Thermal effects on concrete properties

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THERMAL EFFECTS ON CONCRETE PROPERTIES

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ABSTRACT

THERMAL EFFECTS ON CONCRETE PROPERTIES

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The expansion of nuclear energy as a future power source in South Africa, and the use of concrete for the containment vessels, has substantially increased the need for research in the concrete field. The production and development of environmentally friendly construction materials raise concerns about structural fire safety and thermal behaviour, as nuclear radiation shields face high levels of external heat and there is limited research on the performance of these materials at high temperatures. Therefore, the effect of elevated temperatures on the properties of concrete containing recycled aggregates, admixtures as well as blended cements is of importance in the design of concrete structures, such as concrete radiation shields. Operating temperatures of nuclear power plants range between 285°C – 650°C, depending on the reactor type. Other application such as outer shells of industrial chimneys or stacks and structures in metallurgy and chemical industry workshops, can also benefit from research on the thermal behaviour of concrete at high temperatures.

This study highlights the notable influence of aggregate type on the performance of concrete subjected to elevated temperatures. It is well known that heating concrete to elevated temperatures causes shrinkage of the hardened cement paste as well as thermal expansion of the aggregates. This can cause microcracking within the concrete, leading to degradation of the Interface Transition Zone (ITZ) between the aggregate and hardened cement paste, resulting in a reduction in the concrete strength and stiffness. The study proofed that concrete exposed to elevated temperatures in service should preferably contain aggregate with a low coefficient of

thermal expansion. Concrete exposed to 350°*C* retained more than 64*%* of its original strength, while concrete exposed to 500°*C* can retain more than 70*%* of its original strength after recovery as a result of rehydration when exposed to water. The order of preference of natural aggregate type for concrete exposed to elevated temperatures (up to 500°*C*) is felspathic (andesite, dolerite), granitic (granite, