errors in CC testing procedures.

4.9.3. Water sorptivity (WS)

Figure 4-27 illustrates the WS index results for the optimum/critical volumetric substitutions (based on compressive strength test) and the mixed waste substitutions. The lower bounds based on the recommended durability classifications researched by Alexander, *et al*. (2009) are also shown to better illustrate the performance of the mixes.

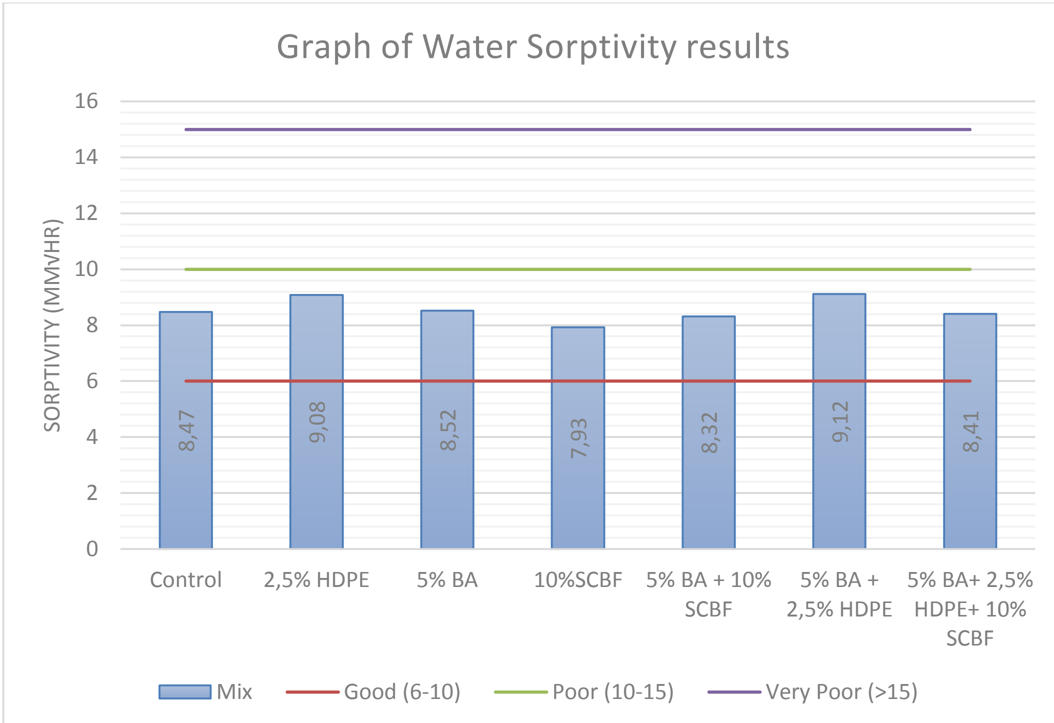


Figure 4-27 : Graph of WS results for mixed waste combinations and optimum volume waste mixes

Based on the WS results shown in Figure 4-27 (lower the results the better the WS resistance), all mixes were classified as having "good" water sorptivity because the results fell within the range of 6-10 mm/hr as per the recommended durability classification researched by Alexander, *et al*. (2009). This meant that all mixes provided relatively good resistance to deterioration from the absorption of harmful agents.

The percentage changes in WS due to the addition of the waste materials relative to the control mix are show in Table 4-18.

Table 4-18: Percentage changes in WS relative to the control mix

2.5 % HDPE	7,16 %	↑
10 % SCBF	6,38 %	↓
5 % BA	0,59 %	↑
2,5% HDPE + 5% BA	7,67 %	↑
10% SCBF + 5% BA	1,77 %	↓
2,5% HDPE + 10% SCBF + 5% BA	0,71 %	↓

In terms of the changes to the control mix yielded by adding the waste materials, the addition of HDPE increased the water sorptivity by 7,2 %. This was because the hydrophobic nature of the HDPE particles possibly repelled water within the aggregate-cement transition zone and hence caused additional capillary voids to form. This statement was substantiated by the increased density (see section 4.5), lower OPI results and reduced compressive strength (see section 4.6.1.2) of the HDPE mix, which is behaviour typically exhibited by concrete when pore content is increased, as per research by Mays (1992).

The addition of SCBF decreased water sorptivity, because the fibres present in the mix may have assisted in restricting pore water capillary flow paths through swelling, or by making flow paths more tortuous. This contrasted the OPI results which showed increases in permeability relative to the control mix albeit to lesser extent of 2,43%, relative to the control mix. This indicated that the potential introduction of additional pores in the mix was either not significant enough to increase WS through capillary action or the pores were restricted significantly by the randomly oriented fibres in the mix.

The BA mix showed little difference with the control, increasing WS by 0,59%, and hence it can be said that at 5% substitution the effect of BA on WS was negligible. This was because the BA was unreactive at 28 days and hence there was no significant pore blocking benefit yielded by the pozzolanic products for the WS test sample. These findings also confirmed those by Andrade, *et al*. (2007) on th effect of BA on WS at 28 days. The WS results for the BA mix also differed from the OPI and CC findings in this study, which did reflect the possibility of a pozzolanic reaction taking place. This may have been due to variations in the BA samples applied to the respective test specimens or errors in the OPI and CC tests.

For the SCBF+BA mix, the possible capillary flow restricting properties of SCBF showed to be the dominant effect on WS, as there was still a reduction in WS despite the BA mix on its own increasing sorptivity relative to the control mix. The HDPE+BA showed an increase in WS because of the additional pores introduced to the mix by a combination of pore inducing properties from the BA and HDPE mentioned above for the individual mix discussions. This supported findings from the OPI results. The mix with all three wastes still showed a slight decrease in WS relative to the control mix because the SCBF had a more significant effect on the WS than the HDPE and BA.

To conclude on water sorptivity, all mixes containing SCBF improved WS properties relative to the control mix. Mixes containing HDPE and/or BA decreased in WS because of the additional pores introduced, however, the reductions were not sufficient to lower the classifications of the respective mixes to “poor”. Therefore, based on the results in this study, the viability of the selected waste materials were not compromised in terms of the transmission of harmful agents through capillary action even though WS results decreased in some cases relative to the control.

4.9.4. Concluding remarks on durability

4.9.4.1. In terms of the varying three durability tests carried out:

The OPI related to transport of deleterious substances under an applied pressure head, the CC related to the slowed ingress of ions under a concentration gradient and the WS referred to the ingress of deleterious substances due to capillary action. The WS test was the critical case as ingress of deleterious substances was the most rapid and did not require an external pressure head or concentration gradient to occur.

The oxygen permeability results showed the best correlation with properties such as pore structure, interface behaviour and density. The results for the CC test varied from literature in a sense that there was no correlation shown with the pore structure and interface behaviour of the mixes, but valid reasoning was provided for the effect of material resistivity properties. The WS results did show a correlation with density, pore structure and interface properties, however, mixes containing SCBF differed from current literature related to pore structure behaviour, thus expanding on knowledge of concrete containing SCBF. For all three tests there was the possibly of laboratory errors due to the complex nature of sample preparation.

The CC tests was the most sensitive to the use of waste materials. The OPI test was the least sensitive showing changes of 0,41% to 7,19% relative to the control mix which was similar to the WS test which varied from 0,59% to 7,67% relative to the control mix. This indicated a possible correlation between the WS and OPI tests which can be explained by the corresponding results which showed similar correlations to the pore structure and interfacial behaviour of the waste materials.

4.9.4.2. In terms of each waste:

The 2,5% HDPE mix was classified as having good sorptivity properties despite showing increases in WS by 7,16% relative to the control. HDPE was also classified as having good CC which was 25% less than the control. The magnitude of decrease was less than the

SCBF mix but this may have been because HDPE was at a lower substitution of 2,5% versus the 10% bagasse substitution and hence had less of an impact. The HDPE mix was however classified as being poor in terms of OPI due to its 3,85% increase in permeability relative to the control mix, which was the worst of the three individual waste mixes. This was because of the poor pore structure in the interfacial zone between the HDPE and the cement paste. This indicated that the durability tests were more sensitive to pore structure for WS and OPI but the HDPE's inherent resistivity properties were more significant in terms of CC. The HDPE mix may be more resistant to ions entering the mix under a concentration gradient and will not likely to be affected by the ingress of deleterious substances due to capillary action, but may be susceptible to attack with deleterious agents permeating into the mix under a pressure gradient.

The 10% SCBF mix was classified as "good" in terms of OPI even though it was 2,43% more permeable than the control. The SCBF was also classified as being good in terms of CC and WS and improved each by 29,37% and 6,38% respectively relative to the control mix. The improved resistance to chloride conductivity was the best from the three materials. This was because of the resistivity properties of SCBF and the relatively higher proportion of SCBF content (10%) compared to the other waste materials. For the SCBF mix, the durability tests were more sensitive to the pore structure for the OPI test but showed a greater correlation with the SCBF resistivity and fibre shape with regards to CC and WS. This meant it was less resistant to permeation of deleterious substances under a pressure gradient but more resistant to ingress of deleterious substances under concentration gradient and capillary action. SCBF would however need to be tested for the possibility of long-term fibre deterioration as part of future research, as this may potentially affect concrete durability and was not factored into the durability tests conducted in this study.

The 5% BA mix was classified as good in terms of OPI and CC as it decreased permeability by 0.41% and lowered CC by 43,18% relative to the control mix. This was the best performance for both properties out of the three individual waste mixes. This was because of the possible pozzolanic behaviour that may have created a pore filler effect. The BA mix was also classified as having "good" WS properties because even though it increased water sorptivity, the increase was minimal (0,59%). Therefore, the possible pore filler effect was noticeable in terms of OPI and CC but was not significant enough to yield any benefits in terms of WS as there may have been pockets of unreactive BA that caused additional capillary voids to form. This subsequently nullified any potential pozzolanic pore filler benefits for the WS sample. The BA mix was therefore more resistant to permeation of deleterious substances under a pressure gradient and the ingress of deleterious substances under concentration gradient, but less resistant to the effects of capillary action.

The mix of HDPE + BA was classified as having good OPI, WS and CC. It improved OPI by 1,93% and CC by 3,32% relative to the control which was the best from the three mixed waste specimens. This was because of the pozzolanic benefits of BA, but it was still more permeable and conductive than the individual BA mix for OPI and both HDPE and BA individual mixes for CC, due to the poor interface between HDPE and BA which possibly introduced more voids to the mix. The poor bond interface between HDPE and BA also resulted in WS being 7,67% more than the control, which was the worst performance out of the three mixed waste specimens. The SCBF + BA mix was classified as poor in OPI and CC because of the effect of the additional voids being introduced. However, it was also classified as having good WS properties, performing better than the control mix and was also the best out of the three mixed waste specimens in this regard. The specimen with all three waste materials performed the worst out of the three mixed waste specimens in OPI and CC but did improve WS by 0,71% due to the benefit of the SCBF. The mixing of the wastes was not advised because for OPI, CC and WS all mixed waste combinations were generally worse off than the respective individual waste mixes.

4.9.4.3. **In relation to past research:**

For the OPI tests, findings confirmed the increase in permeability relative to the conventional mix with the introduction of SCBF as per Omoniyi & Akinyemi (2013) and Gram (1986), as well as the relationship between compressive strength and permeability as per Daniel, (2002), Bird & Callaghan, (1977) and Al-Manaseer & Dalal (1997). Findings from this study also expanded on findings by Nunuz-Jaquez, *et al*. (2012) by indicating the OPI behaviour of BA at 28 days.

For the CC tests, it was also noticed that based on the HDPE and SCBF results, the electrical properties of the aggregates had a more significant effect on CC than the pore structure of the mixes, which contrasted findings by Ganesan, *et al*. (2007) and Yuksel & Demirtas (2013). However, there may have been the possibility of errors in testing procedures for the HDPE and SCBF mixes, which showed no correlation to changes in pore structure shown by the OPI tests.

Findings differed from those by Andrade, *et al*. (2007), because the BA used in this study was possibly less porous, and hence did not increase WS.

To summarise, the OPI and WS tests showed the closest correlations to pore structure properties which followed past research findings, whereas CC was more dependent on the resistivity of the waste materials and differed from past research findings. The use of the individual waste materials did show benefits in terms OPI for the BA, CC for all three individual wastes and WS for SCBF. However, HDPE was shown to fall just short of a “good” classification in terms of OPI. The mixing of wastes was not advised because for OPI, CC and WS, all mixed waste combinations, were generally worse off than the respective individual waste mixes. However, the BA+ HDPE was classified as having good durability properties for all three tests. Therefore, the durability properties of the SCBF and BA individual waste mixes, and the BA+ HDPE mix did not compromise their potential viability based on results from this study. The HDPE mix may be affected by ingress of deleterious substances under external pressure but would not be affected by WS and CC, the former being critical durability criterion as per Hycrete (2011). Therefore, it can still have the potential to be viable in terms of durability as the OPI can be mitigated by decreasing the W/C ratio as per Hycrete (2011). The mix containing all three wastes and the SCBF + BA mix were not viable in wet environments that were susceptible to the ingress of deleterious substances.

4.10. Scanning Electron Microscope (SEM) analysis of waste concrete

The purpose of the SEM analysis was mainly to enrich the study and did not have a bearing on the viability of the waste materials. It involved the SEM imaging of the interfaces between the HDPE pellets and SCBF fibres as well as selected areas in the BA samples. This was done to indicate how the HDPE and SCBF aggregates interact with the cement paste as well as show BA particles appear on a microscopic level.

Figure 4-28 shows a strand of Bagasse fibre embedded in the cement matrix. The image shows the C-S-H matrix forming a reasonable bond interface with the SCBF, however, the presence of distinct voids located in certain areas in the interfacial zone are noticeable.

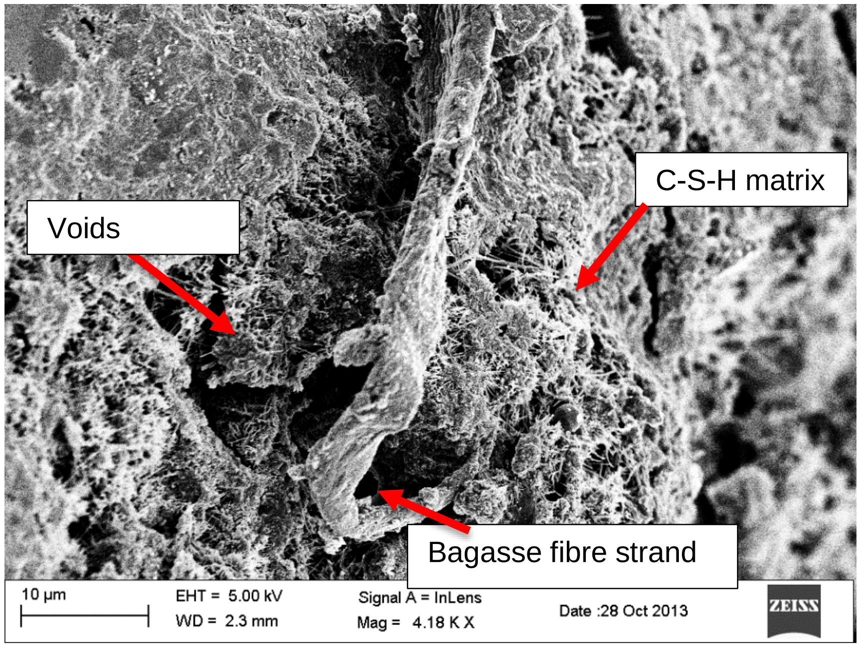


Figure 4-28: SEM image of the SCBF-cement interface

Figure 4-29 shows an HDPE pellet embedded in the cement matrix. The surface of the pellet is relatively clear of cement paste which would be present if the cement particles bonded to its surface. There is also a distinct ridge around the border of the HDPE–cement interface.

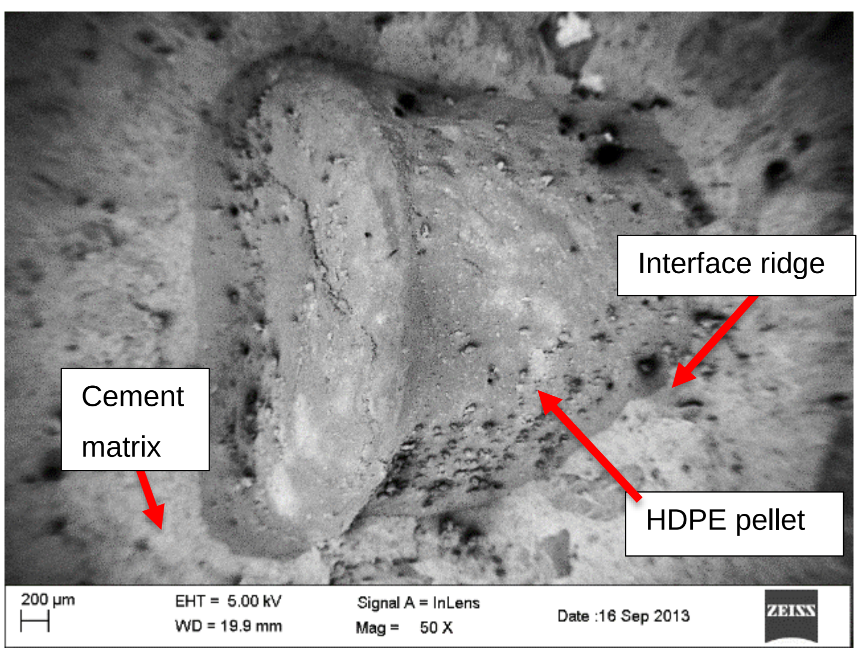


Figure 4-29 SEM image of the HDPE-cement interface

Figure 4-30 is a magnification of Figure 4-29 and shows the noticeable gap between the HDPE pellet and cement matrix clearly. This emphasises the poor bond properties of HDPE with the cement matrix.

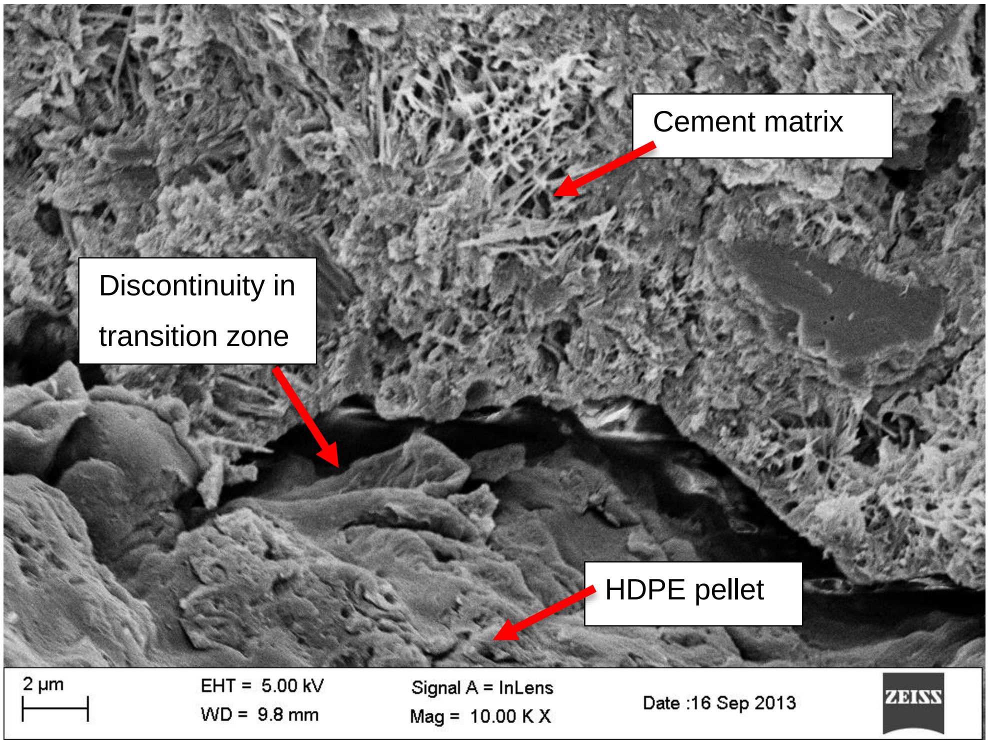


Figure 4-30: SEM image of HDPE interface

Figure 4-31 shows the round BA particles in the C-S-H matrix.

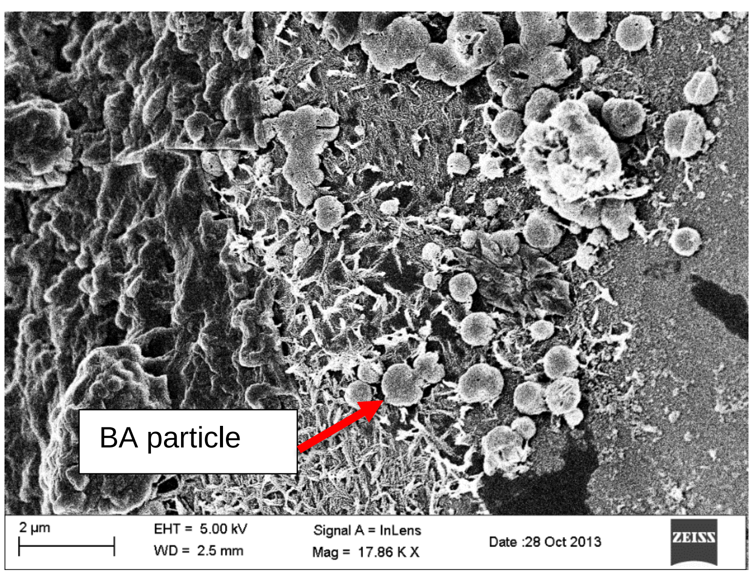


Figure 4-31: SEM image of BA mix

Figure 4-32 shows the matrix of the conventional concrete mix. This was shown to compare with the BA matrix in Figure 4-33.

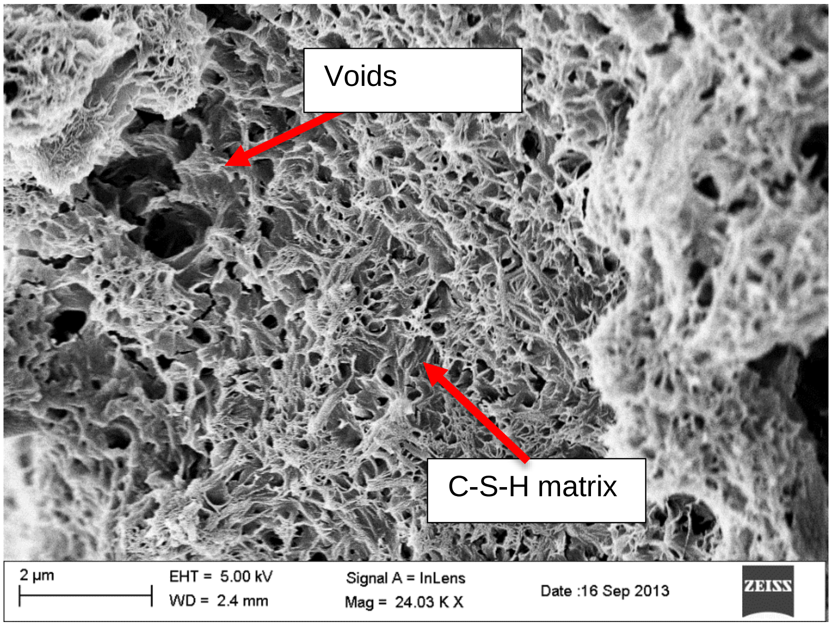


Figure 4-32: SEM image of conventional concrete C-S-H matrix

Figure 4-33 shows the BA matrix. When compared to the control cement matrix sample there was a noticeable increase in the quantity and size of voids present .This could have been due to the unreactive BA particles in the mix.

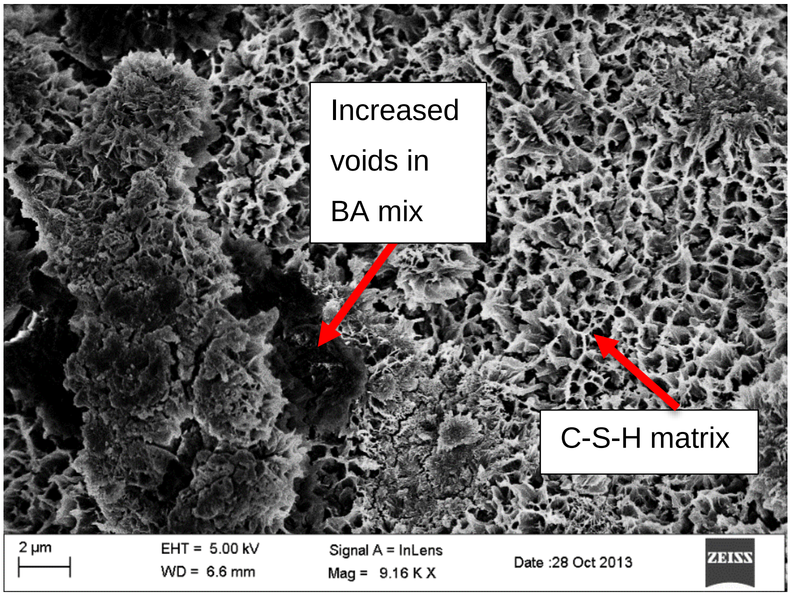


Figure 4-33: SEM image of BA mix C-S-H matrix

The SEM images provided in this chapter indicated the distinct lack of bonding between HDPE and cement. They also showed that SCBF had reasonable bond with the C-S-H. However, there were noticeable voids near the fibre-paste interface. Finally, the images indicated the shape of BA in the mix which conformed to the literature image as well as the increase in voids when comparing the BA mix matrix with the conventional mix. The information regarding the increase in voids may explain the reduced strength for the waste mixes compared to the control mix.

4.11. Moisture analysis of waste aggregates

The HDPE, SCBF and BA samples were oven-dried (OD) before being added to the mix. In theory this meant that the aggregates would be at their maximum absorption potential and hence would reflect the maximum effect on workability and compressive strength because of the respective waste materials due to their moisture properties. The specimens using dry-aggregate were compared to the compressive strength and workability results for the specimens using waste materials at natural moisture state.

The workability results for the specimens using natural and dry waste aggregates are shown in Figure 4-34 (refer to Appendix H for full data).

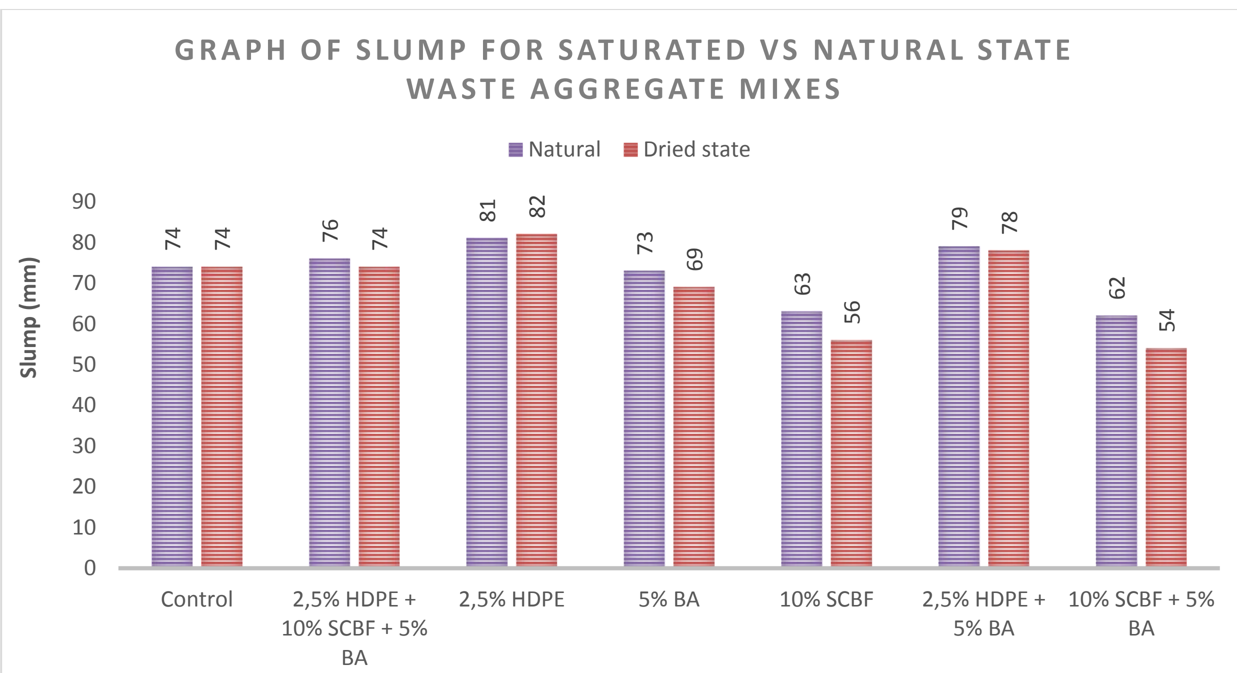


Figure 4-34: workability for natural state & dry state waste aggregates

It was noticed that by drying the waste aggregates, the specimens with BA and SCBF which had a relativity higher moisture absorption (refer to section 4.2.4) than the HDPE, decreased the workability by greater margins of 5,47% and 11,11% respectively compared to the workability of the corresponding mixes using natural state waste aggregates. This behaviour confirmed findings by Gardiner & MacDonald (2013), on the influence of dry aggregates on workability. The HDPE mix changed by 1.2% which indicated it was almost independent of the moisture state because of its hydrophobic nature and low–moisture absorption properties.

When comparing the natural and dry states further, the bagasse was moist at natural state and the difference between moisture absorption and moisture content was 119%. This absorption potential at natural state translated to a 14.86 % decrease in workability relative to the control, but when the dry SCBF was used this variance in workability with the control mix increased to 24% due to the higher moisture absorption potential. The SCBF mixes decreased workability relative to the control mix because of the high cellulose content which gave it a high absorption property as well as the long-fibres possibly inter-locking, thus lowering workability.

For the BA mix, the difference between moisture content and moisture absorption at natural state was 0.9%, which resulted in a decrease in workability of 1,35%. When the dry BA was used the workability relative to the conventional mix was decreased by 6.76%. The decrease in workability was due to the porous nature of BA.

When dry BA was added to dry SCBF, the workability was decreased by 3.5% and when dry BA was added to dry HDPE the workability decreased by 4,8% relative to the respective mixes without BA. When all three dry waste materials were used together, the workability was similar to the control. This was because the increase in fluidity brought about by using the HDPE was cancelled out by the absorption properties of the SCBF and BA.

The average results of the compressive strength tests are shown in Table 4-19 and are illustrated in Figure 4-35 (refer to Appendix H for full data).

Table 4-19 : Table of compressive strength results using dried vs. natural moisture state waste aggregates

Dried vs. natural moisture state waste aggregate concrete compressive strength results (MPa)								
		Mix						
		Control	2,5%HDPE	5% BA	10%SCBF	5% BA + 10% SCBF	5% BA+ 2,5% HDPE	5% BA+ 2,5% HDPE+ 10%SCBF
Waste aggregate Moisture state	Dry 7-day	24,7	19,26	24,5	24,4	20,7	23,4	21,3
	Dry 28-day	42	31,4	37,8	40,1	37,5	35,6	35,9
	Natural 7-day	25,32	18,64	22,09	20,23	12,02	16,60	13,25
	Natural 28-day	42,10	29,95	30,39	32,16	21,33	25,79	22,62

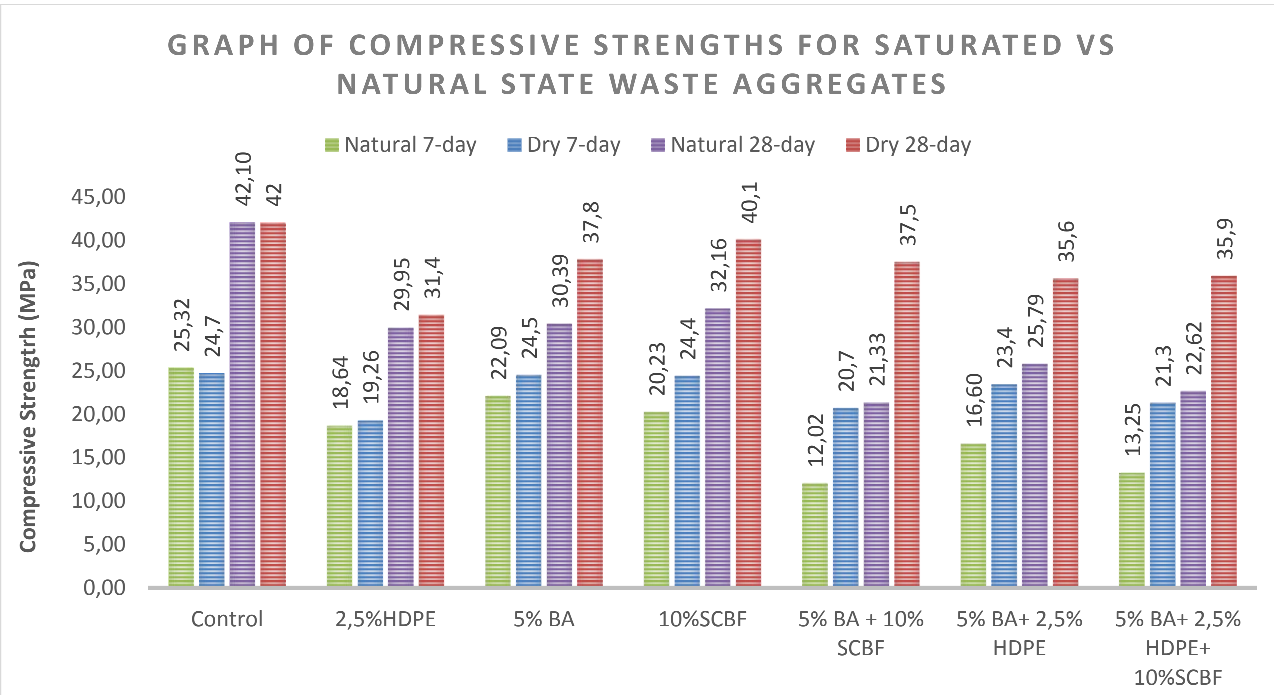


Figure 4-35: Graph of compressive strength using dry and natural moisture state waste aggregates

As shown in Figure 4-35, all compressive strengths were increased by using dry-aggregates as opposed to aggregates at natural state. The extent of the changes, however, was based on the waste materials inherent moisture properties. The increases in compressive strength were shown for both 7 and 28 day results and the differences in 7 and 28 day results also remained relatively consistent when comparing the natural state and dry state waste aggregate specimens, thus indicating that the varying moisture states did not affect the rate of strength gain.

The SCBF mixes showed the most significant increases in compressive strength between natural state and dry state waste aggregate specimens from the three waste materials, of 17% and 19% for 7 and 28 days respectively. This was probably because it was the most absorbent of the three wastes materials and hence decreased W/C ratio more significantly in the interfacial transition zone, thus improving compressive strength. This behaviour, confirmed past research on using aggregates at OD states, by Sallehan & Mahyuddin (2014).

The HDPE mixes showed the least change in compressive strength when comparing dry and natural mixes, of 3,1 % and 4,6% after 7 and 28 days respectively. The HDPE pellets were not absorbent and hence the low variation in strength properties when comparing the natural and dry state aggregate specimens.

The BA mixes showed a 9,83% and 19,59 % increase in compressive strength, for 7 and 28 days respectively, relative to the corresponding natural state mix. The variations in compressive strength were possibly due to the variation in absorbency potential due to the porous nature of the BA particles.

With regards to the mixed waste substitutions, the BA + HDPE mixes showed a 29% and 27% increase in compressive strength at 7 and 28 days respectively, relative to the natural state mixes. The increase in compressive strength was either due to the absorption of water by the BA particles or a possible pozzolanic reaction taking place. However, the 28 day strength achieved for the BA + HDPE was less than the BA mix alone possibly because of the poor bond interface with HDPE. The BA and SCBF showed a 41% and 43% increase in compressive strength at 7 and 28 days respectively, relative to the natural state mixes. The increase in compressive strength over the natural state specimens was due to the absorption properties decreasing the W/C ratio in the interfacial transition zone. The strength achieved by the SCBF+ BA mix was less than the SCBF on its own possibly due to the reduction in hydrates because of the cement removed from the mix after BA substitution. The combination of all three mixes showed a 37% and 36% increase in compressive strength at 7 and 28 days respectively, relative to the natural state mixes. The improvements in strength were probably due to the absorbency properties of the BA and SCBF.

To summarize the effect of waste aggregate moisture properties on the concrete mix. As mentioned, the reason for the changes in compressive strength and workability were due to the increase in absorption potential of the dried waste aggregates. The increased absorption potential resulted in more free water being drawn from the mix in the interfacial transition zone, hence decreasing the W/C ratio, reducing the amount of pores in the mix and increasing compressive strength. The repercussion for the increase in strength however was a corresponding decrease in workability but not to the extent that the mixes became unusable. The increase in strengths for all mixes except for the HDPE mix, were enough to meet the target strength of 35 MPa thus rendering them potentially viable for structural use. For HDPE, as stated when commenting on the natural state waste aggregate compressive strength results, even though the control target strength was not reached, the mix still met the 30 MPa high–strength concrete classification. However, only with further research into producing a waste concrete specification for HDPE to reach a pre-determined target strength, can the HDPE mix be potentially viable for structural applications.

4.12. Economic analysis

The economics behind the use of new materials forms a key role in determining the overall viability of the waste concrete mixes, as industry is unlikely to utilize a material that incurs higher costs unless there are significant concrete property benefits (e.g. improved strength /lowered permeability) to utilizing the material.

As stated in sections 2.4 and 3.12 , the economic sustainability was assessed in two steps. Firstly, the waste concrete material rates in R/m^3 based on market rates were established in section 4.12.1 using the costing method by Popescu, *et al*. (2005). Secondly the scenario cost analysis for cast in-situ equipment room and precast platform coping applications were carried out in section 4.12.2 based on the Quantity Take Off (QTO) method (Dalton, et al., 2011).

Conclusions of economic viability were then made by:

- Comparing the cost of the waste material concrete mixes to the conventional mix.
- Investigating the percentage of the overall cost to construct that was made up by material cost.
- Investigating which aspect of the scenario cost model formed the largest portion of the total cost to construct.

4.12.1. Material costs model

The material costing was based on the costing methodology by Popescu, *et al* (2005). To cost the respective waste mixes individual material costs had to be obtained from suppliers or calculated, as in the case of the SCBF cost calculated in section 4.12.1.1.

4.12.1.1. **Waste material costs**

SCBF

The rates for SCBF were not readily available according to the Sugar mill research institute (SMRI) as they are used for confidential power generation feasibility studies. The cost value of the sugarcane bagasse had to be related to the value of the coal that it replaced in a boiler. The process below was used to calculate the cost value of the sugarcane bagasse based on calorific values and the amount of kcal required to generate 1kW of electricity.

Calorific values of compared values

Coal caloric value = 5500 kcal/kg (India Solar, 2013)

Sugarcane bagasse caloric value = 4400 kcal/kg (dry)   (India Solar, 2013)

Market value of coal

Market value of coal = R1075 /ton (interview from mill)

Calories to generate 1kW = 860 kcal /kW

Coal price per Kg/kW = (860 / 5500) = 0.156kg x R1.075 = R0.168 /kW

Relating coal price to bagasse price

Sugarcane bagasse kg/kW = (860/ 4400) = 0.3909 kg

Therefore Rand /kg of Sugarcane bagasse = 0.168 / 0.3909 = R0.43 /kg

HDPE

The recycled HDPE pellets were supplied by RE-SA situated in Prospecton, Durban. The rate was given in Rands per kg of material as shown in Table 4-20.

Table 4-20: HDPE cost

Material	Rate	Source/supplier
HDPE pellets	R 9,20 / kg	Re-SA

Coal ash

As per recommendations from SRMI and the Sezela sugar mill, the bottom ash used for this study had no value and is essentially free. The only cost incurred for its use, would be transport.

4.12.1.2. **Conventional materials costs**

The material costs associated with the respective materials shown in Table 4-21 were taken as undelivered rates to establish a concrete cost that was independent of location and transport costs and could be used as a base cost for general comparisons. Concrete was priced per bag and quantities rounded to the nearest half-bag of cement. The water costs were based on the assumption of a metered connection using municipal rates.

Table 4-21 : Costs of conventional materials

Material	Undelivered Rate (incl. VAT)	Source/supplier
Cement	R 67.50 per 50 kg bag	Aberdare Hardware
13,2 mm Stone	R 300.30/ m^3	Lafarge aggregates
Umgeni Sand	R 198 / m^3	Aberdare Hardware
Water	R16,47 / kl	Municipality

4.12.1.3. **Costing of waste mixes**

The mixed waste substitutions and the optimum/critical volume waste percentages (from compressive strength analysis) were priced based on the costing method by Popescu, *et al*. (2005), by multiplying the cost of the individual material constituents by the respective quantities required per m^3 of waste concrete. The resulting costs in terms of R/m^3 are shown in Table 4-22.

Table 4-22: Costs of individual waste mixes per m^3 of concrete

Individual waste mixes costing (R/m^3)								
	Control		2,5% HDPE		5% BA		10% SCBF	
	Quantity	Cost (R)	Quantity	Cost (R)	Quantity	Cost (R)	Quantity	Cost (R)
Cement	8 bags	540,00	8 bags	540,00	7,5 bags	506,25	8 Bags	540,00
Sand	0.346 m^3	68,7005	0.35m^3	68,70	0.35 m^3	68,70	0.35 m^3	68,70
Water	225 l	3,71	225 l	3,71	225 l	3,71	225 l	3,71
Stone	0,30 m^3	89,96	0.29 m^3	87,71	0,3 m^3	89,96	0.27 m^3	80,97
HDPE			6.93 kg	63,78				
SCBF							37,43 kg	16,10
BA					13,18	0,00		
Total		R 702,37		R 763,90		R 668,62		R 709,47

From the costing of the individual waste mixes, the most costly based purely on material cost, was the HDPE mix, which was 8.76% more than the control mix at R763,90/m^3. This was because the HDPE pellets were the most costly concrete constituent. The SCBF mix was 1% more expensive than the control mix at R709,47/ m^3. The cheapest mix (including mixed waste combinations) was the BA mix, which was 4,81% cheaper than the control mix at R 668.62/ m^3. The reduction in cost was because BA which partially replaced the cement in the mix, was a free material.

In order to evaluate the impact on cost of using more than one type of waste in the mix, the mixed waste combinations were also priced.

The costing of the mixed waste combinations in terms of a R/m^3 rate are show in Table 4-23.

Table 4-23: Costs of mixed waste combinations per m^3 of concrete

Mixed waste combinations costing (R/m^3)						
	10% SCBF + 5% BA		2,5% HDPE + 5% BA		10% SCBF + 2,5% HDPE + 5% BA	
	Quantity	Cost (R)	Quantity	Cost (R)	Quantity	Cost (R)
Cement	7,5 bags	506,25	7,5 bags	506,25	7,5 bags	506,25
Sand	0,35 m^3	68,70	0,35 m^3	68,70	0,35 m^3	68,70
Water	225 l	3,71	225 l	3,71	225 l	3,71
Stone	0,27 m^3	80,97	0,29 m^3	87,71	0,26 m^3	78,72
HDPE			6,93 kg	63,78	6,93 kg	63,78
SCBF	37,43 kg	16,10			37,43 kg	16,10
BA	13,18	0,00	13,18 kg	0,00	13,18 kg	0,00
Total		R 675,72		R 730,15		R 737,25

The cheapest mix was the SCBF + BA mix, which was 3,95% cheaper than the control mix. By adding BA to SCBF the cost was 4,75% less than the SCBF individual mix because of the BA partial replacement of cement. The individual HDPE mix was also reduced by 4,41% by adding BA. However, the HDPE + BA mix was still 3,95% more expensive than the control mix. The price of the mix with all three waste was 4,97% more expensive than the control mix and was also the most expensive of the waste mixed combinations because of the additional cost incurred by using SCBF and HDPE.

The amount that each constituent contributed to the total cost is shown in Figure 4-36.

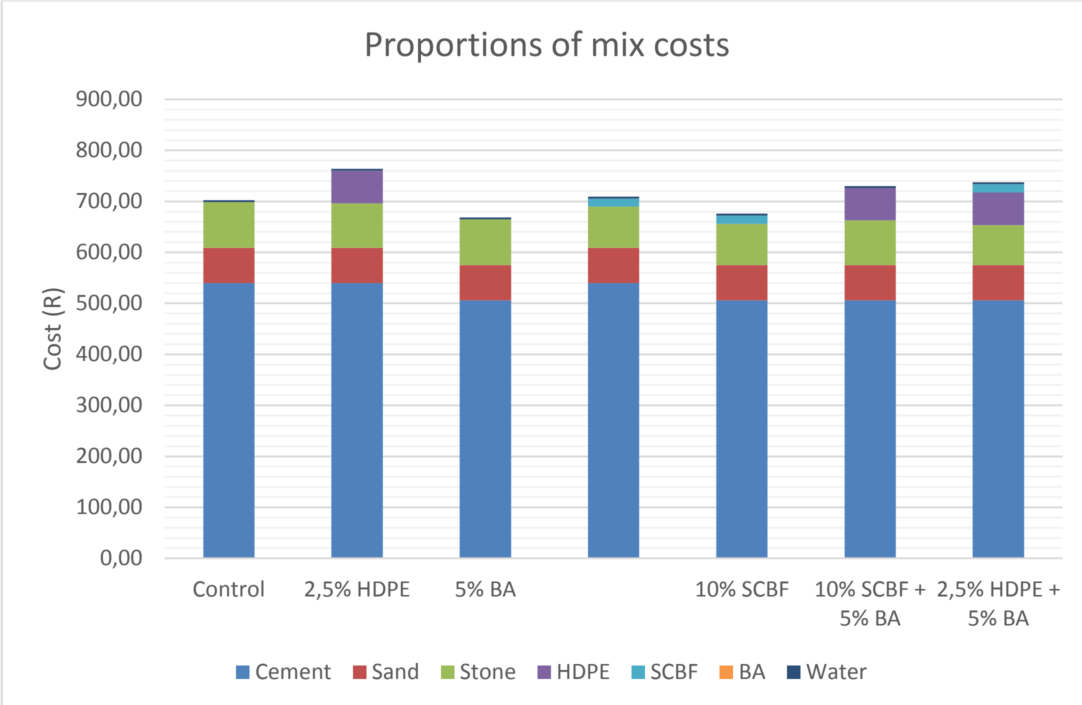


Figure 4-36`: Waste concrete cost/m³ breakdown

Figure 4-36 indicates that cement contributes the majority of material cost followed by, in decreasing order: stone, sand, HDPE, SCBF, BA and water. The fact that HDPE contributed more to cost than sand which was the highest quantity material in the mix by volume indicated how expensive the HDPE pellets were, as HDPE content was only 2,5% of the mix by volume. The contribution of SCBF was minimal and BA was free. Therefore, the introduction of BA, even at the 5% volumetric substitution, significantly reduced the cost of the mix between 4.4% to 4,75%.

4.12.2. Costing scenario analysis

As stated in section 2.4, the scenario analysis was based on the PRASA rail re-signalling project and the costing was based on the QTO method (Dalton, et al., 2011). The scenario was selected because it was relevant to contemporary South African construction landscape due to the current upgrade of rail infrastructure. The Pinetown train station works was used with permission from the contractor as it covered both in-situ and precast concrete applications. The cast-in situ scenario was a signalling storage room and the pre-cast scenario was coping slabs used on platforms. Refer to section 2.4 for further information about the scenarios selected and Appendix K and L for diagrams which were drawn using AutoCAD 2010 software.

The optimum/critical volumetric substitution mixes (based on compressive strength results) and the mixed waste combinations were used for the scenario analysis to add an economic context to viability evaluation of the mixes.

For each scenario, the material, plant, transportation costs and labour were all included in the costing to give a breakdown of each aspect of the cost to construct the signalling room as per the QTO method (Dalton, et al., 2011). This approach for the analysis was structured in a way that would indicate the contributions of each of the major cost categories (material, transport, plant and labour) to evaluate what aspect of construction was the most significant in terms of the cost to construct, thus giving an indication of how significant the changes in material used were.

4.12.2.1. **Cast in-situ signalling room**

The cast in-situ application (Figure 4-37) selected was a concrete signalling equipment room used to house rail signalling equipment.

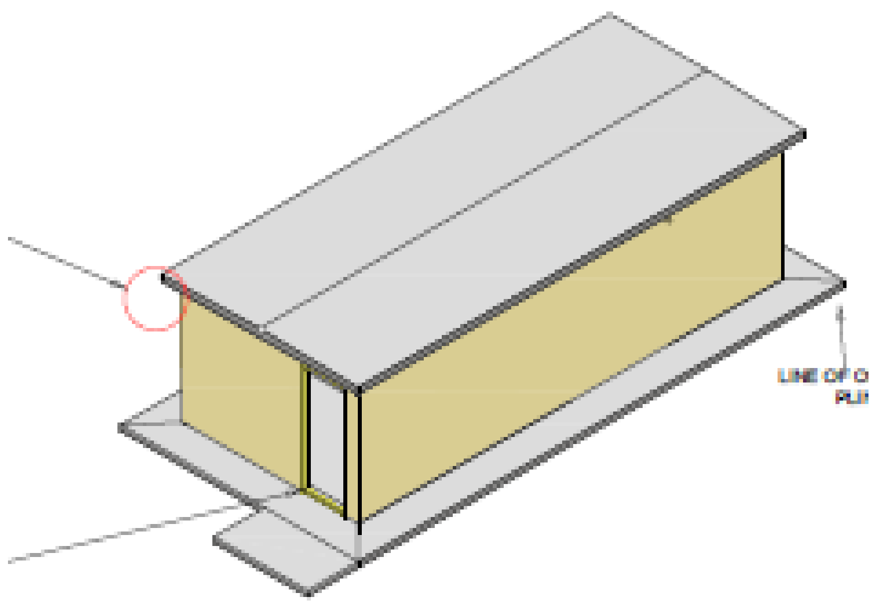


Figure 4-37: 3D diagram of signalling room

Material cost

In order to price the materials, rates were obtained from local suppliers to maintain the local context of the study. The concrete material costs calculated in section 4.12.1 and summarised in Table 4-24 were used for the scenario analysis. These costs used undelivered material rates for sand and stone from section 2.4 to enable a comparison between the components (materials, plant, labour, transport) that made up the total cost to construct.

Table 4-24: Concrete waste mix costs

Mix	Cost (R/m^3)
Control	R 702,37
2,5% HDPE	R 763,90
5% BA	R 668,62
10% SCBF	R 709,47
10% SCBF + 5% BA	R 675,72
2,5% HDPE + 5% BA	R 730,15
10% SCBF + 2,5% HDPE + 5% BA	R 737,25

Concrete only formed part of the total material cost to construct the signalling room. Items such as curing compound, water proofing and formwork were also considered. The rooms were built up from natural ground level and backfilled using G5 crusher hence excavations cost were omitted. The total list of material items that were selected to price the cast in situ equipment room are shown in Table 4-25, which shows the pricing for the control mix based on the format extracted from *"Unified Facilities Criteria (UFC): Handbook: construction cost estimating"* by Dalton, *et al*. (2011). The rates shown were in R/unit, with the "unit" being that shown in the unit's column. The pricing for the waste mixes are shown in a similar format in Appendix J2. The application of the concrete rate for the control mix calculated in section 4.12.1 is circled in Table 4-25. The waste mixes were done in a similar manner but using the respective waste mix material cost as the rate.

Table 4-25: Pricing for the signalling room – control mix

Signaling room Pricing –Control mix				
Item	**Unit**	**Quantity**	**Rate (Incl. VAT)**	**Cost (R)**
20 L Curing Compound	No.	2.00	170.00	R 340.00
G5 Crusher Run Filling Material	Tons	20.00	110.60	R 2,212.00
30 MPa concrete (changes depending on mix considered)	m^3	26.14	702,37	R 18 362,06
250 Micron Waterproofing membrane	m^2	60.32	4.67	R 281.69
Econo Form Panels for foundation	Item	1.00	481.53	R 481.53
900x 2100 Steel door	No.	1.00	4,825.00	R 4,825.00
Electrical - Lights	Item	1.00	6,000.00	R 6,000.00
Clips and wedges allowance	Item	3.00	300.00	R 23.68
Reinforcement Cut & bent (As per Bending Schedule)	Tons	1.11	9,650.00	R 10,682.55
Mesh Ref 245	Sheets	2.00	328.11	R 656.22
Mesh Ref 888	Sheets	3.00	1,189.20	R 3,567.60
Ferrule tubes and cones	item	3.00	263.16	R 20.78
Tie Rods	Item	3.00	632.40	R 49.93
Scaffolding	Item	1.00	263.16	R 263.16
2x3 Timber	m	36.00	15.50	R 558.00
50mm Duct Tape	Rolls	2.00	38.00	R 76.00
Shutter boards 18 or 21mm	sheet	12.00	370.00	R 116.84
Formwork to soffits of roof slabs exc 1.5m & not exc 3.5m high above bearing level	m^2	26.21	28.95	R 758.71
Drip Fillets	m	75.00	63.00	R 124.34
Steel corner fillets	m	150.00	63.00	R 248.68
Internal Paint (2 Coats)	Litre	14.00	70.18	R 982.52
Plaster undercoat	Litre	14.00	61.00	R 854.00
Shutter release oil 200l (coverage 5m^2 /l)	Litre	35.00	25.25	R 883.75
Consumables (nails, floats, gloves etc.) allowance	per unit	1.00	200.00	R 200.00
Cover blocks allowance	per unit	1.00	140.00	R 140.00
40x40x5mm Angle Iron	m	26.00	33.55	R 872.30
8mm Coach Screw	No.	45.00	5.00	R 225.00
4.5mm Heavy Duty Checker plate	m^2	3.69	672.46	R 2,481.38
110mm Cable Access Pipe (2.5m long)	m	25.00	33.80	R 845.00
Drywall Partitioning	m^2	16.00	300.00	R 4,800.00
Doors for drywall	No.	3.00	624.62	R 1,873.86
Total material cost				R 63 806,59

Figure 4-38 shows what percentage of the total material cost was made up of concrete cost because this study focused on the viability of waste concrete. Therefore, the concrete cost was relevant. Not to say that the total cost was irrelevant, as the total cost was needed to provide a more realistic costing perspective and hence a better understating of the impact of using waste concrete versus conventional concrete.

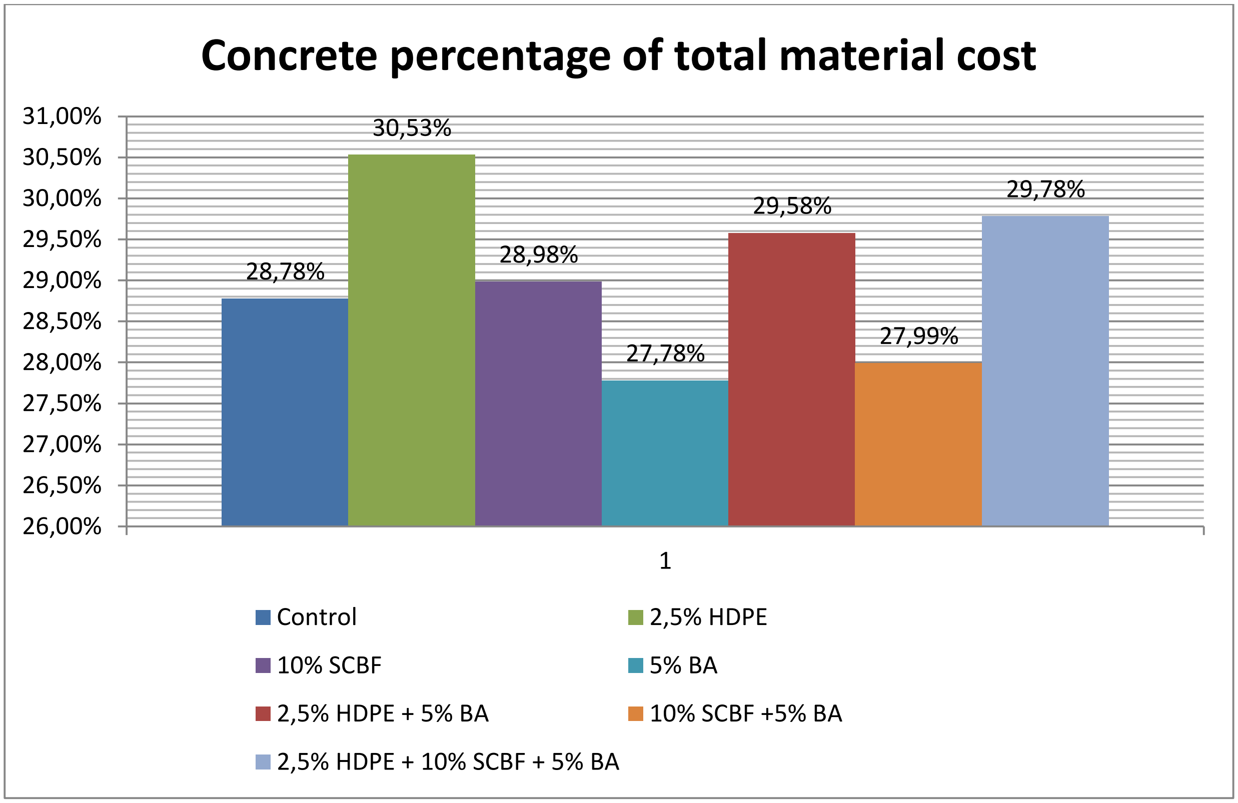


Figure 4-38: Percentage of total material cost

Based on the pricing shown in Appendix J2, the concrete cost made up around 28,98% (BA) to 30,53% (HDPE) of the total material cost, which was the highest contribution amongst materials used. This indicated that the cost of concrete used, was of significance. The mixes containing HDPE were slightly higher in percentage material cost due to the higher cost of HDPE pellets. The mixes containing BA were the lowest as BA was essentially a free material. All mixes except for the BA and SCBF + BA mixes showed increases in the percentages of total material cost represented by the concrete cost. This indicated that only the BA and SCBF + BA mixes reduced the cost price of concrete relative to the control mix.

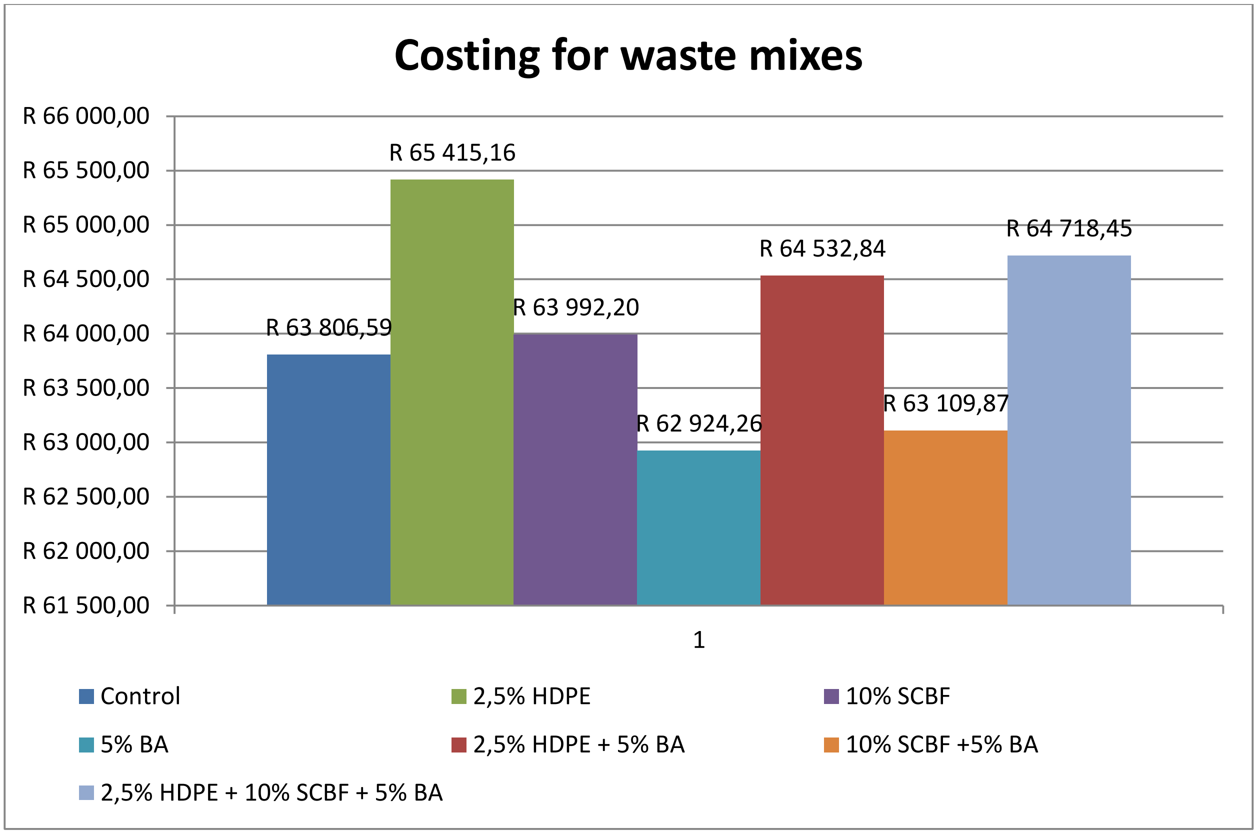


Figure 4-39: Total material costs

With reference to Figure 4-39, although concrete material cost formed the largest portion of the total material cost, by changing the type of concrete used, the total material cost relative to the control varied within a range of -1,4% to 2,5% depending on the waste mix being compared This meant that even though the difference between the control mix concrete price and waste mixes concrete price varied from a reduction of 4,8% (BA) to an increase of 8,7% (HDPE) in terms of R/ m^3 rates, when these mixes were used in a scenario, the total material cost implications were lesser.

The HDPE optimum/critical volume mix showed the highest increase in material cost of 2,52%, up from R63 806 .59 (control) to R 65 415,16. The largest reduction in material cost was noticed with the BA mix, which reduced material cost relative to the control by 1, 38% to R 62924,26. The addition of BA to the SCBF and HDPE mixes did reduce the material cost by 1,38% and 1,35% respectively, relative to the individual SCBF and HDPE mixes.

Labour cost

The variables required to price labour as per Dalton, *et al*. (2011) were:

- The time to construct
- The labour rates (R/hour).

Assuming it would take 8 general workers and a foreman 14 days to build a signalling room, the total time to construct was 126 hours, assuming 9 hours of production time in a working day. The labour rate for general workers was R29/hour and the rate for a foreman was R45/hour. Engineer and project management fees were not included in the costing as the model was focused on costs associated purely with production. The costs are summarised in Table 4-26.

Table 4-26: Signalling room labour fees

Designation	Rate (R/h)	Time(hours)	Cost (R)
8 No. General labour	29	126	R 3654 x 8 = R 29232
1 No. Foreman	45	126	R 5670
Total			R 34 902

Plant & Site establishment cost

A site container was set up, which cost R2840 inclusive of hire and handling costs. The sand and stone were stockpiled on-site once delivered and the SCBF(bagged), BA (bagged), cement and HDPE pellets(bagged) were stored in on-site containers.

The plant required to construct the signalling room are shown in Table 4-27, based on hire/purchase rates used by the contractor. The miscellaneous items covered saw discs, nails, diesel for the generator for power (owned-only cost is diesel) etc.

Table 4-27: Signalling room plant cost

Plant & tools	Rate	Time	Cost (R)
230 mm Skil saw	R 1920 (purchase)	-	R 1920 (purchase)
Plate Compactor	R 140 / day	14	R 1960
Concrete chute	R 131,58 (purchase)	-	R 131,58 (purchase)
Troxler	R 1665,73 (total)	-	R 1665,73 (total)
Concrete mixer	R 210 /day	14	R 2940
Drive unit & needle	R 190/day	14	R 2660
Site Container incl. handling & hire)	R 2840	-	R 2840
Consumable/Miscellaneous	R200	-	R 200
Total			R 11 917,31

Transport cost

The transport costs discussed were those related to the transport of the constituent materials used to construct the signalling room. The stone and sand transport costs were taken as the product of the difference between the delivered and undelivered costs per cubic metre of waste/control and the amount of concrete required, assuming delivery to Pinetown train station. For cement, the delivery charge was a flat rate of R200 from Aberdare hardware. The costing of transport for the SCBF, HDPE and BA were based on the methodology used by (Carter & Troyano-Cuturi, 2009) which considered both fixed (hire) and variable costs (fuel). Firstly, the amount of loads was established by dividing the total material required by the load capacity of the vehicle chosen. Then the distance travelled divided by 100, was multiplied by the vehicle fuel economy (l/100km) and the fuel price to get the variable cost. Fixed costs were then obtained from a plant hire company and the total transport cost for the waste materials was subsequently the sum of fixed and variable costs.

Table 4-28 shows the transport costs for the sand and stone.

Table 4-28: Transport costs for sand & stone

Transport cost for sand & stone (Cast in-situ Signalling room)								
	Control		HDPE		BA		SCBF	
	Delivered - undelivered R /m^3 (of concrete)	Cost (R)	Delivered – undelivered (R) /m^3	Cost (R)	Delivered - undelivered (R) /m^3	Cost (R)	Delivered - undelivered (R) /m^3	Cost (R)
Sand	8,67	226,75	8,67	226,75	8,67	226,75	8,67	226,75
Stone	12,86	336,04	12,53	327,64	12,86	336,04	11,57	302,44
	SCBF + BA		HDPE + BA		SCBF + HDPE + BA			
	Delivered - undelivered (R) /m^3	Cost (R)	Delivered - undelivered (R) /m^3	Cost (R)	Delivered - undelivered (R) /m^3	Cost (R)		
Sand	8,67	226,75	8,67	226,75	8,67	226,75		
Stone	11,57	302,44	12,53	327,64	11,25	294,04		

The rate shown in Table 4-28 for the transport of sand and stone was the difference in cost using the undelivered and delivered rates for the quantities of stone and sand needed per cubic metre of concrete. The difference between the delivered and undelivered cost rate was then multiplied by 26,41 m^3, which was the quantity of concrete required for the signalling room, to get the transportation cost for stone and sand in Rands.

Table 4-29 shows the costs incurred to transport the waste materials.

Table 4-29: Transport costs for waste materials

Transport cost for waste materials							
		Distance (1)	Fuel economy l/100 km(2)	Diesel Price (3)	Hire Cost	Variable cost (1) /100 x (2) x 3)	Total
HDPE	0.181 tons = 1 load	26.5 km x 2	9,9	R 11.61 /l	R 1 350	R 60.92	R 1 410,20
All mixes expect (Individual HDPE)	1 truck load	86,3 km x 2	24	R 11.61 /l	R 2286	R 528,16	R 2814.16

Even though the masses of waste materials required were relatively low, the storage space required to transport the waste materials led to a 10-ton tipper being used to model the transport costs for all waste mixes except the HDPE individual mix. A minimum hire time of 9 hours was required by the supplier as per industry norms, therefore, the minimum hire price for the tipper was 9 x R254 = R2286. The HDPE on its own, however, could be transported with a 1-ton bakkie and this incurred a hire cost of 9 x R150 = R 1350. Single trips were required for all mixes as the waste material loads were less than the respective vehicle loading capacities. (See Table 4-29 on page 273).

The SBCF and BA were sourced from Sezela sugar mill and the recycled HDPE pellets were purchased form RE-SA located in Prospecton. Distances were calculated by multiplying the distance to the destination from Pinetown station by the number of trips taken multiplied by 2, to take into account the departure and return trip. The fuel economy of the vehicles were quoted from the suppliers, and the fuel price was taken from the Engen website as of October 2014 (Engen, 2014). Prospecton was on the way to Sezela and there was no significant deviation from the route. Therefore, if at least one constituent was from Sezela then the distance to and from Sezela was used in the transport calculation

The variance in cost to transport SCBF and BA compared to HDPE was because the tipper required more fuel per 100km and had a higher hire cost than the bakkie that was used to transport the HDPE only.

Table 4-30 shows a summary of the transport costs for each mix.

Table 4-30: Summary of total transportation costs

Summary of total transport costs for mixes							
	Control	2,5% HDPE	10% SCBF	5% BA	2,5% HDPE + 5% BA	10% SCBF + 5% BA	2,5% HDPE + 10% SCBF + 5% BA
Cement	R 200,00	R 200,00	R 200,00	R 200,00	R 200,00	R 200,00	R 200,00
Sand	R 226,75	R 226,75	R 226,75	R 226,75	R 226,75	R 226,75	R 226,75
Stone	R 336,04	R 327,64	R 302,44	R 336,04	R 327,64	R 302,44	R 294,04
Waste	R 0,00	R 1 410,20	R 2 814,16	R 2 814,16	R 2 814,16	R 2 814,16	R 2 814,16
Total	R 762,79	R 2 164,59	R 3 543,35	R 3 576,95	R 3 568,55	R 3 543,35	R 3 534,95

The cost to transport the wastes was roughly 4 to 8 times more than the stone and sand transport costs. This nullified any benefit in material cost yielded for the mixes containing BA (shown in section 4.12.1.3).

Construction cost breakdown

Table 4-30 shows a summary of the total construction costs for each mix.

Table 4-31: Summary of total costs

	Summary of total costs for mixes						
	Control	2,5% HDPE	10% SCBF	5% BA	2,5% HDPE + 5% BA	10% SCBF + 5% BA	2,5% HDPE + 10% SCBF + 5% BA
Total material cost	R 63 806,59	R 65 415,16	R 63 992,20	R 62 924,26	R 64 532,84	R 63 109,87	R 64 718,45
Total labour cost	R 34 902,00	R 34 902,00	R 34 902,00	R 34 902,00	R 34 902,00	R 34 902,00	R 34 902,00
Total plant cost	R 14 318,31	R 14 318,31	R 14 318,31	R 14 318,31	R 14 318,31	R 14 318,31	R 14 318,31
Total transport cost	R 762,79	R 2 164,59	R 3 543,35	R 3 576,95	R 3 568,55	R 3 543,35	R 3 534,95
Total cost to construct	R 113 789,69	R 116 800,07	R 116 755,86	R 115 721,52	R 117 321,70	R 115 873,54	R 117 473,71

Figure 4-40 illustrates the breakdown of costs between material, labour, plant and transport costs.

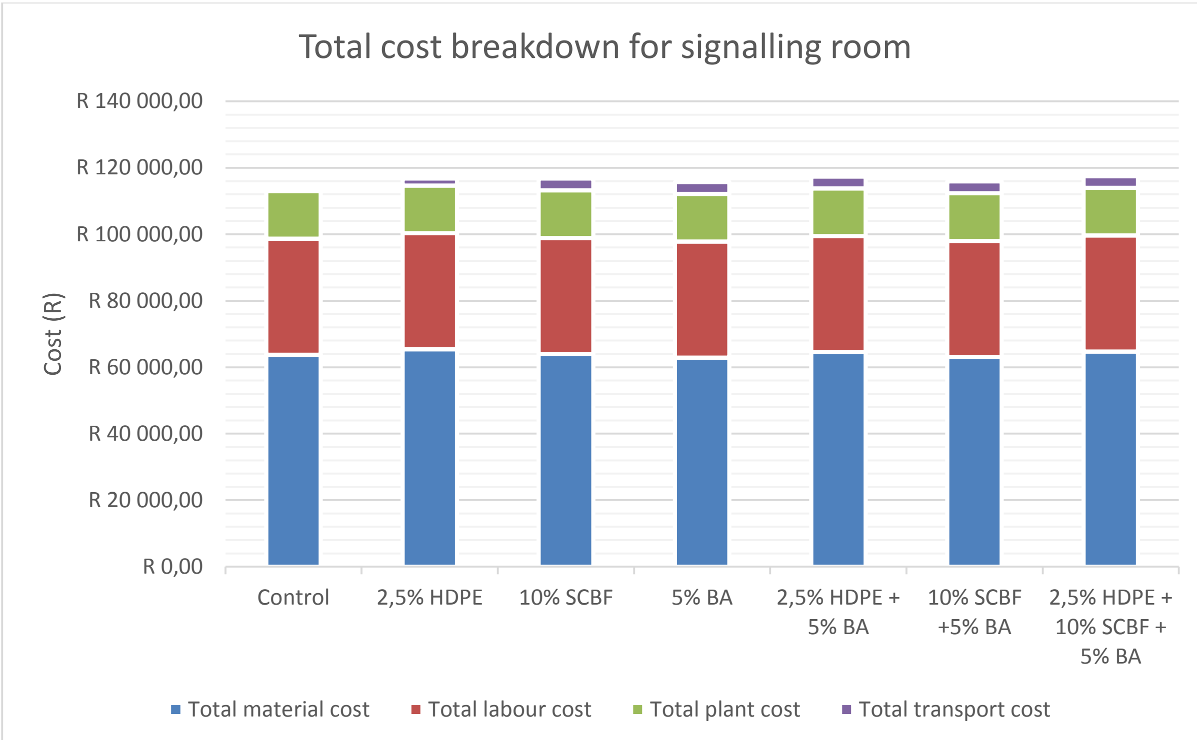


Figure 4-40: Breakdown of signalling room total costs for mixed and optimum waste specimens

The plant and labour cost were constant for all mixes, however, material costs and transportation costs did vary. The material costs contributed the largest proportion of total cost (avg. ±55%), as shown by Figure 4-40, of which 30% on average was purely concrete cost. The total material cost varied from 1,38% less than the control mix for the BA mix, to 2.52% more than the control mix for the HDPE mix, thus giving a range of variance of 3,9% amongst waste mixes. The transportation costs showed a much higher variance when comparing the waste mixes to the control mix. This was because all costs associated with the transportation of waste materials were over and above the transport costs for the conventional mix materials (sand, stone, and cement). Transport costs were subsequently increased by 183,77% for the HDPE mix to between 364,52% and 368,92% for the all the other waste mixes relative to the transport costs for the control mix. Although the variance in transportation costs relative to the conventional mix was much higher than the variance due to material costs, transportation costs only contributed on average 2,85% of the total cost to construct the signalling room. Despite this, the impact of transport costs was still significant. When evaluating the BA mix for example, the cost of materials alone showed decreases in cost relative to the control mix but when the total cost was considered there was an increase in cost relative to the control mix due to the additional transportation cost incurred.

The summary of total costs for each mix shown in Table 4-31 on page 275, indicates that the total cost to construct the signalling room relative to the control mix increased for all waste mixes, albeit within a minimal range of between 1.67% for the BA mix, to 3,14% for the mix consisting of all 3 wastes. The mix with HDPE was the most expensive out of the three individual waste mixes, increasing total cost relative to the control by 2,58% to R 116 800,07. The SCBF mix also increased total cost but to a lesser extent than the HDPE of 2,54%. The changes in the total cost relative to the control were due to a combination of the increased material cost by using SCBF and HDPE as well as the cost to transport the waste materials. Therefore, for all waste mixes, there was no economic “benefit” based on the cost scenario involving the construction of a cast in-situ signalling room at Pinetown station and even if the signalling room was located closer to the mill, the hire cost for the vehicle alone (i.e. no fuel cost) exceeded any material cost reductions gained by using BA.

4.12.2.2. **Pre-cast applications**

The pre-cast scenario was based on precast platform coping blocks and was costed as per the QTO method (Dalton, et al., 2011) . The copings were placed at the train-side edge of the platform to provide a hard-wearing surface for passengers to board and depart the train. At Pinetown there were a total of 83 copings that were required. The total cost to produce all 83 was calculated and this was converted to a price per coping in this study. The precast application was assessed mainly because it was a smaller scale production item than the signalling room in terms of concrete quantity required and hence the comments could be made on whether the use of waste materials have a more significant impact on small scale or larger scale production items.

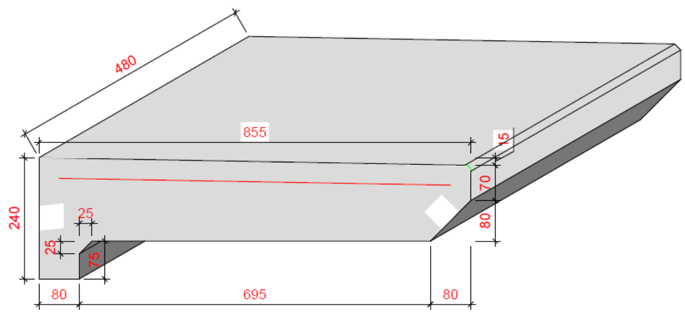


Figure 4-41: Coping slab block

Material's cost

To price the copings, only the concrete and mesh were considered under total material costs. For the concrete costs of the waste mixes and conventional mix, rates were taken from the material cost model in section 4.12.1. To cast the copings, 10 reusable moulds were used at a cost of R3200 each and this cost was split between the 83 moulds. The material costs for the copings are shown in Table 4-32.

Table 4-32: Table of coping material costs

MATERIALS				Control	2,5% HDPE	10% SCBF	5% BA	2,5% HDPE + 5% BA	10% SCBF + 5% BA	2,5% HDPE + 10% SCBF + 5% BA
Item	**Unit**	**Quantity**	**Rate (Exc. VAT)**	**Amount (Excl. VAT)**						
30 MPa concrete	m^3	$0,06m^3$	Refer to mix costing	R 45,37	R 49,35	R 45,83	R 43,19	R 47,17	R 43,65	R 47,63
Mesh ref.311	m^2	$0,35m^2$	$R28,932/m^2$	R 10,13	R 10,13	R 10,13	R 10,13	R 10,13	R 10,13	R 10,13
Moulds#	unit	10	R 3200 *10 /83	R 385,54	R 385,54	R 385,54	R 385,54	R 385,54	R 385,54	R 385,54
Total material cost				R 441,04	R 445,01	R 441,50	R 438,86	R 442,83	R 439,32	R 443,29

#Cost for moulds were taken as the cost for all 83 copings and then divided by 83 to get an idea of the cost per coping.

The cost of concrete contributed between 9, 84% for the BA mix to 11.09% for the HDPE toward the total material cost. This was almost a third of the material cost contribution realised for the signalling room because there was a lesser quantity of concrete required per coping and the majority of costs was made up by the cost of the moulds.

Labour

With the assumption that it would take 4 general workers and a foreman 30 minutes to cast a single coping block, the labour costs were calculated as per Table 4-33. Even though it only took 30min to cast a coping, there were only 10 moulds available. Therefore, the labour was not busy the entire day just casting copings and the total cost for labour was taken as 83 multiplied by the time to cast one coping.

Table 4-33: Labour costs for coping slabs

Designation	Rate (R/h)	Time(hours)	Cost (R)
4 No. General labour	29	0.5	R 14.50x 4 = R 58
1 No. Foreman	45	0.5	R 22.5
Cost per coping			R 80.5 / coping
Total cost for 83 copings			R6681.50

Plant & hire

The precast yard set-up was separate to the signalling room hence the need for a separate site container to be established and Skil saw to be purchased. The cost for the mixer and drive unit was calculated by assuming that 10 moulds were cast in a day and the total duration for all 83 moulds would be 9 days.

Table 4-34: Plant and hire costs for copings

Plant & tools	Rate	Time	Cost (R)
230 mm Skil saw	R 1920 (purchase)	-	R 1920 (purchase)
Concrete mixer	R 210 /day	9 days	R 1890
Drive unit & needle	R 190/day	9 days	R 1710
Site Container incl. handling & hire)	R 2840	-	R 2840
Consumable/Miscellaneous	R200	-	R 200
Total			R 8560
Total/ mould = total /83			R 103.13

Transport

In total, 83 copings were needed which equated to 5.4 m^3 of concrete. The transport cost rate (R/m^3) for stone and sand, as with signalling room, was taken as the difference between the delivered and undelivered cost of sand and stone to produce a cubic metre of concrete for each mix. The difference was then multiplied by the amount of cubic metres of concrete required for the copings, to calculate transport cost for stone and sand. (See Table 4-35)

Table 4-35: Transport cost for stone and sand (Copings)

Transport cost for sand & stone (Pre-cast coping scenario)								
	Control		2,5% HDPE		5% BA		10% SCBF	
	Delivered - undelivered R /m^3 (of concrete)	Cost (R)	Delivered – undelivered (R) /m^3 (of concrete)	Cost (R)	Delivered - undelivered (R) /m^3 (of concrete)	Cost (R)	Delivered - undelivered (R) /m^3 (of concrete)	Cost (R)
Sand	8,67	R 46,84	8,67	R 46,84	8,67	R 46,84	8,67	R 46,84
Stone	12,86	R 69,42	12,53	R 67,68	12,86	R 69,42	11,57	R 62,48
	10% SCBF + 5% BA		2,5% HDPE + 5% BA		10% SCBF + 2,5% HDPE + 5% BA			
	Delivered - undelivered (R) /m^3 (of concrete)	Cost (R)	Delivered - undelivered (R) /m^3 (of concrete)	Cost (R)	Delivered - undelivered (R) /m^3 (of concrete)	Cost (R)		
Sand	8,67	R 46,84	8,67	R 46,84	8,67	R 46,84		
Stone	11,57	R 62,48	12,53	R 67,68	11,25	R 60,74		

The waste material transport costs for the copings worked out to be the same as those used for the signalling room in Table 4-29 on page 273 as it required the same amount of vehicle loads.

Summary of construction costs per coping

The cost per coping was used as it allowed for a general coping cost rate for comparative purposes. It also allowed for a comparison between a relatively small item (coping) and a relatively large item (signalling room).

Table 4-36: Summary of construction costs per coping

Summary of costs per coping for mixes							
	Control	2,5% HDPE	10% SCBF	5% BA	2,5% HDPE + 5% BA	10% SCBF + 5% BA	2,5% HDPE + 10% SCBF + 5% BA
Total material cost	R 441,04	R 445,01	R 441,50	R 438,86	R 442,83	R 439,32	R 443,29
Total labour cost	R 80,50	R 80,50	R 80,50	R 80,50	R 80,50	R 80,50	R 80,50
Total plant cost	R 103,13	R 103,13	R 103,13	R 103,13	R 103,13	R 103,13	R 103,13
Total transport cost	R 3,81	R 20,78	R 37,63	R 37,72	R 37,69	R 37,63	R 37,61
Total cost to construct	R 628,48	R 649,43	R 662,76	R 660,21	R 664,16	R 660,58	R 664,54

Figure 4-42 shows the breakdown in costs to construct a single coping.

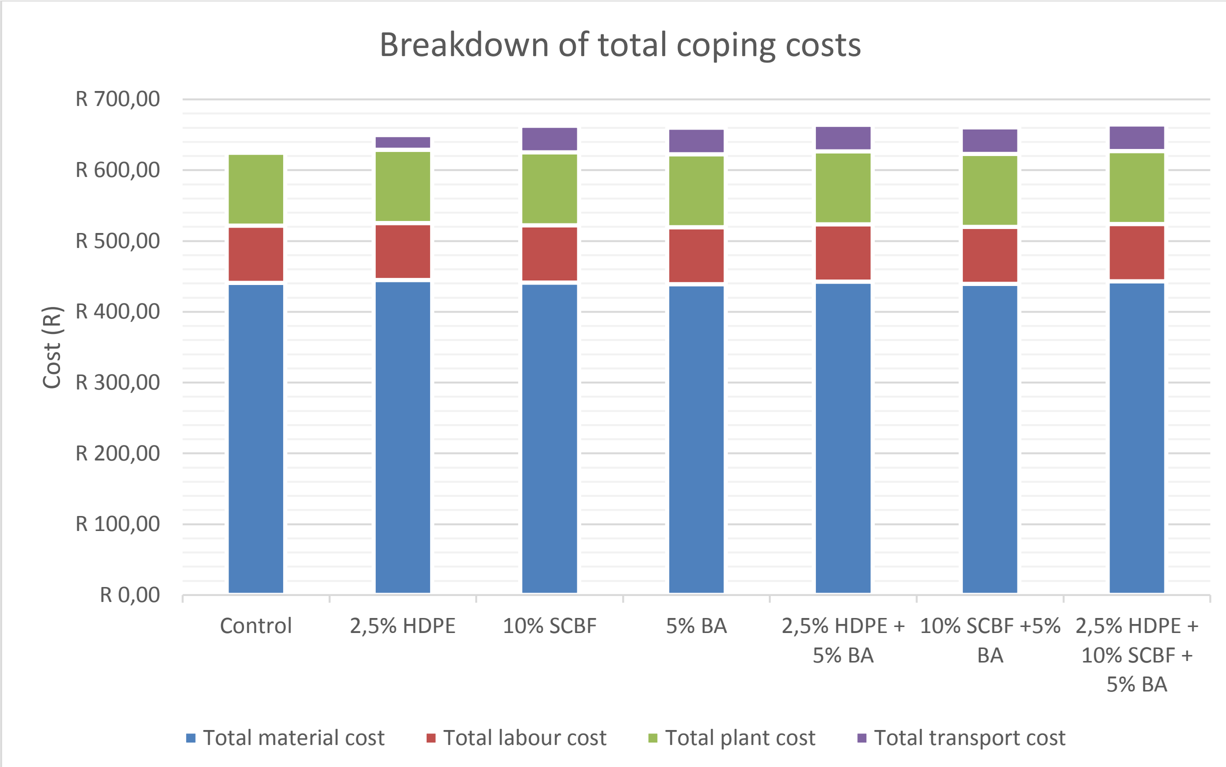


Figure 4-42: Breakdown of coping total costs for mixed and optimum waste specimens

The plant and labour cost were constant for all mixes, however, material costs and transportation costs did vary. With reference to Figure 4-42, the material costs contributed a larger proportion of the construction cost per coping (avg. ±67%) than the signalling room (avg. +-55% - Figure 4-40). This was because there was significantly less labour required to construct the copings than the signalling room.

The total material cost varied from 0,49% less than the control mix for the BA mix, to 0,9% more than the control mix for the HDPE mix. The variance in material cost per coping, was less than a percent and hence the choice of mix used (waste/control), despite forming the largest portion of the total costs, did not have a significant impact. The transportation costs were the same as those used for the signalling room but formed a larger portion of the total construction costs for the copings (avg.5,27%) in comparison to the signalling room (avg.2,85%).

The summary of total costs for each mix shown in Table 4-36 on page 281, indicated that the total cost to construct the copings relative to the control mix increased for all waste mixes but, to a slightly larger extent than the signalling room of between 3,33% (HDPE) to 5,74% (3 waste mix)

The changes in the total cost were due to a combination of the increased material cost by using SCBF and HDPE as well as the cost to transport the waste materials. Therefore, as with the signalling room, for all waste mixes, there was no economic "benefit" based on the cost scenario at Pinetown station and even if the site was located closer to the mill, the hire cost for the vehicles alone (i.e. no fuel cost) exceeded any material cost reductions gained by using BA.

4.12.3. Concluding remarks on economic analysis

In terms of the individual waste mixes costs, HDPE and SCBF increased the cost of the mix relative to the control by 8,76% to R763.90/m^3 and by 1% to R709,47/ m^3 respectively. The BA mix lowered cost relative to the control by 4,81% to R 668.62/ m^3 because cement, which was the most expensive conventional constituent and contributed the most to the cost of the mix, was replaced with BA which was essentially free. The addition of BA to the individual mixes did lower the cost of the HDPE+ BA and SCBF + BA mixes by 4,41% and 4,75% respectively, relative to the individual waste mixes. However relative to the control mix, the HDPE+BA mix and mix with all three waste were still 3,95 % and 4,97% more costly than the control mix respectively. This indicated that the cost reductions due to BA were not significant enough to offset the cost increases due to HDPE.

To conclude on the scenario analyses. Concrete costs contributed more towards the total material cost for the signalling room than the coping. For both the signalling room and the coping scenarios there were increases in the total material costs relative to the conventional mix. The material cost of the signalling room increased by a maximum of 2,52%, up from R63 806.59 (control) to R 65 415.16 for the HDPE mix. The largest reduction in material cost was noticed for the BA mix, which was 1,38% less than the control. The SCBF mix increased material cost, as with the HDPE mix but by a lesser extent of 0.29% relative to the control mix. The SCBF + BA, HDPE +BA and HDPE + BA + SCBF mixes showed a 1,09% decrease, 1,14% increase and 1,43% increase respectively in material cost relative to the control mix. Indicating that the addition of BA was significant enough to reduce the material cost of using SCBF but not enough to offset the cost of HDPE. The copings showed similar trends for the materials costs relative to the control mix but to a much lesser extent. The HDPE, SCBF and BA mixes showed a 0,9% increase, 0,10% increase and a 0,49% decrease in material cost relative to the control mix. For the mixed waste combinations, the HDPE+BA, SCBF+BA and SCBF+HDPE+BA mixes showed 0,41% increase, 0,39% decrease and 0,51% increase in material costs respectively. The material cost percentage change for the copings was roughly a third of the percentage material cost changes for the signalling rooms. This indicated the use of waste material was more significant in terms of changes to material costs as the amount of concrete required increased.

With regard to the total construction costs for the signalling rooms and the copings, all mixes showed increases in the respective costs relative to the control. For the signalling room, the HDPE, SCBF and BA mixes showed 2,58%, 2,54% and 1,67% increases in total cost respectively. For the mixed waste combinations, HDPE+BA, SCBF+BA and SCBF+HDPE+BA mixes increased the total cost by 3,01%, 1,8% and 3,14% respectively ,relative to the control mix. For the copings, the HDPE, SCBF and BA mixes showed 3,33%, 5,45% and 5,05% increases respectively. For the mixed waste combinations, HDPE +BA, SCBF +BA and SCBF+HDPE + BA mixes increased the total cost by 5,68%, 5,11% and 5,74% respectively ,relative to the control mix. This indicated that even though the change in material costs was significantly less for the copings than the signalling rooms, when the total cost was considered, the percentage increases relative to the control mix was more for the copings than for signalling rooms. This was because the transport costs remained constant for both the copings and the signalling room. Even though the transport cost was split between the 83 copings, it was because the transport costs were increased considerably relative to the control mix that made them significant. The mixes containing SCBF and/or BA required the tipper trip to Sezela and this was significantly more than the transport costs for HDPE. It was for that reason that the SCBF and BA mixes were more than the HDPE mix despite HDPE having a higher material cost.

It can be said that even though the material cost of concrete can be reduced by using mixes containing BA, when the BA concrete is applied to a scenario and the total cost is considered, the change in cost is not enough to offset the transport cost required for the waste materials. Based on the scenario analyses carried out in this study, there was also no feasible proximity around the mill and the recycling factory that the BA cost reduction would offset the hire costs to transport the waste materials. Therefore, the use of waste materials (HDPE,SCBF,BA) will not yield any economic benefit whether they are used for small scale (0.06m^3 concrete/coping) precast works or larger scale cast in-situ equipment rooms (26,1m^3 concrete) regardless of where the site is located in the context of this study. Although, the impact on cost by using waste materials are less significant the larger the project. So if the reduction in cost change showed a linear relationship, then a project requiring approximately 435 times more concrete than the signalling room can possibly reduce the increase in cost to less than a percent, if transport loads for the waste materials remained constant. Therefore from an economic perspective, the cost increases for the copings and signalling room meant that the waste mixes were not viable if the project was profit-centred. However, if the project was required to incorporate corporate social responsibility (CSR) measures or there was the possibility of government environmental incentives, then the environmental benefits of using the waste materials such as reducing virgin mineral consumption and pollution, may warrant the higher costs thus making the waste mixes potentially viable in this context from an economic perspective.

4.13. Chapter summary

This chapter discussed findings from this study that fulfilled the research objectives set out in section 1.5 as well as met the research aim in stated in section 1.4, by following the research programme explained in chapter 3.

This study assessed the effect of varying waste proportions and the general behaviour of the selected waste materials in terms of hardened density, workability, hardened strength properties and specific heat. The findings are summarised as follows:

4.13.1. Summary of workability

Workability was tested for volumetric substitutions of 2,5%, 5%, 10%, 20% and 40% for each waste mix. It was noticed that the proportions of waste as well as the physical properties of the wastes, in particular the absorbency and surface properties, did have an impact on the workability of the concrete. This confirmed findings by (Weymouth, 1933). All waste specimens were within the reasonable workability ranges of 40 mm to 150 mm prescribed for concrete use. However, the SCBF and BA at higher volume substitutions would not be recommended for thin sections or members with closely packed reinforcing.

The HDPE mixes showed an increase in workability with increasing proportions of HDPE pellets in the mix from 81 mm at 2.5 % HDPE substitution, to 140 mm at 40% HDPE substitution. This behaviour was attributed to the hydrophobic nature and smooth surface of the HDPE pellets having poor adhesion with the cement paste thus reducing interlocking resistance (Weymouth, 1933) which confirmed findings by Ahmad, *et al*. (2008) and Ferreira, *et al*. (2012).

The SCBF mixes showed greatest reductions in workability out of the three materials, with increasing proportions of SCBF decreasing workability relative to the control by 12% to 65 mm at 5% substitution up to 55 mm at 40 % substitution. The lower workability was attributed to the absorption of the fibres which decreased the fluidity of the mix, as well as the interlocking and balling between long fibres as per Sivarja, *et al*. (2010).These findings confirmed similar findings on the effect of fibres on workability by Jorillo & Shimizu (1992), Silva & Suely (1984) and Vandewalle (2006).

The BA mixes had the same slump as the control at 2.5 % volumetric substitution but increased by up to 12.55% to 65 mm at 40% volumetric substitution. The porous nature of BA when compared to cement particles was possibly why there was the general decrease in slump, as moisture was absorbed hence reducing the fluidity of the mix.

When the wastes were mixed there were slight reductions of 1.5% and 2,5% from the respective individual SCBF and HDPE mixes respectively. This was possibly due to the absorbency properties of BA and SCBF combined, or the possibility of a pozzolanic reaction taking place in the HDPE + BA mix.

4.13.2. Summary of density

The hardened density of specimens was tested at 7 and 28 days for volumetric substitutions of 2,5%, 5%, 10%, 20% and 40% for each waste mix. The findings on the density behaviour of the waste concrete mixes showed that all mixes had reductions in density and the effect on concrete density due to aggregate variations was more significant than cement substitution. This supported the idea that aggregates have the greater influence on concrete density because they constitute 65-75% (MAST, 2014) of the concrete mix. Generally, the densities of the mixes increased after 28 days when compared to the respective mix density at 7 days due to the hydration reaction closing voids in the mix. However, the effect on concrete density due to the degree of volumetric waste substitution and the waste material type was greater than the effect of hydration from 7 to 28 days.

HDPE showed the largest reductions in density at 28 days out of the three waste materials and ranged from 0,06% at 2,5% substitution to 10,4% at 40% substitution. This was due to the hydrophobic nature of HDPE which caused poor bonding with the cement mix, repelled water in the interfacial zone and thus increased the amount of voids and subsequently, density.

The density of BA mixes varied from 0.25% to 3.1% at 2.5% and 40% substitutions respectively. This was very close to the control mix density indicating that BA had little effect on the density of the mix. The slight decrease could have been due to variations in the composition of cement used or the lack of pozzolanic reaction to replace the cement's hydration compounds.

The density of SCBF mixes varied from 0.13% to 4.73% at 2.5% and 40% substitutions respectively. The reductions in density were minimal and could have either been attributed to variations in the cement composition used, the lighter mass of the SCBF compared to the stone it replaced or additional voids being introduced to the mix.

The effect of adding BA to another waste generally reduced the density of the concrete mix. The mix containing HDPE+SCBF+BA had the lowest density reduction of 5% relative to the control mix. This was due to the additional pores introduced to the mix, lighter weight waste aggregates and the lack of significant pozzolanic reaction to block pores.

The density results could be related to properties such as strength and durability, due to implied effect on pore structure.

4.13.3. Summary of hardened strength properties

The hardened strength properties (Compressive, flexure and splitting) were tested using volumetric substitutions of 2,5%, 5%, 10%, 20% and 40% for each waste mix. All mixes using waste at natural moisture state showed decreases in compressive strength when compared to the conventional mix. This confirmed findings by (Okajima, 1972) that stated, if the aggregate has a modulus of elasticity lower than the matrix, compressive strength is decreased due to lowered resistance to micro-cracking. Therefore, the critical/optimum volume was not the volume at which there was a "peak" in strength, but rather the substitution before there was a significant decline in strength relative to the control mix.

HDPE showed a decline in compressive strength with increasing substitutions of waste. The optimum/critical volume percentage that yielded the least reduction in compressive strength of 29.95 MPa was at 2,5%. The reduction in strength was because HDPE had a hydrophobic nature which caused a film of water to form in the interfacial zone thus increasing the W/C in the zone, increasing porosity and decreasing compressive strength. This expanded on research by Rahman *et al.* (2012), Suganthy, *et al*. (2013) and Al-Manaseer & Dalal (1997) by broadening their findings on compressive strength properties for concrete containing plastic, to include low-volume substitutions. In flexure, HDPE also showed a decline in strengths with increasing proportions of waste. The least reduction in strength was 5.06 MPa at 2,5% substitution. However, HDPE resulted in the highest flexural strength properties out of the three waste materials, possibly due to its particle packing benefits. Findings in this study on HDPE in terms of flexure, confirmed and expanded on past research by Ismail & AL-Hashmi (2007). In terms of splitting strength the HDPE mixes decreased in strength with increasing proportions of waste. The least reduction was 10% at 2,5% substitution. The decreases in splitting strengths for HDPE mixes may be attributed to the weakened aggregate-cement interfaces falling within the applied tension plane. Findings on splitting strength correlated with past research by Ferreira, *et al*. (2012). Based on findings from his study, there was no benefit to using the HDPE at natural moisture state in terms of compressive, flexural or splitting strength.

SCBF in general, had the highest compressive strengths out of the three waste materials. SCBF may have decreased the W/C due to the absorption of water from the mix thus increasing compressive strength in the interfacial zone. This reduced the loss in strength incurred by adding a weaker material in place of stone that possibly induced voids. The optimum/critical volume percentage of SCBF was at 10% and yielded a compressive strength of 32.16 MPa after which strength rapidly declined. The strength at 2,5% substitution was still however less than the control. This confirmed and expanded on research by Racines & Pama (1978) by showing that low-volume substitutions of natural fibres exhibited a similar behaviour to high-volume substitutions. In terms of flexural strength, which declined with increasing volumes of waste, the least reduction in strength relative the control was 4.68 MPa (SCBF) at 2,5% substitution. The reduction in flexural strength was possibly due to the inherent tensile properties and fibre length of SCBF, as well as balling which may have increased voids in the mix thus reducing strength results as per past findings by Wang, *et al*. (2000) and Ramirez-Correti (1992). The bagasse fibres may have also softened after being exposed to the alkaline pore water thus decreasing their resistance to flexural load. SCBF however, increased splitting strength for the 2.5% and 5% volumetric substitutions. The 5 % substitution was where the peak splitting strength value occurred before performance started to decrease. This indicated that SCBF offered more resistance to shearing than flexural load by improving splitting strength by 15,4% relative to the control mix. These findings expanded on research by Sivarja *et al.* (2010) by stating that splitting strength benefits for SCBF are not only limited to substitutions of 1,5% but continue until 10% volumetric substitutions. Based on findings from his study, the only benefit to using the SCBF at natural moisture state was in splitting strength.

BA had the lowest compressive strength at high volume substitutions out of the three waste material mixes. This was due to the lack of cement to form C-S-H as well as the lack of pozzolanic reaction taking place at 28 days thus having a negative impact on compressive, flexural and splitting strengths. The optimum/critical volume percentage was at 5% and yielded a compressive strength of 30,39 MPa. These findings confirmed those by Kurama & Kaya (2007), which stated that significant strength benefits are generally not realised at curing ages prior to and including 28 days when using BA. The BA mixes were also the worst performing of the three wastes in terms of flexural strength. The least reduction of 4.59 MPa was noticed at 2,5% substitution. These findings correlated with Aggarwal, *et al*. (2007), who showed decreases in flexural performance at low-volume substitutions but, contrasted findings by Kurama & Kaya (2007) which showed flexural strength benefits at 5% substitution. In terms of splitting strength, BA mixes declined in strength with increasing waste proportions. The least reduction was yielded at 5% substitution and was a 13,7% decrease in splitting strength relative to the control mix. Based on findings from this study, there was no benefit to using the BA at natural moisture state in terms of compressive, flexural or splitting strength.

None of the waste mixes met the target strength of 35 MPa, therefore concrete with waste aggregates (HDPE, SCBF, BA) at natural moisture state, were not viable for structural applications based on this study because of the variation to the pre-determined strength. The reason was because of the unpredictability in the strength yielded. Therefore, with further research into developing a specification guideline for waste concrete mix design to yield a predetermined concrete strength based on a calculated mix proportions, there was future potential for the waste mixes to be used for structural applications as they were classified as high-strength (30 MPa) concrete based on the compressive strength results from this study.

Mixing waste materials was not viable in terms of compressive strength, flexural strength and splitting strength, as all mixes showed declines in strengths from that of the individual waste mixes.

In terms of the relationships between strength properties, the 28 day flexural strengths were generally 15% of 28 day compressive strength for HDPE and BA, and 17% for SCBF. The splitting strengths were generally ±10% of 28 day compressive strength and ±60% of 28 day flexural strength for all mixes. The flexural strengths were higher than the splitting strength results because as mentioned in section 2.3.5.3, the beam specimen offered some compressive resistance when under flexural load as per Oregon State University (2014).

In terms of compressive strength development, the control developed 60% of 28 day compressive strength after 7 days. The HDPE and SCBF specimens developed ± 60% of 28 days compressive strength after 7 days, and BA developed approximately 70% of its 28 day compressive strength after 7 days. This showed that HDPE and SCBF, which were aggregate substitutions, did not change the rate of compressive strength development when compared to the control whereas the BA mix developed 28 days strength 10% faster. This was because the 28 day strength was the lowest due to the removal of cement from the mix and hence the change in strength after 7 days was minimal. This indicated that the rates and degree of compressive strength development was more dependent on cement hydration than the aggregates used.

In terms of flexural strength development, the control developed 78% of 28 day flexural strength after 7 days. The HDPE sample developed an average of ± 73 % of 28 day flexural strength after 7 days, the SCBF developed an average of ± 87% of 28 day flexural strength after 7 days and the BA developed an average of ± 77% of 28 day flexural strength after 7 days. This indicated that the type of material used had an effect on the rate of flexural strength development. HDPE developed a lesser percentage of the 28 day strength after 7 days, and SCBF developed more of its 28 day strength after 7 days when compared to the control. The BA mix remained relatively consistent only varying by 1% with the control in terms of 28 day strength gain after 7 days.

The HDPE + BA , SCBF + BA and HDPE+ SCBF + BA mixed waste specimens developed an average of 64% ,56% and 58% of 28 day compressive strength after 7 days respectively. In terms of flexural strength 77% , 80% and 77% of 28 day flexural strength was realised after 7 days for the HDPE + BA , SCBF + BA and HDPE+ SCBF + BA mixes respectively. The compressive strength gain for the waste mixes were more than the control because the 28 day strength was less due to waste aggregates used, but the rate of hydration of cement remained constant. The rate of flexural strength also developed faster than compressive strength after 7 days just like the individual waste mixes and the degree of strength gain for the mixed waste specimens were similar to the individual waste specimen rates they were comprised of.

In terms of the relationship with workability, the natural state compressive strengths of SCBF and BA decreased with decreasing workability. This meant that reductions in workability were likely due to the shape and surface texture of the waste materials. If reductions in workability were due to moisture absorption then there should have been a degree of strength gain as the W/C ratio in the interfacial zone would have decreased and this was not noticed. Alternatively, the voids induced by the waste material had a more significant impact on reducing strength than the benefits yielded from a possible decrease in W/C ratio in the interfacial zone. The natural state HDPE mix, showed an inverse relationship with workability indicating additional free water was present in the interfacial zone thus decreasing strength due to additional void content.

In relation to density, there was a direct correlation for all three wastes because as density decreased so did compressive strength. This emphasised the possibility of additional voids being introduced to the mix thus reducing strength.

4.13.4. Summary of specific heat properties

Findings confirmed those by Al Faruq, *et al*. (2013), on the influence of varying types of materials on specific heat.

The use of SCBF reduced specific heat relative to the control mix from a 4% decrease of 847.839J/kg°C at 2.5% substitution to an 18% decrease of 722.9J/kg°C at 5% substitution, after which specific heat remained relatively consistent. The specific heat sample containing SCBF and respective density results were directly proportional because as density decreased relative to the control sample, so did the specific heat of the bagasse samples. This confirmed findings by Holiday (2013), on the proportional relationship between density and specific heat. The lower specific heat however, would not be of any benefit in terms of improving passive heating/cooling measures.

The specific heat of the BA mixes did not vary significantly from the control and ranged from 0,32% less at 2,5% substitution to 7% more at 5% substitution, after which it remained relatively consistent. This meant that cement could be partially replaced with BA and the effect on the thermal mass of the building would be almost negligible in comparison to the control mix. Differences in specific heat may have been attributed to chemical variations in the cement or the porous and unreactive nature of the BA used.

The HPDE mix results showed that as the proportions of HDPE doubled, the specific heat of the concrete increased by an average of 10%. The peak value of 1272.88 J/kg°C was found at 40% substitution and was a 44% increase relative to the control. The increase in specific heat relative to the control was because the specific heat of the HDPE used was significantly higher than that of the stone it replaced. This statement confirmed findings by Elzafraney *et al.(*2005) who showed an increase in thermal mass, which is related to specific heat of concrete, by adding plastic aggregates. The density of HDPE had an inverse relationship to specific heat. HDPE in concrete would be beneficial in terms of passive heating and cooling for non–structural insulation.

When wastes were mixed the specific heat was higher due to a combination of the individual waste specific heat properties and the possible pore blocking pozzolanic effect of the BA.

This study further evaluated static elastic modulus, durability, SEM analysis, effects of moisture and economic analysis to expand knowledge on the behaviour of the waste materials. The findings are summarised as follows:

4.13.5. Summary of static elastic modulus

The elastic modulus properties of each mix were affected differently based on the type of waste material used, as per Kaplan (1959). All mixes reduced elastic modulus except for the HDPE mix relative to the control mix. The HDPE showed a higher elastic modulus probably because 2,5% substitution, which was the optimum/critical HDPE volumetric substitution, may not have been sufficient to impose any negative effects on the concrete elastic modulus but was enough to bridge cracks and stiffen the mix. Another reason would be that at 2,5% substitution the effect of HDPE was negligible and it was a variation in cement composition or error in strain gauge fixity that affected results.

The SCBF mix decreased relative to the control because the SCBF mixes may have softened under exposure to the alkaline cement matrix thus decreasing the stiffness of the fibres and hence the stiffness of the concrete mix. This confirmed findings by Lamond & Pielert (2006) and Bentur & Mindess (1990) which stated that low-modulus aggregates were likely to reduce the elastic modulus of the concrete mix. Sivarja, *et al*. (2010), showed increases in the elastic modulus at 1,5% volume substitution indicating that 1,5% was a limiting value in terms of the effect on elastic modulus performance of SCBF specimens.

The BA mix by itself at optimum/critical volume substitution (5%), reduced the elastic modulus relative to the control mix. This was because the BA used was shown to be unreactive based on the lack of beneficial strength development, and hence reduced load transfer between the aggregates and the paste matrix due to poor bond characteristics. The negative effect of adding BA to the mix was emphasized when BA was added to the other waste mixes which subsequently decreased the stiffness of the respective mixes further.

In terms of elastic modulus, waste materials which have a low elastic modulus or are unreactive, result in a low concrete elastic modulus. However, any negative effect of HDPE at low-substitution is negligible.

The predicted values of elastic modulus corresponded well with the HDPE results emphasising the increase in elastic modulus was more likely due to changing cement composition as opposed to the HDPE pellets. However, the predictions for all the other waste mixes were not accurate and this creates an opportunity for future research to develop a prediction formula for waste concrete mixes.

4.13.6. Summary of durability

Each durability test showed the effects of different aspects that changed within the waste mixes in comparison to the control mix. The OPI and WS tests showed the closest correlations to pore structure properties whereas CC was more dependent on the resistivity of the waste material.

The HDPE mix was more permeable and showed a greater degree of water ingress through capillary action than the control but could still be classified as having “good” durability properties in terms of WS. However, it was classified as having “poor” durability properties in terms of OPI. In terms of CC it was classified as having good durability properties and improved CC relative to the control because of it’s high resistivity properties. The increase in permeability indicated by the WS and OPI tests validated the possibility that more voids were introduced to the mix which had influence on properties such as density and strength. The HDPE mix may be affected by ingress of deleterious substances under external pressure but would not be affected by WS and CC, the former being critical durability criterion as per Hycrete (2011). Therefore, the HDPE mix can still have the potential to be viable in terms of durability as the OPI can be mitigated by decreasing the W/C ratio as per Hycrete (2011).

The SCBF mix was more permeable than the control but improved CC and WS. This showed that additional voids were added to the mix which increased the OPI but the tortuous fibres disrupted flow paths used for capillary action and thus lowered the WS. The resistivity properties of the SCBF contributed to the lower CC relative to the control mix. The SCBF mix was classified as having good durability properties based on all three tests done and hence viability was not compromised in terms of durability in the context of this study. SCBF would however need to be tested for the possibility of long-term fibre deterioration as part of future research, as this may also potentially affect concrete durability.

The 5% BA mix was classified as good in terms of OPI and CC as it decreased permeability and lowered CC relative to the control mix. This was because of the possible pozzolanic behaviour that may have created a pore filler effect for the durability specimens. The BA mix was also classified as having “good” WS properties even though it decreased water sorptivity relative to the control. Therefore, the possible pore filler effect was noticeable in terms of OPI and CC but was not significant enough to yield any benefits in terms of WS. The reason for this may have because there were pockets of unreactive BA that caused additional capillary voids to form for the WS sample specimen, thus nullifying any potential pozzolanic pore filler benefits. The viability of the BA mix was not compromised in terms of durability, based on the durability index results, and improved permeation and chloride diffusion resistance relative to the control mix

The mixing of wastes was not advised because for OPI, CC and WS for all mixed waste combinations were generally worse off than the respective individual waste mixes. However, the BA+ HDPE was classified as having good durability properties for all three tests. Therefore, the durability properties of the SCBF and BA individual waste mixes, and the BA+ HDPE mix did not compromise their potential viability based on results from this study. The mix containing all three wastes and the SCBF + BA mix however, were not viable in wet environments susceptible to ingress of deleterious substances.

4.13.7. Summary of SEM imaging

The SEM images indicated the distinct lack of bonding between HDPE and cement commented on throughout the study. They also showed that SCBF had reasonable bond with the C-S-H, however there were noticeable voids near the fibre-paste interface. The information regarding the increase in voids may explain the reduced strength for the waste mixes compared to the control mix.

4.13.8. Summary of moisture properties

The workability and compressive strength was tested using dry waste aggregates as opposed to natural slate aggregates to investigate the effects on moisture state of the mix.

The HDPE, SCBF and BA mixes increased in compressive strength relative to the natural moisture sate specimens by; 4,85%, 24,41% and 24,37% to 31,4MPa, 40,1 MPa and 37,8 MPa respectively. The workability of the HDPE increased by 1,2%, the SCBF decreased by 12,5% and the BA mix decreased by 5,7% also in relation to the natural state mix results..

The reason for the changes in compressive strength and workability were due to the increase in absorption potential of dried waste aggregates which meant that more free water was drawn from the mix in the interfacial transition zone but not to the extent that hydration was compromised. Hence, decreasing the W/C ratio, reducing the amount of capillary pores in the mix and increasing compressive strength. The absorption of moisture is also what decreased the workability of the respective mixes. The SCBF which was the most absorbent waste material, yielded the most significant changes in strength and workability. In the case of HDPE, the workability increased by 1,2% relative to the natural state sample and was probably attributed to variances in the cement composition because HDPE is not absorbent.

The increase in strengths for all mixes except for the HDPE mix, were enough to meet the target strength of 35 MPa thus rendering them potentially viable for structural use. However, as stated when commenting on the natural state waste aggregate compressive strength results because the HDPE mix still met the 30 MPa high–strength concrete classification, with further research into producing a waste concrete specification for HDPE to reach a pre-determined target strength it still had the future potential to be used for structural applications.

For the SCBF + BA, HDPE + BA and SCBF + HDPE + BA mixed waste substitutions, compressive strength relative to the natural moisture sate specimens increased by 75%, 38% and 58 % to 37,5 MPa , 35,6 MPa and 35,9 MPa respectively. However, as with the individual wastes there were decreases in workability of 27,03% and 5,4% for the SCBF + BA and HDPE + BA mixes respectively, relative to the control . The changes in strength and workability relative to the natural moisture state specimens were due to the compounding effects of the waste constituents used. All mixed waste samples were above the target strength and hence were viable for structural use.

4.13.9. Summary of economic analysis

The use of BA reduced the cost of concrete per cubic metre by % relative to the control mix. The use HDPE and SCBF increased the cost of concrete per cubic metre by 8.76% and 1,01% respectively relative to the control mix. The SCBF + BA mix decreased cost per cubic metre by 3,79% and the HDPE + BA mix decreased cost per cubic metre by 3,95% relative to the control mix. However, the SCBF + HDPE + BA mixed waste combination was 4,97 % more expensive than the control mix per cubic metre of concrete due to the HDPE cost. The reductions in cost noticed for mixes containing BA was because BA was essentially a free material and partially replaced the most costly conventional constituent being cement.

The reduction in cost by using BA was not enough to offset the transport cost required for the waste material when considering the total construction costs. When the mixes were applied to costing scenarios, all total construction costs were more than that of the scenarios using conventional concrete. The increases in costs ranged from 1,67% to 3,14% for the signalling room and 3,33% to the 5,74% for the coping scenario. The transport costs were high because of the distance required to obtain the SCBF and BA samples. Based on the quantities of materials used in the scenario analyses carried out for this study, there was no feasible proximity around the mill and the recycling factory where the BA cost reduction would offset the hire costs to transport the waste materials. Therefore, the use of waste material will not yield any economic benefit whether they are used for small scale (0.06m^3 concrete/coping) precast works or larger scale cast in-situ equipment rooms (26,1m ^{3} concrete) regardless of where the site is located, in the context of this study. This meant that projects incorporating corporate social responsibility measures or had the possibility of government environmental incentives were the most likely markets for the potential use of the optimum/critical volume waste mixes in this study.

Inter-leading paragraph

Based on the findings from the testing results carried out in this study it was possible to draw conclusions on the potential viability of the selected waste materials. As stated in the research aim in section 1.4, the viability of the materials depended on their performance in strength, workability, durability and cost. A summary of the findings are as follows.

4.13.10. Summary on viability criterion

To assess the viability of the mixes, the dry state aggregate mixes were considered as these met the target strength and were still potentially viable for structural use unlike the mixes using natural state aggregates. To better illustrate the evaluation of the potential viability of the waste mixes, a comparison table (Table 4-37) was used which showed the rank order of the viability properties tested. Workability was either acceptable or not and hence a simple green or red block indicating acceptance or rejection was used with no rank allocation. The lower the score, the more viable the mix was.

Table 4-37: Viability table

Property	Control	2,50% HDPE	5% BA	10% SCBF	HDPE + BA	SCBF + BA	HDPE + SBF + BA
Workability	-						
7-day Compressive	1	7	2	3	4	5	6
28-Day Compressive	1	7	3	2	4	6	5
7-day flexure	1	2	5	3	4	6	7
28-day flexure	1	2	4	3	5	6	7
Splitting	1	3	4	2	4	7	6
OPI	2	5	1	4	3	6	7
CC	5	3	1	2	4	6	7
WS	4	6	5	1	7	2	3
Cost (signal room)	1	5	2	4	6	3	7
Cost (copings)	1	2	3	5	6	4	7
Totals	18	42	30	29	47	51	62
Best option			Control				

Table 4-37 ranked the performance of each viability property using results from this study. Although workability was not compromised by using the waste materials, it was found that the waste concrete mixes were outperformed by the conventional concrete mix in all respects with regards to viability criterion, except for durability in the cases of BA and SCBF. The HDPE mix was still classified as having good overall durability but did not improve durability over the control. The SCBF and BA individual waste mixes were also the most viable out of the waste mixes tested as shown by the points scored relative to the other waste mixes. The mixed waste combinations were deemed unfeasible as they did not grant any benefit over the individual waste mixes.

Therefore, if the project was profit-centred, none of the waste mixes were viable as they all cost more than the control without granting any significant benefits apart from improved WS (SCBF) and CC & OPI (BA). However, if the project was focused on incorporating corporate social responsibility measures, or there was the possibility of government environmental incentives, then the use of BA and SCBF was potentially viable for structural applications when used with dry-state aggregate (if the mix design was not modified to factor in the moisture properties of the waste materials). HDPE was also potentially viable if corporate social responsibility and environmental incentives were considered, although only for non-structural applications as it did not meet the target strength in this study. Statements made on viability are however subject to the development of an official mix design guideline for waste mix concrete, future research aspects identified from this study and local standards and accreditation to sanction the use of waste concrete.

Chapter 5 : Conclusion and Recommendations

5.1. Introduction

This research was aimed at investigating the viability of utilizing SCBF, HDPE and BA as partial material replacements in terms of workability, strength, durability and cost. To enrich knowledge on the behaviour of the waste concrete investigated, the effects of varying proportions of waste (2,5%, 5%, 10%,20% and 40%) as well as properties such as elastic modulus, density, SEM analysis, specific heat and the moisture effects of the selected waste materials (SCBF, HDPE, and BA) were included in this study.

5.1.1. Conclusions on waste concrete property behaviour

5.1.1.1. SCBF

The use of SCBF offered the possibility of using a natural, renewable material as a partial aggregate substitute in concrete. Upon testing, the SCBF mixes showed the largest reduction in workability of the three waste materials. This was not only due to the higher absorbency of the SCBF but also possibly the long strands of bagasse restricting movement in the fresh mix. The workability of 63mm at 10% substitution however, was still classified as an S2 class slump which is specified for most concrete applications. The drop in density yielded by the SCBF mixes, coupled with the reduction in specific heat and increase in oxygen permeability relative to the control, showed that additional voids, such as those shown in the SEM images in section 4.10, did have an influence on the mix properties. The lowered specific heat meant that SCBF would not be of much benefit in terms of passive heating and cooling. The SCBF mixes did however improve on the control in terms of CC and WS. This may have been due to the tortuous fibres which disrupted flow paths used for capillary action and thus lowered the WS. Besides the possibility of errors in testing, the resistivity properties of the SCBF may have also contributed to the lower CC relative to the control mix. In terms of overall durability, SCBF mix was classified as having good durability properties and hence viability in this respect, was not compromised. Durability in terms of possible fibre deterioration however, was not part of this study and should be researched along with the effect of using preserving agents on SCBF when used in concrete. The possibility of additional voids being introduced to the mix may have also caused the reductions in compressive and flexural strength relative to the control mix. In terms of the effect of varying proportions of SCBF, generally concrete performance dropped as the proportions of bagasse increased. The critical volume for SCBF use was found to be 10% based on the compressive strength results. The splitting strength of the mixes up to 10% substitution, also increased relative to the control and this indicated that SCBF offered better resistance to shear than flexure at this percentage substitution. The other possibility for the reduced flexural strength relative to the control may have been that the fibres softened affecting the bond properties with the cement material and reducing their resistance to flexure. This reasoning complemented findings on elastic modulus, which was reduced relative to the control, emphasising the possibility of fibres being softened. The decrease in elastic modulus meant that the SCBF mixes offered less resistance to deformation than conventional concrete. Issues related to cracking were not part of this study and should be undertaken as future research along with developing an elastic modulus prediction model for SCBF concrete. Drying the SCBF did yield significant increases in compressive strength relative to the mixes with natural moisture state waste aggregates. To the extent that the mixes yielded 28 day compressive strength results above the target strength. However, this came at the price of a reduction in workability relative to the control. The reduction in workability was not to the degree that the mix was unworkable but applications for thin sections or members with clustered reinforcing was not recommend.

Economically the use of SCBF increased cost relative to the conventional mix, not only for the R/m^3 rate but also the total costs for the signalling room and coping scenarios. This meant that profit-centred markets would be less likely to utilize it. A plausible market would be that of Corporate Social responsibility (CSR) projects or those with environmental incentives, were the aim of the project may be more environmentally focused. This is where the benefit of reducing virgin minerals with a renewable materials that does not compromise compressive strength may be useful. Research into developing a standard (SANS) and mix deign guideline would however be required before any possible use.

5.1.1.2. **HDPE**

HDPE in the recycled form used for this study, allowed a possible means to utilize a material that was not biodegradable and reduced the consumption of virgin minerals, hence preserving the environment. Upon testing, performance generally dropped with increasing proportions of HDPE in the mix and the optimum/critical volume percentage for the HDPE mixes was found to be 2.5% substitution. The poor bond properties of the HDPE mixes were evident in the strength and durability tests conducted. The HDPE specimens did not meet the target compressive strength requirements and were less than the control in terms of compressive, flexural and splitting strengths. Hence, the HDPE mixes were not viable in terms of strength. The HDPE pellets were not very absorbent however, the dried specimens showed better compressive strength results than the specimens using natural moisture state HDPE.

Durability was also worse than the control in terms of WS and OPI due to the increase in void content. However, the CC improved possibly due to the resistivity properties of the HDPE pellets. Overall, the HDPE mix was still classified as having good durability properties.

The density of the HDPE mixes was also found to be less than the control. The reductions in density were due to the HPDE pellets weighing less than the stone they partially replaced, as well as the possible introduction of voids to the mix due to the poor bond properties of the smooth inert HDPE pellets. These voids were noticeable in the SEM images taken. The smooth surface of the HDPE also increased workability over the control significantly which may favour thin section non-structural applications. In terms of specific heat properties, the use HDPE increased the specific heat of the respective concrete mixes. Therefore, this could aid a building in terms of passive heating and cooling measures in terms of utilizing the HDPE mix for non-structural insulation panels.

The elastic modulus of the mixes containing HDPE was higher than the control mix. The stiffer mix may have been due the HDPE volumetric substitution at 2,5% not being sufficient for the poor bond properties of HDPE with the cement matrix to impose any negative effects on the concrete elastic modulus. This was a possibility because the elastic modulus model for conventional concrete from literature was reasonably consistent with the behaviour displayed by the HDPE mix results. The higher elastic modulus did not however correspond with the tensile properties tested and possible variations in cement composition for the HDPE batch may have also resulted in the higher elastic modulus compared to the control.

Economically the HDPE pellets were more expensive than the stone it replaced so naturally the mix cost rate per cubic metre was higher. The increase in concrete cost combined with the increase in transport cost to collect the HDPE pellet from the recycling facility resulted in both the cost for the signalling room and precast coping scenarios being more expensive than the respective control scenarios. Therefore, as with SCBF, the economic viability of the HDPE mixes was limited to projects incorporating CSR initiatives or environmental incentives. However, the viability of the HDPE mixes was further restricted by the lack of compressive strength not meeting the control target strength. Hence, based on this study, the HDPE mixes would be possibly viable only for non-structural applications. Its use would provide a workable mix that is lighter in mass and potentially supports passive heating and cooling measures.

5.1.1.3. **Bottom ash**

The use of BA would aid the environment not only in terms of reducing ash dumps but also decreasing the amount of cement used in concrete. Upon testing, performance generally dropped with increasing volumetric substitutions of BA in the mix. The critical volume/optimum percentage substitution for bottom ash was found to be at 5 % substitution.

The BA mixes showed slight reductions in workability and had little effect on density. The negligible reduction in density, reduced strength properties and elastic modulus relative to the control, indicated the possibility of a lack in pozzolanic reaction taking. Strength was however improved when using the oven-dry BA due to the effect of free-water absorption on the W/C in the transition zone. This corresponded to a decrease in workability but not to the extent that the mix was unworkable. In contrast to the findings on density, strength and elastic modulus, the OPI was better than the control indicating that permeability was reduced because of the possible pozzolanic behaviour of BA that may have created a pore filler effect for that sample. CC was also improved relative to the control and the durability overall was classified as good despite the BA mixes showing higher sorptivity than the control. The contrasting strength and durability results gave the impression that the consistency of bottom ash varied considerably. There may have been pockets of unreactive BA that caused additional voids to form or pockets of reactive BA that supported the pozzolanic reaction. The BA mix did not vary significantly from the control in terms of specific heat indicating that cement and BA behaved similar in this regard. However, the variation between actual and predicted elastic modulus indicated that BA behaved differently to conventional concrete in terms of relating compressive strength to elastic modulus.

The use of BA mix reduced the cost of concrete per cubic metre but the reduction in cost was not enough to offset the transport cost required for the waste material when considering the total construction costs. Subsequently, for both the coping and signalling room scenarios using BA was more costly than the control scenario. Therefore, as with HDPE and SCBF, the economic viability of the HDPE mixes was limited to projects incorporating CSR initiatives or government incentives.

5.1.2. Validation of objective completion

The objectives explained in section 1.5 to meet the research aim were achieved as follows:

- ✓ ***Investigate the effect of varying volumetric proportions (2.5%, 5%, 10%, 20%, and 40%) of waste materials (SCBF, HDPE, and BA) in density, strength and specific heat.***

  Refer to sections - 4.5, 4.6, 4.7 and 4.13.2, 4.13.3, 4.13.4

  The effect of varying proportions of waste showed a general decline in performance as waste proportions increased for compressive, flexural and splitting strengths. Density followed the same trend as strength with regard to changes with increasing proportions of waste. Workability decreased with increasing proportions of SCBF and BA whereas workability increased with increasing proportions of HDPE. In terms of specific heat, the HDPE mixes showed an increase in specific heat with increased proportions of waste, BA remained relatively constant and SCBF showed a decrease with increasing substitution percentages.

- ✓ ***Establish the critical volumetric substitution within a range of 2,5% to 40% substitution, to be used for further investigation from the compressive cube strength results.***

  Refer to section- 4.6.1 & 4.13.3

  The critical volume substitutions were not the substitutions at which a peak in performance was yielded but rather, were the substitutions after which performance declined significantly. Critical/optimum volumes substitutions were established at 2,5% for HDPE, 5% for BA and 10% for SCBF

- ✓ ***Further investigate the concrete mixes for each waste at critical volumes (based on compressive strength) and in combinations of waste at critical volume (BA+ SCBF, HDPE+ BA, SCBF + BA + HDPE), in terms of elastic modulus (28 day) and durability (28 day) (OPI, CC, and WS).***

  Durability – refer to section 4.9 & 4.13.6

Each durability test showed the effects of different aspect that changed within the waste mixes in comparison to the control mix. The SCBF mix was more permeable than the control but improved CC and WS. The HDPE mix was more permeable and showed a greater degree of water ingress through capillary action than the control but was still classified as having “good” durability properties in terms of WS. The HDPE mix was however classified as having “poor” durability properties in terms of OPI. In terms of CC, the HDPE mix was classified as having good durability properties and improved CC. The 5% BA mix was classified as good in terms of OPI and CC as it decreased permeability and lowered CC relative to the control mix. The BA mix was also classified as having “good” WS properties even though it increased water sorptivity relative to the control mix.

Elastic modulus – refer to sections 4.8 & 4.13.5

All mixes reduced elastic modulus except for the HDPE mix relative to the control mix. The negative effect of adding BA to the mix was emphasized when BA was added to the other waste mixes which subsequently decreased the stiffness of the respective mixes further.

- ✓ ***Identify whether the chosen waste materials perform better independently or in conjunction with each other by comparing individual waste mixes with mixed waste results.***

Refer to Chapter 4

All three waste materials generally performed better when used individually than when used in mixed combinations of waste.

- ✓ **Investigate the effect of moisture absorption on compressive cube strength and workability for each waste material at critical volumes, by testing mixes with wastes at dry moisture state and comparing the results to the natural state mix results.**

Refer to section - 4.11 & 4.13.8
The HDPE, SCBF and BA mixes increased in 28 day compressive strength by 4,85%, 24,71% and 24,37 to 31,4MPa, 40,1 MPa and 37,8 MPa respectively, relative to the specimens using waste a natural moisture state. The workability of

the HDPE mix increased by 1,2%, the SCBF decreased by 11,11% and the BA mix decreased by 5,5%, relative to the specimens using waste a natural moisture state.

- ✓ **Evaluate the material cost of each mix by creating and comparing the R/m^3 value using current market rates.**

Refer to section - 4.12.1 & 4.13.9

The use of BA mix reduced the cost of concrete per cubic metre by 4,81 % and the HDPE and SCBF increased the cost of concrete per cubic metre by 8.76% and 1,01% respectively relative to the control mix. The SCBF + BA decreased cost per cubic metre by 3,79% and the HDPE + BA decreased cost by 3,95% relative to the control mix. However, the SCBF + HDPE + BA mixed waste combinations was still 4,97 % more expensive per cubic metre than the control mix. The reductions in cost were because BA was essentially a free material and partially replaced the most costly conventional constituent being cement.

- ✓ **Evaluate construction cost implications in terms of a scenario analysis.**

Refer to sections - 4.12.2 & 4.13.9
All waste mixes were more costly than the scenario using the conventional concrete mix.

- ✓ **Comment on the viability of the waste concrete mixes in terms of strength, workability, durability and cost**

Refer to section - 4.4, 4.6, 4.9 and 4.12
The most viable waste mixes were the SCBF mix and BA mix. The HDPE mix did not meet the target strength and hence was limited to non-structural applications pending further research. The mixed waste combinations were deemed unfeasible as they did not grant any benefit over the individual waste mixes or conventional concrete.

If the project was profit-driven, none of the waste mixes would be viable as they all cost more than the control without granting any significant benefits apart from improved WS (SCBF) and CC & OPI (BA). If the project was focused on utilization of sustainable material in the case of SCBF or materials that reduce pollution (reduce dumps) in the case of BA, then the BA would be potentially viable for both small and large production works and the SCBF would be potentially viable for large scale works. These statements are in relation to the context of this study.

5.1.3. Response to research question

All three wastes were viable for projects incorporating CSR initiatives or environmental incentives, as the increased cost would be warranted by the environmental benefits of reducing virgin mineral consumption and utilizing waste materials. Based on findings from this study only SCBF and BA would be viable for possible structural use as the target strength was met when using dry-aggregates. The workability and durability were also not compromised for these mixes. HDPE however, was only viable for non-structural applications as the strength and durability properties were not adequate.

5.2. Recommendations

The recommendations based on the outcomes of this research are summarised as follows.

- Waste (BA, HDPE, and SCBF) aggregates are best used if oven-dried. If not oven dried, the W/C should be adjusted to cater for material moisture properties.
- The waste materials (BA, HDPE, and SCBF) should not be mixed together in concrete.
- The HDPE mix at 2,5% substitution has potential for non-structural applications only (based on this study).
- If the project is environmentally driven then the use of BA at 5% substitution or SCBF at 10% is potentially viable. However, this requires a standard, accreditation and specification guideline to be developed before implementation.

5.3. Possible future research

Possible avenues for future research based on this study are summarised as follows.

- A full housing model should be conducted for the life-cycle analysis of a building utilizing waste materials to fully understand the potential sustainable benefits in practice to reinforce the theoretical implications identified by this research. This should be conducted as separate research due to the depth involved for a life-cycle analysis.

- An energy model should be created to model the behaviour of a house using waste materials during different climatic conditions and times of day as the specific heat values obtained in this research were under fixed conditions.

- Research should be carried out into the effects on the optimum/critical volume independent mixes and mix combinations in terms of long term strength over 6 months to a year of curing which could not be covered in this research due to time constraints. This should be evaluated in future research. This would also indicate if the coal bottom ash from the sugar mill performs the same as coal ash from an electricity plant in terms of long term strength benefits.

- An investigation into the effect of varying W/C ratios using waste materials.

- An investigation into the long-term durability of bagasse fibres and the effect of using fibre preserving agents on concrete properties.

- An investigation into the effect of varying cement types using waste materials.

- Investigation into the effect of waste aggregates on cracking and shrinkage behaviour.

- The development an elastic modulus prediction formula/model for waste concrete.

- The development a chart for Compressive strength vs. W:C ratio with the selected waste aggregates and subsequent design method specification, to allow for use in industry by ensuring that a minimum strength can be achieved using a repeatable standard mix design.

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Appendix A: Sand fineness modulus raw data

SAND FINENESS MODULUS			
Total sample	500	g	
Test 1			
Sieve Size	Mass retained	Percentage retained	Percentage passing
4,75	6	1,2	98,8
2,36	24	6	94
1,18	157	37,4	62,6
0,6	248	87	13
0,3	57	98,4	1,6
0,15	7	99,8	0,2
pan		329,8	270,2
FM 1	3,298	FM 2	3,298
Test 2			
Sieve Size	Mass retained	Percentage retained	Percentage passing
4,75	6	1,2	98,8
2,36	24	6	94
1,18	189	43,8	56,2
0,6	214	86,6	13,4
0,3	57	98	2
0,15	7	99,4	0,6
pan		335	265
FM 1	3,35	FM 2	3,35
Test 3			
Sieve Size	Mass retained	Percentage retained	Percentage passing
4,75	3	0,6	99,4
2,36	35	7,6	92,4
1,18	175	42,6	57,4
0,6	211	84,8	15,2
0,3	67	98,2	1,8
0,15	7	99,6	0,4
pan	1	333,4	266,6
FM 1	3,334	FM 2	3,334

Test 4			
Sieve Size	Mass retained	Percentage retained	Percentage passing
4,75	6	1,2	98,8
2,36	24	6	94
1,18	175	41	59
0,6	231	87,2	12,8
0,3	56	98,4	1,6
0,15	6	99,6	0,4
pan		333,4	266,6
FM 1	3,334	FM 2	3,334
Test 5			
Sieve Size	Mass retained	Percentage retained	Percentage passing
4,75	3	0,6	99,4
2,36	42	9	91
1,18	294	67,8	32,2
0,6	130	93,8	6,2
0,3	29	99,6	0,4
0,15	1	99,8	0,2
pan	0	370,6	229,4
FM 1	3,706	FM 2	3,706
Averages			
FM 1	3,4044	FM 2	3,4044

Appendix B: Material Density and moisture data

RELATIVE DENSITY OF HDPE PLASTIC		
Mass of holder	185,1	g
Submerged mass	Oven-dry mass	Relative density
168,8	197,7	0,924
170,5	200,5	0,932
168,6	195,4	0,922
	Average	0,926
Relative density	926,029	kg/m^3

RELATIVE DENSITY OF 13.2mm STONE		
Mass of holder	185,6	g
Submerged mass	Oven-dry mass	Relative density
816	987,5	2,77
821,4	991,1	2,79
821	988,8	2,80
846,3	1030,5	2,79
844,2	1024	2,80
	Average	2,79
Relative density	2,788	kg/m^3

Material moisture properties		
Tested by: Contest- Concrete Technology Services Durban		
	Tested Moisture Absorption	Moisture content
HDPE	0,05%	0,01%
SCBF	181,80%	62,60%
BA	1,8 %	0,90%

Concrete Density

Individual waste mix density data					
Specimen		Mass (g)	7 day Density (kg/m³)	Mass (g)	28 day Density (kg/m3)
Control	1,00	7974	2362,67	8071	2391,41
	2,00	8068	2383,41	8062	2388,74
	3,00	8036	2388,15	8040	2382,22
	average	**8026**	**2378,07**	**8058**	**2387,41**
Specimen		Mass (g)	7 day Density (kg/m³)	Mass (g)	28 day Density (kg/m3)
HDPE 2.5%	1,00	7990	2367,41	8095	2398,52
	2,00	7779	2304,89	8019	2376,00
	3,00	7883	2335,70	8042	2382,81
	average	**7884**	**2336,00**	**8052**	**2385,78**
5%	1,00	7873	2332,74	7904	2341,93
	2,00	7889	2337,48	7896	2339,56
	3,00	7887	2336,89	7885	2336,30
	average	**7883**	**2335,70**	**7895**	**2339,26**
10%	1,00	7797	2310,22	7912	2344,30
	2,00	7936	2351,41	7902	2341,33
	3,00	7852	2326,52	7832	2320,59
	average	**7862**	**2329,38**	**7882**	**2335,41**
20%	1,00	7476	2215,11	7515	2226,67
	2,00	7537	2233,19	7571	2243,26
	3,00	7506	2224,00	7513	2226,07
	average	**7506**	**2224,10**	**7533**	**2232,00**
40%	1,00	7158	2120,89	7239	2144,89
	2,00	7151	2118,81	7205	2134,81
	3,00	7155	2120,00	7215	2137,78
	average	**7155**	**2119,90**	**7220**	**2139,16**

Specimen		Mass (g)	7 day Density (kg/m^3)	Mass (g)	28 day Density (kg/m^3)
2.5% SCBF	1,00	7910	2343,70	8026	2378,07
	2,00	7890	2337,78	8067	2390,22
	3,00	7951	2355,85	8047	2384,30
	average	**7917**	**2345,78**	**8046,67**	**2384,20**
5%	1,00	7876	2333,63	8003	2371,26
	2,00	7940	2352,59	8018	2375,70
	3,00	7863	2329,78	8014	2374,52
	average	**7893**	**2338,67**	**8011,67**	**2373,83**
10%	1,00	7867	2330,96	8081	2394,37
	2,00	7827	2319,11	7949	2355,26
	3,00	7786	2306,96	7963	2359,41
	average	**7827**	**2319,01**	**7998**	**2369,68**
20%	1,00	7655	2268,15	7763	2300,15
	2,00	7713	2285,33	7946	2354,37
	3,00	7645	2265,19	7889	2337,48
	average	**7671**	**2272,89**	**7866**	**2330,67**
40%	1,00	7651	2266,96	7638	2263,11
	2,00	7676	2274,37	7730	2290,37
	3,00	7619	2257,48	7662	2270,22
	average	**7649**	**2266,27**	**7677**	**2274,57**

Specimen		Mass (g)	7 day Density (kg/m^3)	Mass (g)	28 day Density (kg/m^3)
2.5% BA	1,00	7972	2362,07	8059,00	2387,85
	2,00	8049	2384,89	8105,00	2401,48
	3,00	7996,00	2369,19	8001,00	2370,67
	average	**8006**	**2372,05**	**8055**	**2386,67**
5%	1,00	7974,00	2362,67	8061	2388,44
	2,00	8024,00	2377,48	8023	2377,19
	3,00	8014,00	2374,52	8012,00	2373,93
	average	**8004**	**2371,56**	**8032**	**2379,85**
10%	1,00	8014	2374,52	7964	2359,70
	2,00	7965	2360,00	8002	2370,96
	3,00	7945,00	2354,07	7983,00	2365,33
	average	**7975**	**2362,86**	**7983**	**2365,33**

	1,00	7820	2317,04	7881	2335,11
	2,00	7878	2334,22	7847	2325,04
	3,00	7842,00	2323,56	7864,00	2330,07
20%	**average**	**7847**	**2324,94**	**7864**	**2330,07**
	1,00	7771	2302,52	7855	2327,41
	2,00	7697	2280,59	7781	2305,48
	3,00	7727,00	2289,48	7788,00	2307,56
40%	**average**	**7732**	**2290,86**	**7808**	**2313,48**

Mixed waste mix density data					
Specimen		Mass (g)	7 day Density (kg/m^3)	Mass (g)	28 day Density (kg/m^3)
2.5% HDPE + 5% BA	1,00	7935,00	2351,11	8032,00	2379,85
	2,00	7965,00	2360,00	7978,00	2363,85
	3,00	7943,00	2353,48	8005	2371,85
	average	**7948**	**2354,86**	**8005**	**2371,85**
10% SCBF + 5% BA	1	7801,00	2311,41	7853,00	2326,81
	2	7861,00	2329,19	7898,00	2340,15
	3	7801	2311,41	7876	2333,63
	average	**7821**	**2317,33**	**7876**	**2333,53**
10% SCBF + 2,5% HDPE + 5% BA	1	7620,00	2257,78	7658,00	2269,04
	2	7615,00	2256,30	7647,00	2265,78
	3	7622,00	2258,37	7653	2267,56
	average	**7619**	**2257,48**	**7653**	**2267,46**

Appendix C1: BA Mix designs

MSc Engineering: COAL-ASH NATURAL FIBRE REINFORCED AND RECYLCED PLASTIC REINFORCED CONCRETE-COAL ASH

Material properties					
Stone					
Compacted bulk density (CBD)	1492	kg/m³	Water density	1000	kg/m³
k-value	0,9		Cement density	3070	kg/m³
Relative density(kg/m³	2788				
Coal ash					
Relative density(kg/m³	2050,03	kg/m³			
Sand					
Relative density(kg/m³	2600	kg/m³			
Fineness Modulus(FM)	3,4044				

General		
Required strength	35	MPa
Water/Cement ratio	0,57	
Control*	5	SD
Deffects*	5	%
Error Margin*	7	MPa

Legend	
	Other input
	Mix Inputs
	Mix

Concrete mix proportions

Material	1(Control)		2.5%		5%		10%	
	Mass (kg)/m³	Absolute volume(l)/m³	Mass (kg)/m³	Absolute volume(l)/m³	Mass (kg)/m³	Absolute volume(l)/m³	Mass (kg)/m³	Absolute volume(l)/m³
Cement	394,74	128,58	394,74	128,58	394,74	128,58	394,74	128,58
Cement after sub	394,74	128,58	384,87	125,36	375,00	122,15	355,26	115,72
Sand	902,13	346,97	902,13	346,97	902,13	346,97	902,13	346,97
Stone	834,86	299,45	834,86	299,45	834,86	299,45	834,86	299,45
Water	225,00	225,00	225,00	225,00	225,00	225,00	225,00	225,00
coal ash	0,00	0,00	6,59	3,21	13,18	6,43	26,36	12,86

Material	20%		40%	
	Mass (kg)/m³	Absolute volume(l)/m³	Mass (kg)/m³	Absolute volume(l)/m³
Cement	394,74	128,58	394,74	128,58
Cement after sub	315,79	102,86	236,84	77,15
Sand	902,13	346,97	902,13	346,97
Stone	834,86	299,45	834,86	299,45
Water	225,00	225,00	225,00	225,00
coal ash	52,72	25,72	105,44	51,43

Appendix C2: SCBF Mix design

MSc Engineering: COAL-ASH NATURAL FIBRE REINFORCED AND RECYLCED PLASTIC REINFORCED CONCRETE-SUGARCANE BAGASSE

Material properties

Stone					
Compacted bulk density (CBD)	1492	kg/m^3	Water density	1000	kg/m^3
k-value	0,9		Cement density	3070	kg/m^3
Relative density(kg/m^3	2788				
Sugarcane fibre					
Relative density(kg/m^3	1250	kg/m^3			
Sand					
Relative density(kg/m^3	2600	kg/m^3			
Fineness Modulus(FM)	3,4044				

General

Required strength	35	MPa
Water/Cement ratio	0,57	
Control*	5	SD
Defects*	5	%
Error Margin*	7	MPa

Legend

(peach)	Other input
(yellow)	Mix Inputs
(green)	Mix

Concrete mix proportions

Material	(Control)		2.5%		5%		10%	
	Mass (kg)/m^3	Absolute volume(l)/m^3	Mass (kg)/m^3	Absolute volume(l)/m^3	Mass (kg)/m^3	Absolute volume(l)/m^3	Mass (kg)/m^3	Absolute volume(l)/m^3
Cement	394,74	128,58	394,74	128,58	394,74	128,58	394,74	128,58
Sand	902,13	346,97	902,13	346,97	902,13	346,97	902,13	346,97
Stone	834,86	299,45	834,86	299,45	834,86	299,45	834,86	299,45
Stone after sub	834,86	299,45	813,99	291,96	793,12	284,48	751,38	269,50
Water	225,00	225,00	225,00	225,00	225,00	225,00	225,00	225,00
Sugarcane fibre	0,00	0,00	9,36	7,49	18,72	14,97	37,43	29,94

Material	20%		40%	
	Mass (kg)/m^3	Absolute volume(l)/m^3	Mass (kg)/m^3	Absolute volume(l)/m^3
Cement	394,74	128,58	394,74	128,58
Sand	902,13	346,97	902,13	346,97
Stone	834,86	299,45	834,86	299,45
Stone after sub	667,89	239,56	500,92	179,67
Water	225,00	225,00	225,00	225,00
Sugarcane fibre	74,86	59,89	149,72	119,78

Appendix C3: HDPE Mix designs

MSc Engineering: COAL-ASH NATURAL FIBRE REINFORCED AND RECYLCED PLASTIC REINFORCED CONCRETE-HDPE

Material properties

Stone					
Compacted bulk density (CBD)	1492	kg/m³	Water density	1000	kg/m³
k-value	0,9		Cement density	3070	kg/m³
Relative density(kg/m³	2788				
HDPE pellets					
Relative density(kg/m³	926	kg/m³			
Sand					
Relative density(kg/m³	2600	kg/m³			
Fineness Modulus(FM)	3,4044				

General

Required strength	35	MPa
Water/Cement ratio	0,57	
Control*	5	SD
Deffects*	5	%
Error Margin*	7	MPa

Legend

(peach)	Other input
(yellow)	Mix Inputs
(green)	Mix

Concrete mix proportions

Material	Control		2.5%		5%		10%	
	Mass (kg)/m³	Absolute volume(l)/ m³	Mass (kg)/m³	Absolute volume(l)/m³	Mass (kg)/m³	Absolute volume(l)/ m³	Mass (kg)/m³	Absolute volume (l)/m³
Cement	394,74	128,58	394,74	128,58	394,74	128,58	394,74	128,58
Sand	902,13	346,97	902,13	346,97	902,13	346,97	902,13	346,97
Stone	834,86	299,45	834,86	299,45	834,86	299,45	834,86	299,45
Stone after sub	834,86	299,45	813,99	291,96	793,12	284,48	751,38	269,50
Water	225,00	225,00	225,00	225,00	225,00	225,00	225,00	225,00
HDPE pellets	0,00	0,00	6,93	7,49	13,86	14,97	27,73	29,94

Material	0,20		0,40	
	Mass (kg)/m³	Absolute volume(l)/ m³	Mass (kg)/m³	Absolute volume(l)/m³
Cement	394,74	128,58	394,74	128,58
Sand	902,13	346,97	902,13	346,97
Stone	834,86	299,45	834,86	299,45
Stone after sub	667,89	239,56	500,92	179,67
Water	225,00	225,00	225,00	225,00
HDPE pellets	55,46	59,89	110,92	119,78

Appendix C4: Final Mix designs

MSc Engineering: COAL-ASH NATURAL FIBRE REINFORCED AND RECYLCED PLASTIC REINFORCED CONCRETE-FINAL MIXES

Material properties

Stone		
Compacted bulk density (CBD)	1492	kg/m³
k-value	0,9	
Relative density(kg/m³	2788	
Sugarcane fibre		
Relative density(kg/m³	1250	kg/m³
HDPE pellets		
Relative density(kg/m³	926	kg/m³
Coal ash		
Relative density(kg/m³	2050,03	kg/m³
Sand		
Relative density(kg/m³	2600	kg/m³
Fineness Modulus(FM)	3,4044	

Water density	1000	kg/m³
Cement density	3070	kg/m³

General

Required strength	35	MPa
Water/Cement ratio	0,57	
Control*	5	SD
Deffects*	5	%
Error Margin*	7	MPa

Legend

(orange)	Other input
(yellow)	Mix Inputs
(tan)	Mix

Concrete mix proportions

Material	(Control)		2,5% HDPE + 5% Ash		10% SCBF + 5% Ash		2,5% HDPE + 10%SCBF	
	Mass (kg)/m³	Absolute volume(l)/m³	Mass (kg)	Absolute volume(l)/m³	Mass (kg)	Absolute volume(l)/m³	Mass (kg)	Absolute volume(l)/m³
Cement	394,74	128,58	394,74	128,58	394,74	128,58	394,74	128,58
Cement after sub	394,74	128,58	375,00	122,15	375,00	122,15	375,00	122,15
Sand	902,13	346,97	902,13	346,97	902,13	346,97	902,13	346,97
Stone	834,86	299,45	834,86	299,45	834,86	299,45	834,86	299,45
Stone after sub	834,86	299,45	813,99	291,96	751,38	269,50	730,51	262,02
Water	225,00	225,00	225,00	225,00	225,00	225,00	225,00	225,00
Coal ash	0,00	0,00	13,18	6,43	13,18	6,43	13,18	6,43
HDPE pellets	0,00	0,00	6,93	7,49	0,00	0,00	6,93	7,49
Sugarcane fibre	0,00	0,00	0,00	0,00	37,43	29,94	37,43	29,94

Appendix D: Natural moisture state slump data

Individual waste mix workability data

	Specimen	Slump (mm)						
Control	1	74	*Note: Slump rounded to the nearest whole number*					
	2	76						
	3	73						
	average	74						
Specimen		Slump (mm)	Specimen		Slump (mm)	Specimen		Slump (mm)
HDPE 2.5%	1	80	SCBF 2,5%	1	78	**BA 2,5%**	1	75
	2	79		2	77		2	76
	3	84		3	73		3	71
	average	**81**		average	**76**		average	**74**
5%	1	92	5%	1	64	**5%**	1	70
	2	91		2	65		2	75
	3	90		3	67		3	73
	average	**91**		average	**65**		average	**73**
10%	1	120	10%	1	67	**10%**	1	69
	2	116		2	65		2	70
	3	115		3	57		3	72
	average	**117**		average	**63**		average	**70**
20%	1	130	20%	1	58	**20%**	1	68
	2	122		2	54		2	69
	3	123		3	59		3	67
	average	**125**		average	**57**		average	**68**
40%	1	146	40%	1	53	**40%**	1	62
	2	136		2	58		2	67
	3	139		3	55		3	66
	average	**140**		average	**55**		average	**65**

Mixed waste mix workability data

Specimen		Slump (mm)	Specimen		Slump (mm)	Specimen		Slump (mm)
2.5% HDPE + 5% BA	1	80	10% SCBF + 5% BA	1	63	10% SCBF + 2,5% HDPE+ 5% BA	1	79
	2	78		2	59		2	75
	3	79		3	64		3	74
	average	79		average	62		average	76

Appendix E: Hardened natural moisture state strength results

HARDENED CONCRETE STRENGTHS											
Note: Tolerance = 15% of average		Compressive strength (MPa)				Flexural strength (MPa)				Splitting strength (MPa)	
		7 Day		28 Day		7 Day		28 Day		28 Day	
Mix name	Specimen number	load(kN)	Compressive strength(MPa)	load(kN)	Compressive strength(MPa)	load(kN)	Flexural strength(MPa)	load(kN)	Flexural strength(MPa)	load(kN)	Splitting strength(MPa)
Control											
Control	1	553,10	24,58	944,80	41,99	13,44	4,03	18,03	5,41	128,70	3,64
	2	569,50	25,31	947,7	42,12	14,65	4,40	18,18	5,45	113,90	3,22
	3	586,80	26,08	949,50	42,20	14,57	4,37	18,12	5,44	115,40	3,27
	average	**569,80**	**25,32**	**947,15**	**42,10**	**14,22**	**4,27**	**18,11**	**5,43**	**119,33**	**3,38**
Recycled HDPE plastic											
2,50%	1	401,20	17,83	658,40	29,26	13,16	3,95	16,09	4,83	107,40	3,04
	2	437,40	19,44	689,50	30,64	13,15	3,95	16,84	5,05	105,30	2,98
	3	419,90	18,66	673,50	29,93	13,23	3,97	17,68	5,30	110,10	3,12
	average	**419,50**	**18,64**	**673,80**	**29,95**	**13,18**	**3,95**	**16,87**	**5,06**	**107,60**	**3,04**
5%	1	383,80	17,06	614,40	27,31	10,79	3,24	14,90	4,47	96,90	2,74
	2	382,70	17,01	644,60	28,65	11,90	3,57	16,93	5,08	94,70	2,68
	3	383,20	17,03	631,90	28,08	11,84	3,55	16,77	5,03	98,80	2,80
	average	**383,23**	**17,03**	**630,30**	**28,01**	**11,51**	**3,45**	**16,20**	**4,86**	**96,80**	**2,74**
10%	1	381,50	16,96	631,30	28,06	11,15	3,35	16,39	4,92	96,90	2,74
	2	377,90	16,80	627,60	27,89	12,19	3,66	14,26	4,28	96,20	2,72
	3	378,50	16,82	629,50	27,98	10,17	3,05	15,06	4,52	97,30	2,75
	average	**379,30**	**16,86**	**629,47**	**27,98**	**11,17**	**3,35**	**15,24**	**4,57**	**96,80**	**2,74**
20%	1	387,10	17,20	612,20	27,21	11,50	3,45	15,29	4,59	91,90	2,60
	2	368,70	16,39	616,20	27,39	10,48	3,14	14,57	4,37	93,10	2,63
	3	375,50	16,69	609,10	27,07	10,36	3,11	14,88	4,46	91,00	2,57
	average	**377,10**	**16,76**	**612,50**	**27,22**	**10,78**	**3,23**	**14,91**	**4,47**	**92,00**	**2,60**
40%	1	370,60	16,47	459,10	20,40	8,99	2,70	13,22	3,97	92,40	2,61
	2	356,90	15,86	489,30	21,75	9,92	2,98	14,06	4,22	93,20	2,64
	3	365,90	16,26	489,60	21,76	9,83	2,95	12,49	3,75	90,40	2,56
	average	**364,47**	**16,20**	**479,33**	**21,30**	**9,58**	**2,87**	**13,26**	**3,98**	**92,00**	**2,60**

SCBF											
Note: Tolerance = 15% of average		Compressive strength (MPa)				Flexural strength (MPa)				Splitting strength (MPa)	
		7 Day		28 Day		7 Day		28 Day		28 Day	
		load(kN)	Compressive strength(MPa)	load(kN)	Compressive strength(MPa)	load(kN)	Flexural strength(MPa)	load(kN)	Flexural strength(MPa)	load(kN)	Splitting strength(MPa)
2,50%	1	438,30	19,48	673,90	29,95	13,29	3,99	15,45	4,64	136,80	3,87
	2	416,80	18,52	736,10	32,72	12,42	3,73	15,35	4,61	135,90	3,85
	3	423,40	18,82	725,80	32,26	13,52	4,06	15,98	4,79	140,70	3,98
	average	**426,17**	**18,94**	**711,93**	**31,64**	**13,08**	**3,92**	**15,59**	**4,68**	**137,80**	**3,90**
5%	1	438,50	19,49	692,80	30,79	13,07	3,92	15,22	4,57	141,80	4,01
	2	449,70	19,99	737,60	32,78	12,32	3,70	15,96	4,79	136,80	3,87
	3	421,50	18,73	710,70	31,59	13,82	4,15	14,27	4,28	134,80	3,81
	average	**436,57**	**19,40**	**713,70**	**31,72**	**13,07**	**3,92**	**15,15**	**4,55**	**137,80**	**3,90**
10%	1	474,60	21,09	698,50	31,04	12,54	3,76	14,56	4,37	124,10	3,51
	2	438,60	19,49	747,50	33,22	13,12	3,94	14,85	4,46	114,20	3,23
	3	452,10	20,09	724,50	32,20	11,08	3,32	14,89	4,47	125,30	3,55
	average	**455,10**	**20,23**	**723,50**	**32,16**	**12,25**	**3,67**	**14,77**	**4,43**	**121,20**	**3,43**
20%	1	308,40	13,71	528,90	23,51	10,78	3,23	13,48	4,04	91,80	2,60
	2	281,40	12,51	539,80	23,99	11,52	3,46	12,62	3,79	86,70	2,45
	3	324,90	14,44	519,30	23,08	10,64	3,19	11,55	3,47	96,30	2,72
	average	**304,90**	**13,55**	**529,33**	**23,53**	**10,98**	**3,29**	**12,55**	**3,77**	**91,60**	**2,59**
40%	1	363,60	16,16	510,70	22,70	10,60	3,18	12,69	3,81	84,30	2,39
	2	383,30	17,04	510,10	22,67	9,45	2,84	11,96	3,59	77,20	2,18
	3	350,90	15,60	505,90	22,48	11,75	3,53	11,89	3,57	85,40	2,42
	average	**365,93**	**16,26**	**508,90**	**22,62**	**10,60**	**3,18**	**12,18**	**3,65**	**82,30**	**2,33**

Note: Tolerance = 15% of average		Bottom ash									
		Compressive strength (MPa)				Flexural strength (MPa)				Splitting strength (MPa)	
		7 Day		28 Day		7 Day		28 Day		28 Day	
		load(kN)	Compressive strength(MPa)	load(kN)	Compressive strength(MPa)	load(kN)	Flexural strength(MPa)	load(kN)	Flexural strength(MPa)	load(kN)	Splitting strength(MPa)
2,50%	1	496,20	22,05	686,40	30,51	10,87	3,26	15,61	4,68	100,80	2,85
	2	508,30	22,59	692,30	30,77	11,23	3,37	15,40	4,62	98,00	2,77
	3	487,20	21,65	683,50	30,38	10,51	3,15	14,89	4,47	101,20	2,86
	average	**497,23**	**22,10**	**687,40**	**30,55**	**10,87**	**3,26**	**15,30**	**4,59**	**100,00**	**2,83**
5%	1	487,70	21,68	677,50	30,11	10,68	3,20	14,02	4,21	104,90	2,97
	2	516,30	22,95	693,20	30,81	9,94	2,98	15,19	4,56	103,50	2,93
	3	487,20	21,65	680,80	30,26	11,51	3,45	14,63	4,39	100,29	2,84
	average	**497,07**	**22,09**	**683,83**	**30,39**	**10,71**	**3,21**	**14,61**	**4,38**	**102,90**	**2,91**
10%	1	364,30	16,19	559,80	24,88	10,60	3,18	13,63	4,09	103,10	2,92
	2	358,90	15,95	519,20	23,08	9,68	2,90	13,51	4,05	89,20	2,52
	3	346,70	15,41	538,10	23,92	11,56	3,47	12,07	3,62	110,40	3,12
	average	**356,63**	**15,85**	**539,03**	**23,96**	**10,61**	**3,18**	**13,07**	**3,92**	**100,90**	**2,85**
20%	1	329,40	14,64	417,20	18,54	9,08	2,72	13,59	4,08	86,80	2,46
	2	300,20	13,34	422,30	18,77	8,27	2,48	13,74	4,12	84,40	2,39
	3	305,80	13,59	465,50	20,69	9,89	2,97	12,18	3,65	89,30	2,53
	average	**311,80**	**13,86**	**435,00**	**19,33**	**9,08**	**2,72**	**13,17**	**3,95**	**86,83**	**2,46**
40%	1	173,30	7,70	228,00	10,13	9,36	2,81	10,37	3,11	38,60	1,09
	2	165,10	7,34	251,00	11,16	8,53	2,56	9,54	2,86	36,60	1,04
	3	167,20	7,43	209,50	9,31	8,95	2,69	9,41	2,82	37,70	1,07
	average	**168,53**	**7,49**	**229,50**	**10,20**	**8,95**	**2,68**	**9,77**	**2,93**	**37,63**	**1,06**

Specimen		Average load(kN)	7 day Compressive strength(MPa)	Average load(kN)	28 day Compressive strength(MPa)	Average load(N)	7 day Flexural strength(MPa)	Average load(N)	28 day Flexural strength(MPa)	Average load(kN)	28 day Splitting strength(MPa)
2.5% HDPE+ 5% Ash	1	375,90	16,71	606,10	26,94	10,95	3,29	14,94	4,48	103,30	2,92
	2	374,70	16,65	574,40	25,53	11,58	3,47	13,13	3,94	103,50	2,93
	3	370,00	16,44	560,20	24,90	10,38	3,11	14,35	4,31	101,90	2,88
	average	**373,53**	**16,60**	**580,23**	**25,79**	**10,97**	**3,29**	**14,14**	**4,24**	**102,90**	**2,91**
10%SCBF + 5% Ash	1	251,80	11,19	479,40	21,31	10,23	3,07	12,89	3,87	84,80	2,40
	2	291,00	12,93	483,70	21,50	10,20	3,06	13,63	4,09	82,70	2,34
	3	268,40	11,93	476,60	21,18	10,32	3,10	11,76	3,53	83,90	2,37
	average	**270,40**	**12,02**	**479,90**	**21,33**	**10,25**	**3,08**	**12,76**	**3,83**	**83,80**	**2,37**
2.5% HDPE+10%SCBF + 5% Ash	1	288,10	12,80	510,60	22,69	9,48	2,84	12,17	3,97	95,40	2,70
	2	317,40	14,11	517,80	23,01	8,56	2,57	13,24	3,49	94,30	2,67
	3	289,10	12,85	498,60	22,16	10,34	3,10	11,28	3,54	93,50	2,65
	average	**298,20**	**13,25**	**509,00**	**22,62**	**9,46**	**2,84**	**12,23**	**3,67**	**94,40**	**2,67**

Appendix F: Elastic modulus data

Control $f_{Ck = 33,60\ MPa}$	Strain readings (Initial Huggenberger reading – Huggenberger reading at load)						
Stress (MPa)	Load (kN)	Stress	Strain 1	Strain 2	Strain 3	Strain 4	Average
0	0,00	0,00	0,00	0,00	0,00	0,00	0,00
0,5 MPa	8,84	0,50	0,01	0,01	0,00	0,00	0,01
0,33 X f_{Ck}	197,92	11,20	0,49	0,38	0,15	0,31	0,28
0,4 X f_{Ck}	237,50	13,44	0,57	0,45	0,20	0,38	0,34

2,5% HDPE $f_{Ck =}$ 23,96 MPa	Strain readings (Initial Huggenberger reading – Huggenberger reading at load)						
Stress (MPa)	Load (kN)	Stress	Strain 1	Strain 2	Strain 3	Strain 4	Average
0	0,00	0,00	0,00	0,00	0,00	0,00	0,00
0,5 MPa	8,84	0,50	0,02	0,02	0,02	-0,01	0,02
0,33 x f_{Ck}	141,12	7,99	0,25	0,18	0,18	0,00	0,18
0,4 x f_{Ck}	169,34	9,58	0,28	0,20	0,20	0,03	0,20

10% SCBF f_{Ck} = 25,72 MPa	Strain readings (Initial Huggenberger reading – Huggenberger reading at load)						
Stress (MPa)	Load (kN)	Stress	Strain 1	Strain 2	Strain 3	Strain 4	Average
0	0,00	0,00	0,00	0,00	0,00	0,00	0,00
0,5 MPa	8,84	0,50	0,02	0,03	0,00	0,01	0,01
0,33 x f_{Ck}	151,53	8,57	0,40	0,43	0,19	0,24	0,29
0,4 x f_{Ck}	181,84	10,29	0,47	0,49	0,22	0,30	0,34

5% BA f_{Ck} = 24,31 MPa	Strain readings (Initial Huggenberger reading – Huggenberger reading at load)						
Stress (MPa)	Load (kN)	Stress	Strain 1	Strain 2	Strain 3	Strain 4	Average
0	0,00	0,00	0,00	0,00	0,00	0,00	0,00
0,5 MPa	8,84	0,50	0,02	0,03	0,01	0,01	0,01
0,33 x f_{Ck}	143,23	8,10	0,10	0,24	0,28	0,28	0,27
0,4 x f_{Ck}	171,87	9,73	0,14	0,29	0,31	0,32	0,31

2,5% HDPE + 5% BA $f_{Ck} = 20{,}63$ MPa	Strain readings (Initial Huggenberger reading – Huggenberger reading at load)						
Stress (MPa)	Load (kN)	Stress	Strain 1	Strain 2	Strain 3	Strain 4	Average
0	0,00	0,00	0,00	0,00	0,00	0,00	0,00
0,5 MPa	8,84	0,50	0,00	0,02	0,04	0,00	0,02
0,33 x f_{Ck}	121,53	6,88	0,02	0,36	0,48	0,08	0,25
0,4 x f_{Ck}	145,83	8,25	0,04	0,41	0,55	0,11	0,30

10% SCBF + 5% BA $f_{Ck} = 17{,}06$ MPa		Strain readings (Initial Huggenberger reading – Huggenberger reading at load)					
Stress (MPa)	Load (kN)	Stress	Strain 1	Strain 2	Strain 3	Strain 4	Average
0	0,00	0,00	0,00	0,00	0,00	0,00	0,00
0,5 MPa	8,84	0,50	0,01	0,00	0,02	0,04	0,02
0,33 x f_{Ck}	100,51	5,69	0,21	0,05	0,20	0,41	0,22
0,4 x f_{Ck}	120,61	6,83	0,25	0,08	0,24	0,47	0,26

2,5% HDPE + 10% SCBF + 5% BA $f_{Ck} = 18{,}1$ MPa		Strain readings (Initial Huggenberger reading – Huggenberger reading at load)					
Stress (MPa)	Load (kN)	Stress	Strain 1	Strain 2	Strain 3	Strain 4	Average
0	0,00	0,00	0,00	0,00	0,00	0,00	0,00
0,5 MPa	8,84	0,50	0,00	0,01	0,03	0,03	0,02
0,33 x f_{Ck}	106,60	6,03	0,21	0,11	0,25	0,40	0,25
0,4 x f_{Ck}	127,93	7,24	0,24	0,14	0,31	0,45	0,30

	Average cumulative strain						
Stress (MPa)	Control	2,5% HDPE	10% SCBF	5% BA	10% SCBF + 5% BA	2,5% HDPE + 10% SCBF	2,5% HDPE + 10% SCBF + 5% BA
0,5 MPa	0,01	0,02	0,01	0,01	0,02	0,02	0,02
fc/3	0,28	0,18	0,29	0,27	0,22	0,25	0,25
0,4 f_{ck}	0,34	0,20	0,34	0,31	0,26	0,30	0,30

Elastic modulus		
	Actual	Predicted
2,5% HDPE	49,10	48,95
10% SCBF	30,28	51,16
5% BA	31,45	49,39
2,5% HDPE + 5% BA	27,69	44,79
10% SCBF + 5% BA	25,99	40,33
2,5% HDPE + 10% SCBF + 5% BA	24,45	41,62
Control	38,63	61,10

Appendix G: Durability index data

TESTING CARRIED OUT BY: CONTEST CONCRETE TECHNOLOGY SERVICES DURBAN

OPI	Control	2,5% HDPE	5% BA	10%SCBF	5% BA + 10% SCBF	5% BA + 2,5% HDPE	5% BA+ 2,5% HDPE+ 10% SCBF
A	10	10,16	9,99	9,79	9,74	9,85	9,61
B	9,88	9,91	10,03	9,8	9,21	9,64	8,76
C	9,74	9,4	9,75	9,34	9,44	9,52	9,47
D	9,89	9,15	9,94	9,83	8,94	9,8	9,34
AVERAGE	9,87	9,49	9,91	9,63	9,24	9,68	9,16

Sorptivity (mm/√hr)	Control	2,5% HDPE	5% BA	10%SCBF	5% BA + 10% SCBF	5% BA + 2,5% HDPE	5% BA+ 2,5% HDPE+ 10% SCBF
A	8,05	8,64	8,9	8,02	7,51	8,64	7,47
B	8,92	9,11	8,35	8,03	6,36	8,42	9
C	8,91	11,31	8,89	7,73	8,13	7,75	7,71
D	8	9,48	7,94	7,95	9,32	8,06	8,68
AVERAGE	8,47	9,08	8,52	7,93	8,32	9,12	8,41

Samples "C" for 2,5% and "A" for 5% BA + 10% SCBF discarded.

Chlorides (mS/cm)	Control	2,5% HDPE	5% BA	10%SCBF	5% BA + 10% SCBF	5% BA + 2,5% HDPE	5% BA+ 2,5% HDPE+ 10% SCBF
A	1,46	0,94	0,8	0,95	1,49	1,39	1,78
B	1,47	1,2	0,87	1,08	1,46	1,4	1,74
C	1,25	1,08	0,75	1,03	1,63	1,35	1,81
D	1,55	1,07	0,83	0,98	1,54	1,39	1,75
AVERAGE	1,43	1,07	0,81	1,01	1,53	1,38	1,77

Appendix H: Dry waste aggregate concrete data

TESTING CARRIED OUT BY: CONTEST CONCRETE TECHNOLOGY SERVICES DURBAN

	slump (mm)
2,5% HDPE + 10% SCBF+ 5% BA	74
2,5% HDPE	82
5% BA	69
10% SCBF	56
2,5% HDPE + 5% BA	78
10% SCBF + 5% BA	54
Control	74

Dry vs. natural moisture state waste aggregate concrete compressive strength results								
		Mix						
		Control	2,5%HDPE	5% BA	10%SCBF	5% BA + 10% SCBF	5% BA+ 2,5% HDPE	5% BA+ 2,5% HDPE+ 10%SCBF
Waste aggregate Moisture state	Dry 7-day	24,7	19,26	24,5	24,4	20,7	23,4	21,3
	Dry 28-day	42	31,4	37,8	40,1	37,5	35,6	35,9
	Natural 7-day	25,32	18,64	22,09	20,23	12,02	16,60	13,25
	Natural 28-day	42,10	29,95	30,39	32,16	21,33	25,79	22,62

Appendix I1: Specific heat

SPECIFIC HEAT OF CONCRETE MIXES												
Name o f sample	Control	Control	2.5%HDPE	5%HDPE	10%HPDE	20% HPDE	40% HDPE	2.5%SCBF	5%SCBF	10%SCBF	20%SCBF	40%SCBF
Trial 1		**Trial 3**										
Mass of sample(g)	74,64	74,64	80,74	80	81,3	76,89	86,54	83,07	82,73	83,25	82,37	84,39
Mass of water(g)	100,02	97,09	99,88	99,87	100,28	99,21	99,37	100,25	100	99,08	99,06	99,78
Intial temperature (ti)	24	24	26	25	25	25	26	25	25	25	25	25
Final combined temperaure(tf)	34	34	37	37	38	38	41	36	34	34	35	35
Specifc heat of water (C_w)- J/kg °C	4184	4184	4184	4184	4184	4184	4184	4184	4184	4184	4184	4184
		824,61										
Spefic heat of sample (C_s)- J/kg °C	849,50	(C_s) - J/kg °C	903,72	994,90	1082,10	1131,95	1221,43	867,85	689,65	679,03	774,12	761,08
		% change	102,69	113,05	122,96	128,62	138,79	98,61	78,36	77,16	87,96	86,48
Name o f sample	control		2.5%HDPE	5%HDPE	10%HPDE	20% HPDE	40% HDPE	2.5%SCBF	5%SCBF	10%SCBF	20%SCBF	40%SCBF
Trial 2												
Mass of sample(g)	75,27		81,02	79,07	80,81	81,21	80,74	87,67	74,92	86,2	89,44	86,51
Mass of water(g)	101,12		99,66	99,49	100,61	100,84	100,52	100,92	99,3	100,28	100,36	100,01
Intial temperature (ti)	25		26	25	25	25	26	25	25	25	25	25
Final combined temperaure(tf)	36		37	37	38	39	41	36	34	36	36	35
Specifc heat of water (C_w)- J/kg.°C	4184		4184	4184	4184	4184	4184	4184	4184	4184	4184	4184
Spefic heat of sample (C_s)- J/kg °C	966,09	(C_s) - J/kg °C	898,61	1002,77	1092,24	1192,38	1324,33	827,81	756,21	836,59	806,93	744,14
		% change	102,11	113,94	124,11	135,49	150,48	94,06	85,93	95,06	91,69	84,55
Average	control		2.5% HDPE	5% HDPE	10% HDPE	20% HDPE	40% HDPE	2.5% SCBF	5% SCBF	10%S SCBF	20% SCBF	40% SCBF
	880,07	(C_s) - J/kg °C	901,17	998,83	1087,17	1162,17	1272,88	847,83	722,93	757,81	790,52	752,61
		% change	102,40	113,49	123,53	132,05	144,63	96,34	82,14	86,11	89,82	85,52

Appendix I2: Specific heat

SPECIFIC HEAT OF CONCRETE MIXES								
Name o f sample	2.5% BA	5% BA	10% BA	20% BA	40% BA	2,5% HDPE+5%BA	10%SCBF+5%BA	2.5%HDPE+10%SCBF+5%BA
Trial 1								
Mass of sample(g)	80,07	83,73	88,82	79,99	78,1	79,63	85,92	81,48
Mass of water(g)	100,86	99,72	99,33	100,36	99,54	99,52	99,52	99,1
Intial temperature (ti)	25	25	25	25	25	25	25	25
Final combined temperaure(tf)	35	36	38	36	36	37	36	37
Specifc heat of water (C_w)-J/kg°C	4184	4184	4184	4184	4184	4184	4184	4184
Spefic heat of sample (C_s)-J/kg°C	810,83	856,46	981,10	902,26	916,54	996,02	832,95	969,29
% change	92,13	97,32	111,48	102,52	104,14	113,17	94,65	110,14
Name o f sample	2.5% BA	5% BA	10% BA	20% BA	40% BA	2,5% HDPE+5%BA	10%SCBF+5%BA	2.5%HDPE+10%SCBF+5%BA
Trial 2								
Mass of sample(g)	83,78	89,08	87,86	88,38	85,73	82,89	89,68	85,7
Mass of water(g)	99,21	100,16	99,52	99,42	99,98	100,68	100,35	99,49
Intial temperature (ti)	25	25	25	25	25	25	25	25
Final combined temperaure(tf)	37	38	37	38	37	37	36	37
Specifc heat of water (C_w)-J/kg°C	4184	4184	4184	4184	4184	4184	4184	4184
Spefic heat of sample (C_s)-J/kg°C	943,73	986,41	902,72	986,88	929,42	968,00	804,69	925,19
% change	107,23	112,08	102,57	112,14	105,61	109,99	91,43	105,13
	2.5% BA	5% BA	10% BA	20% BA	40% BA	2,5% HDPE+5%BA	10%SCBF+5%BA	2.5%HDPE+10%SCBF+5%BA
Averaeg Spefic heat of sample (C_s)- J/kg°C	877,28	921,43	941,91	944,57	922,98	982,01	818,82	947,24
% change	99,68	104,70	107,03	107,33	104,88	111,58	93,04	107,63

Appendix J1: Cost analysis – Waste mix rates

Material	Rate	Source/supplier
HDPE pellets	R 9,20 / kg	Re-SA
Material	Undelivered Rate	Source/supplier
Cement	R 67.50 per 50 kg bag	Aberdare Hardware
13,2 mm Stone	R 300.30/ m^3	Lafarge aggregates
Umgeni Sand	R 198 / m^3	Aberdare Hardware

Individual waste mixes costing (R/m3)								
	Control		2,5% HDPE		5% BA		10% SCBF	
	Quantity	Cost (R)	Quantity	Cost (R)	Quantity	Cost (R)	Quantity	Cost (R)
Cement	8 bags	540,00	8 bags	540,00	7,5 bags	506,25	8 Bags	540,00
Sand	0.346 m^3	68,7005	0.35m^3	68,70	0.35 m^3	68,70	0.35 m^3	68,70
Water	225 l	3,71	225 l	3,71	225 l	3,71	225 l	3,71
Stone	0,30 m^3	89,96	0.29 m^3	87,71	0,3 m^3	89,96	0.27 m^3	80,97
HDPE			6.93 kg	63,78				
SCBF							37,43 kg	16,10
BA					13,18	0,00		
Total		R 702,37		R 763,90		R 668,62		R 709,47

Mixed waste combinations costing (R/m^3)						
	10% SCBF + 5% BA		2,5% HDPE + 5% BA		10% SCBF + 2,5% HDPE + 5% BA	
	Quantity	Cost (R)	Quantity	Cost (R)	Quantity	Cost (R)
Cement	7,5 bags	506,25	7,5 bags	506,25	7,5 bags	506,25
Sand	0,35 m^3	68,70	0,35 m^3	68,70	0,35 m^3	68,70
Water	225 l	3,71	225 l	3,71	225 l	3,71
Stone	0,27 m^3	80,97	0,29 m^3	87,71	0,26 m^3	78,72
HDPE			6,93 kg	63,78	6,93 kg	63,78
SCBF	37,43 kg	16,10			37,43 kg	16,10
BA	13,18	0,00	13,18 kg	0,00	13,18 kg	0,00
Total		R 675,72		R 730,15		R 737,25

Appendix J2: Cost analysis – signalling room

MATERIALS				Control	2,5% HDPE	10% SCBF	5% BA	2,5% HDPE + 5% BA	10% SCBF +5% BA	2,5% HDPE + 10% SCBF + 5% BA
Item	Unit	Quantity	Rate (Exc VAT)	Amount (Exc VAT)						
20 L Curing Compound	No	2,00	170,00	R 340,00	R 340,00	R 340,00	R 340,00	R 340,00	R 340,00	R 340,00
G5 Chrusher Run Filling Material	Tons	20,00	110,60	R 2 212,00	R 2 212,00	R 2 212,00	R 2 212,00	R 2 212,00	R 2 212,00	R 2 212,00
30 Mpa concrete	m3	26,14	Refer to mix costing	R 18 362,06	R 19 970,64	R 18 547,67	R 17 479,73	R 19 088,31	R 17 665,35	R 19 273,93
250 Micron Waterproofing membrane	m2	60,32	4,67	R 281,69	R 281,69	R 281,69	R 281,69	R 281,69	R 281,69	R 281,69
Econo Form Panels for foundation	Item	1,00	481,53	R 481,53	R 481,53	R 481,53	R 481,53	R 481,53	R 481,53	R 481,53
900x 2100 Steel door	no	1,00	4 825,00	R 4 825,00	R 4 825,00	R 4 825,00	R 4 825,00	R 4 825,00	R 4 825,00	R 4 825,00
Electrical - Lights	Item	1,00	6 000,00	R 6 000,00	R 6 000,00	R 6 000,00	R 6 000,00	R 6 000,00	R 6 000,00	R 6 000,00
Clips and wedges	Item	3,00	300,00	R 23,68	R 23,68	R 23,68	R 23,68	R 23,68	R 23,68	R 23,68
Reinforcment Cut & bent (As per Bending Schedule)	Tons	1,11	9 650,00	R 10 682,55	R 10 682,55	R 10 682,55	R 10 682,55	R 10 682,55	R 10 682,55	R 10 682,55
Mesh Ref 245	Sheets	2,00	328,11	R 656,22	R 656,22	R 656,22	R 656,22	R 656,22	R 656,22	R 656,22
Mesh Ref 888	Sheets	3,00	1 189,20	R 3 567,60	R 3 567,60	R 3 567,60	R 3 567,60	R 3 567,60	R 3 567,60	R 3 567,60
Ferrule tubes and cones	item	3,00	263,16	R 20,78	R 20,78	R 20,78	R 20,78	R 20,78	R 20,78	R 20,78
Tie Rods	Item	3,00	632,40	R 49,93	R 49,93	R 49,93	R 49,93	R 49,93	R 49,93	R 49,93
Scaffolding	Item	1,00	263,16	R 263,16	R 263,16	R 263,16	R 263,16	R 263,16	R 263,16	R 263,16
2x3 Timber	m	36,00	15,50	R 558,00	R 558,00	R 558,00	R 558,00	R 558,00	R 558,00	R 558,00
50mm Duct Tape	Rolls	2,00	38,00	R 76,00	R 76,00	R 76,00	R 76,00	R 76,00	R 76,00	R 76,00
Shutter boards 18 or 21mm	sheet	12,00	370,00	R 116,84	R 116,84	R 116,84	R 116,84	R 116,84	R 116,84	R 116,84
Formwork to soffits of roof slabs exc 1.5m & not exc 3.5m high above bearing level	m2	26,21	28,95	R 758,71	R 758,71	R 758,71	R 758,71	R 758,71	R 758,71	R 758,71
Drip Fillets	m	75,00	63,00	R 124,34	R 124,34	R 124,34	R 124,34	R 124,34	R 124,34	R 124,34
Steel corner fillets	m	150,00	63,00	R 248,68	R 248,68	R 248,68	R 248,68	R 248,68	R 248,68	R 248,68
Internal Paint (2 Coats)	Litre	14,00	70,18	R 982,52	R 982,52	R 982,52	R 982,52	R 982,52	R 982,52	R 982,52
Plaster undercoat	Litre	14,00	61,00	R 854,00	R 854,00	R 854,00	R 854,00	R 854,00	R 854,00	R 854,00
Shutter release oil 200l (coverage 5m2/l)	Litre	35,00	25,25	R 883,75	R 883,75	R 883,75	R 883,75	R 883,75	R 883,75	R 883,75
Consumables (nails, floats, gloves etc)	per unit	1,00	200,00	R 200,00	R 200,00	R 200,00	R 200,00	R 200,00	R 200,00	R 200,00
Cover blocks	per unit	1,00	140,00	R 140,00	R 140,00	R 140,00	R 140,00	R 140,00	R 140,00	R 140,00
40x40x5mm Angle Iron	m	26,00	33,55	R 872,30	R 872,30	R 872,30	R 872,30	R 872,30	R 872,30	R 872,30
8mm Coach Screw	No	45,00	5,00	R 225,00	R 225,00	R 225,00	R 225,00	R 225,00	R 225,00	R 225,00
4.5mm Heavy Duty Checker plate	m2	3,69	672,46	R 2 481,38	R 2 481,38	R 2 481,38	R 2 481,38	R 2 481,38	R 2 481,38	R 2 481,38
110mm Cable Access Pipe (2.5m long)	m	25,00	33,80	R 845,00	R 845,00	R 845,00	R 845,00	R 845,00	R 845,00	R 845,00
Drywall Partitioning (KZN Partioning)	m2	16,00	300,00	R 4 800,00	R 4 800,00	R 4 800,00	R 4 800,00	R 4 800,00	R 4 800,00	R 4 800,00
Doors for drywall (Lafarge)	no	3,00	624,62	R 1 873,86	R 1 873,86	R 1 873,86	R 1 873,86	R 1 873,86	R 1 873,86	R 1 873,86
Total material cost				R 63 806,59	R 65 415,16	R 63 992,20	R 62 924,26	R 64 532,84	R 63 109,87	R 64 718,45

Labour									
1 Foreman	Hours	126,00	45,00	R 5 670,00	R 5 670,00	R 5 670,00	R 5 670,00	R 5 670,00	R 5 670,00
8 General Labour	Hours	126,00	29,00	R 29 232,00	R 29 232,00	R 29 232,00	R 29 232,00	R 29 232,00	R 29 232,00
Total labour cost				R 34 902,00	R 34 902,00	R 34 902,00	R 34 902,00	R 34 902,00	R 34 902,00
Plant									
230mm Skill Saw Makita	Item	1,00	1 920,00	R 1 920,00	R 1 920,00	R 1 920,00	R 1 920,00	R 1 920,00	R 1 920,00
Plate Compactor/Wacker	Days	14,00	140,00	R 1 960,00	R 1 960,00	R 1 960,00	R 1 960,00	R 1 960,00	R 1 960,00
Shoot to place concrete	Item	1,00	131,58	R 131,58	R 131,58	R 131,58	R 131,58	R 131,58	R 131,58
Concrete mixer	Days	14,00	210,00	R 2 940,00	R 2 940,00	R 2 940,00	R 2 940,00	R 2 940,00	R 2 940,00
Troxler	mths	17,40	3 639,99	R 1 666,73	R 1 666,73	R 1 666,73	R 1 666,73	R 1 666,73	R 1 666,73
Site Establishment	Item	1,00	2 840,00	R 2 840,00	R 2 840,00	R 2 840,00	R 2 840,00	R 2 840,00	R 2 840,00
Drive unit & Needle	Days	14,00	190,00	R 2 660,00	R 2 660,00	R 2 660,00	R 2 660,00	R 2 660,00	R 2 660,00
Consumables/Miscellaneous	item	1,00	200,00	R 200,00	R 200,00	R 200,00	R 200,00	R 200,00	R 200,00
Total plant cost				R 14 318,31	R 14 318,31	R 14 318,31	R 14 318,31	R 14 318,31	R 14 318,31
Transport									
Cement	Item	1,00	200,00	R 200,00	R 200,00	R 200,00	R 200,00	R 200,00	R 200,00
Stone	As per transportation calculations section 4.12.2			R 336,04	R 327,64	R 302,44	R 336,04	R 327,64	R 302,44
Sand				R 226,75	R 226,75	R 226,75	R 226,75	R 226,75	R 226,75
Waste materials				R 0,00	R 1 410,20	R 2 814,16	R 2 814,16	R 2 814,16	R 2 814,16
Total transport cost				R 762,79	R 2 164,59	R 3 543,35	R 3 576,95	R 3 568,55	R 3 543,35
TOTAL ESTIMATED COST				R 113 789,69	R 116 800,07	R 116 755,86	R 115 721,52	R 117 321,70	R 115 873,54

Appendix J3: Cost analysis – Copings

MATERIALS				Control	2,5% HDPE	10% SCBF	5% BA	2,5% HDPE + 5% BA	10% SCBF + 5% BA	2,5% HDPE + 10% SCBF + 5% BA
Item	**Unit**	**Quantity**	**Rate (Exc. VAT)**	**Amount (Excl. VAT)**						
30 MPa concrete	m^3	$0{,}06m^3$	Refer to mix costing	R 45,37	R 49,35	R 45,83	R 43,19	R 47,17	R 43,65	R 47,63
Mesh ref.311	m^2	$0{,}35m^2$	$R28{,}932/m^2$	R 10,13	R 10,13	R 10,13	R 10,13	R 10,13	R 10,13	R 10,13
Moulds#	unit	10	R 3200 *10 /83	R 385,54	R 385,54	R 385,54	R 385,54	R 385,54	R 385,54	R 385,54
Total material cost				R 441,04	R 445,01	R 441,50	R 438,86	R 442,83	R 439,32	R 443,29

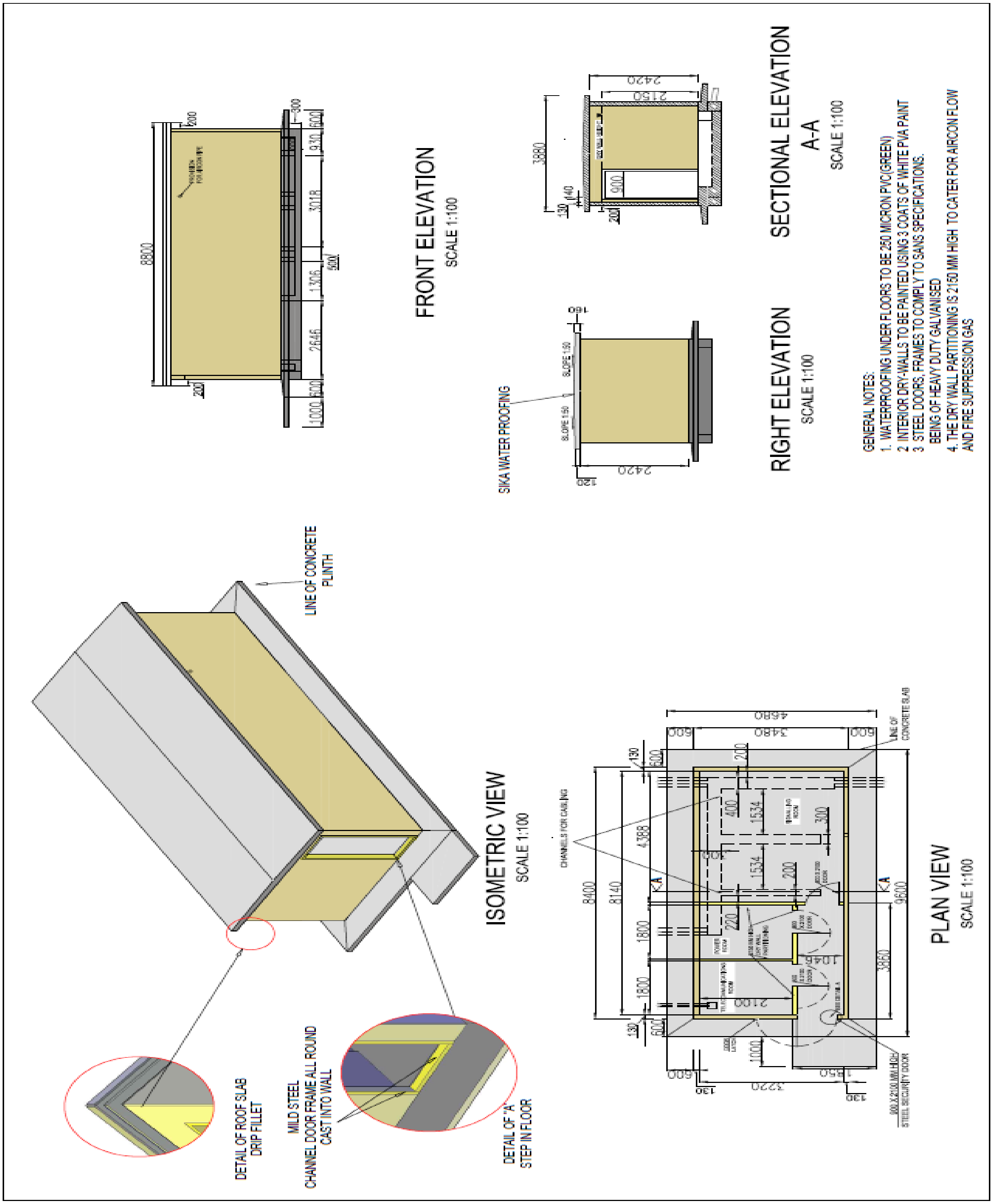
Designation	Rate (R/h)	Time(hours)	Cost (R)
4 No. General labour	29	0.5	R 14.50x 4 = R 58
1 No. Foreman	45	0.5	R 22.5
Cost per coping			R 80.5 / coping
Total cost for 83 copings			R6681.50

Plant & tools	Rate	Time	Cost (R)
230 mm Skil saw	R 1920 (purchase)	-	R 1920 (purchase)
Concrete mixer	R 210 /day	9 days	R 1890
Drive unit & needle	R 190/day	9 days	R 1710
Site Container incl handling & hire)	R 2840	-	R 2840
Consumable/Miscellaneous	R200	-	R 200
Total			R 8560
Total/ mould = total /83			R 103.13

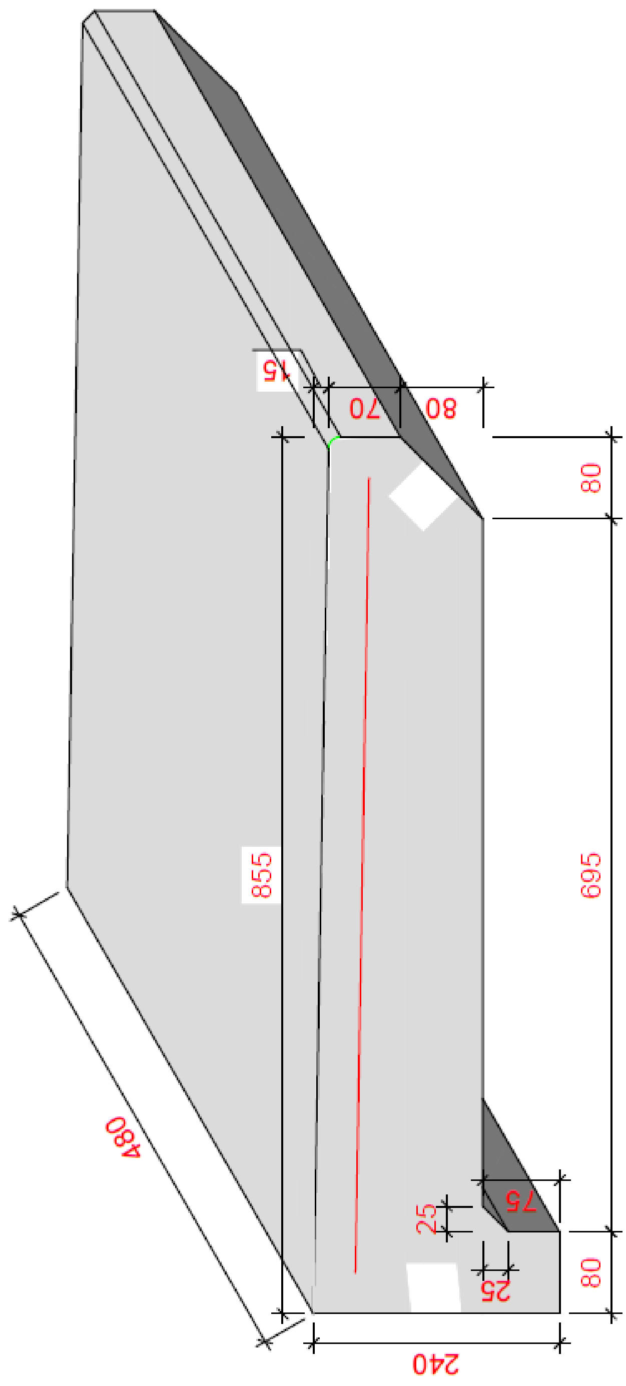
	Summary of transport costs for mixes (83 copings)						
	Control	2,5% HDPE	10% SCBF	5% BA	2,5% HDPE + 5% BA	10% SCBF + 5% BA	2,5% HDPE + 10% SCBF + 5% BA
Cement	R 200,00	R 200,00	R 200,00	R 200,00	R 200,00	R 200,00	R 200,00
Sand	R 46,84	R 46,84	R 46,84	R 46,84	R 46,84	R 46,84	R 46,84
Stone	R 69,42	R 67,68	R 62,48	R 69,42	R 67,68	R 62,48	R 60,74
Waste		R 1 410,20	R 2 814,16	R 2 814,16	R 2 814,16	R 2 814,16	R 2 814,16
Total	R 316,26	R 1 724,72	R 3 123,48	R 3 130,42	R 3 128,68	R 3 123,48	R 3 121,74

	Summary of costs for mixes							
	Control	HDPE	SCBF	BA	HDPE + BA	SCBF + BA	HDPE + SCBF + BA	
Total material cost	R 441,04	R 445,01	R 441,50	R 438,86	R 442,83	R 439,32	R 443,29	
Total labour cost	R 80,50	R 80,50	R 80,50	R 80,50	R 80,50	R 80,50	R 80,50	
Total plant cost	R 103,13	R 103,13	R 103,13	R 103,13	R 103,13	R 103,13	R 103,13	
Total transport cost	R 3,81	R 20,78	R 37,63	R 37,72	R 37,69	R 37,63	R 37,61	
Total cost to construct	R 628,48	R 649,43	R 662,76	R 660,21	R 664,16	R 660,58	R 664,54	

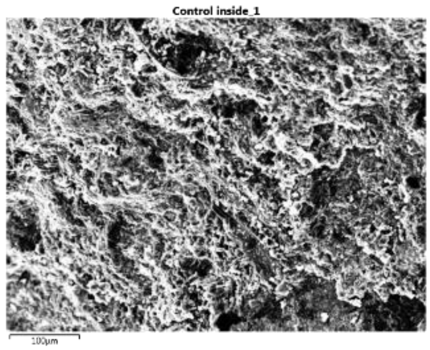
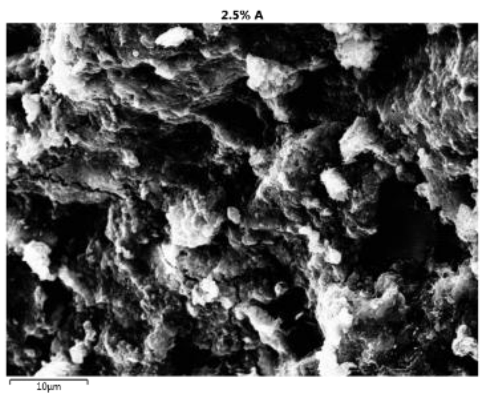
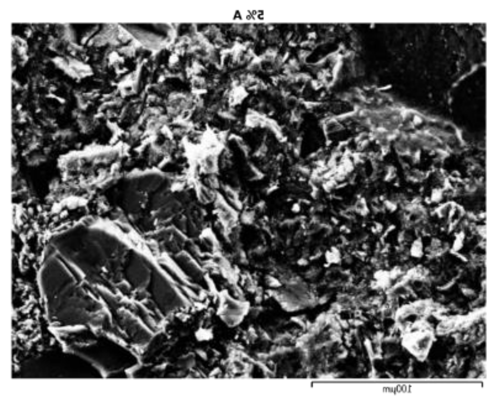
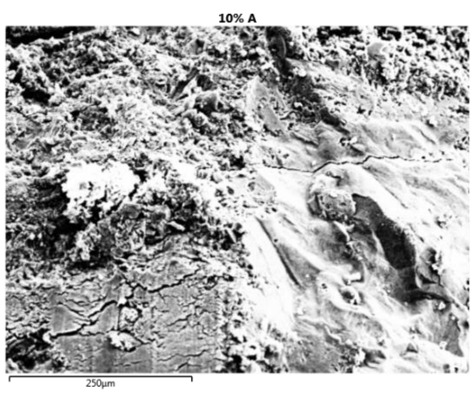
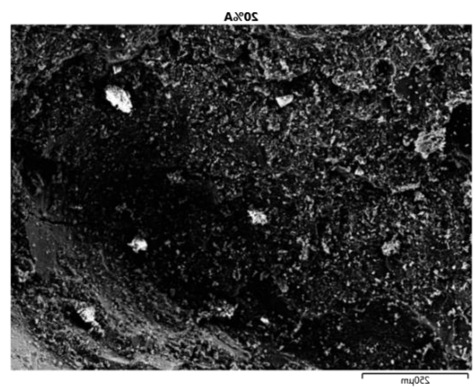
Appendix K: Signalling room drawing

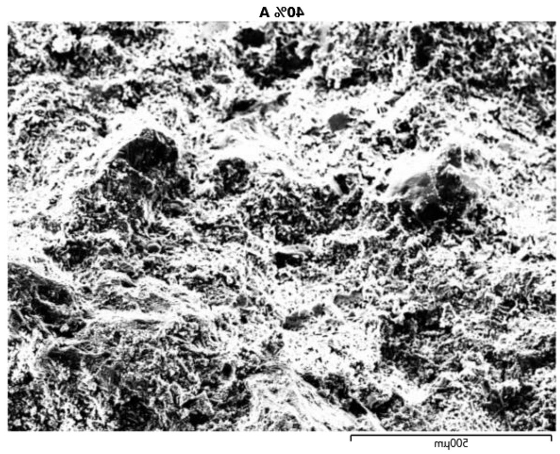
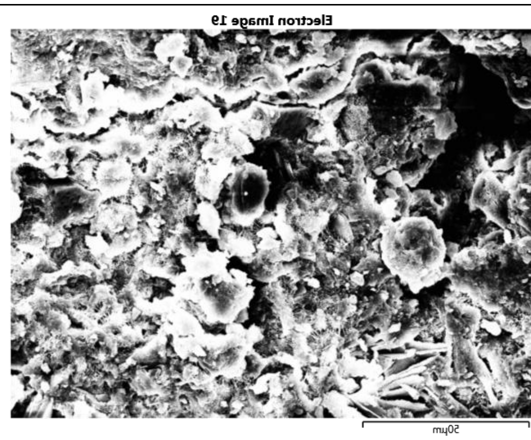
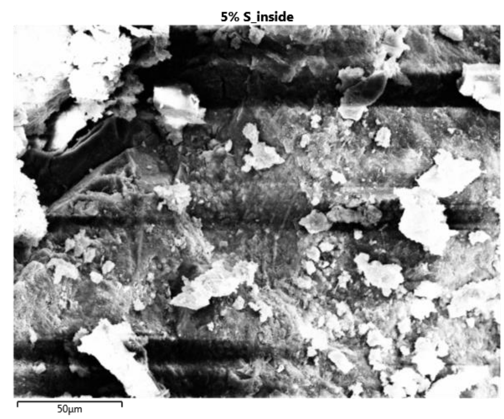
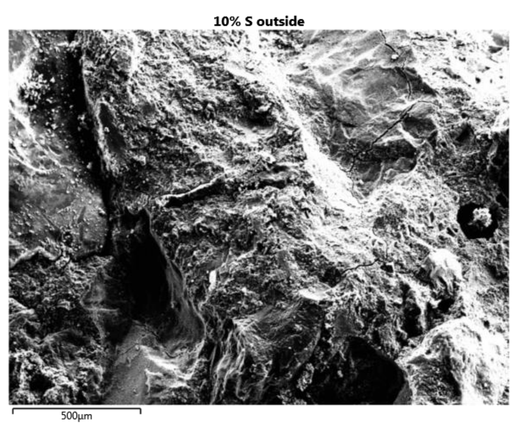
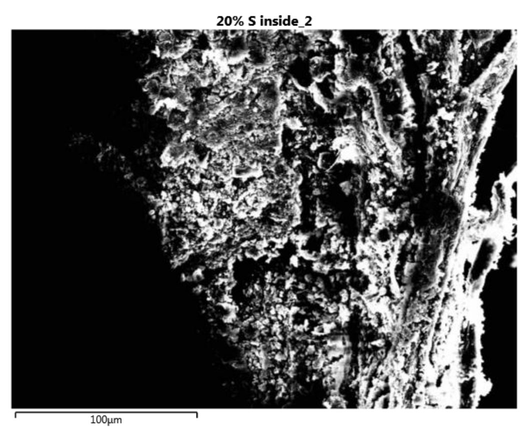


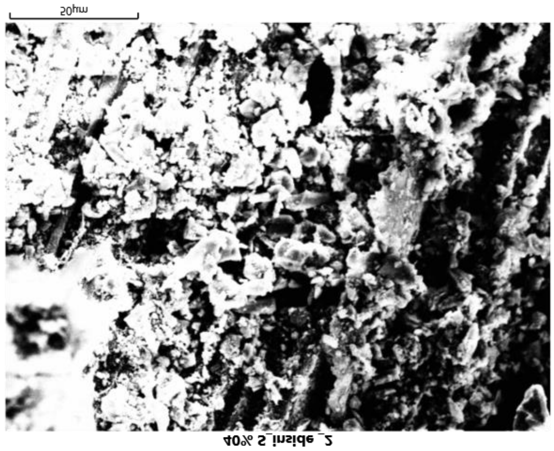
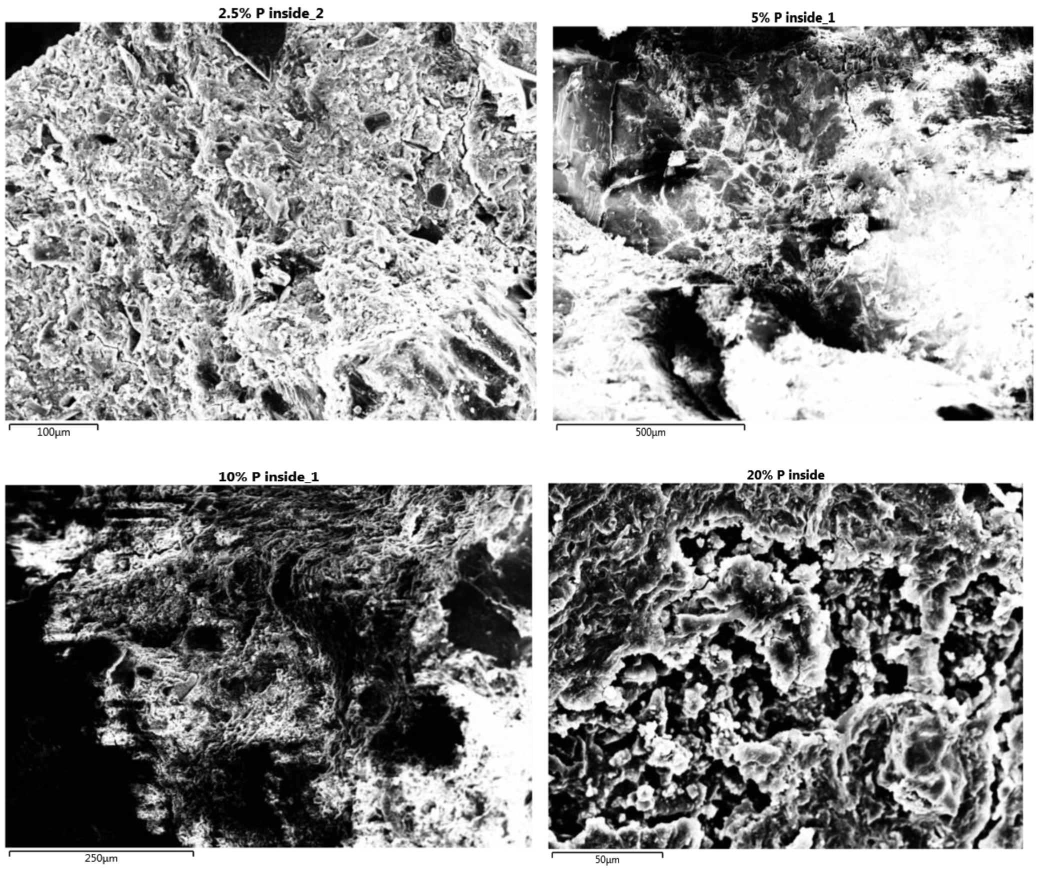
Appendix L: Coping drawing

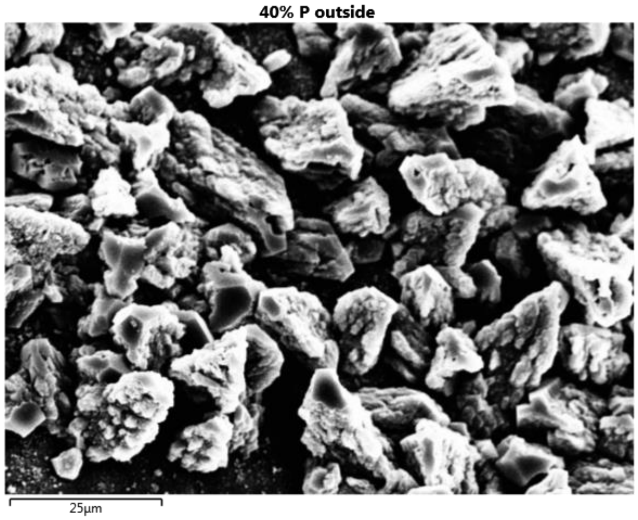


Appendix M: SEM images

Description	Image
Control	
BA	   

BA	 40% A 500µm
SCBF	 Electron Image 19 50µm  5% S_inside 50µm  10% S outside 500µm  20% S inside_2 100µm

SCBF	 50µm 40% S_inside_2
HDPE	 2.5% P inside_2 100µm 5% P inside_1 500µm 10% P inside_1 250µm 20% P inside 50µm

<table>
<tr><td>HDPE</td><td>
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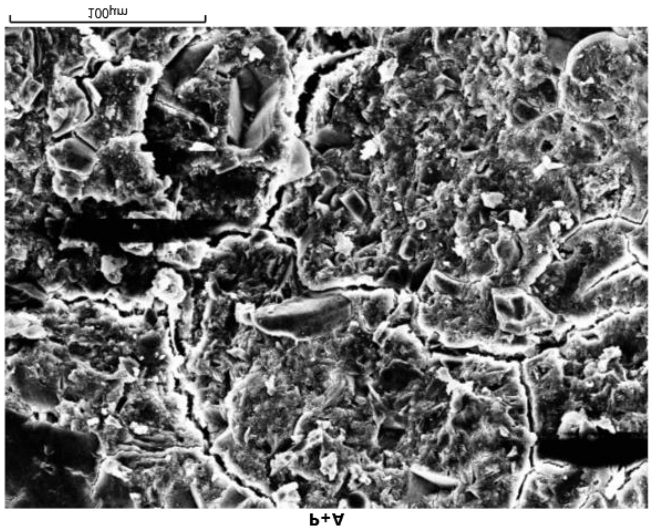
<tr><td>HDPE + BA</td><td>
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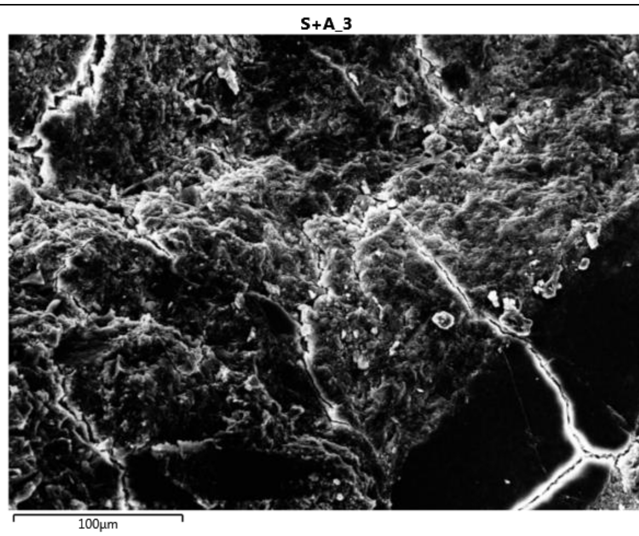
<tr><td>SCBF + BA</td><td>
</td></tr>
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HDPE SCBF+ BA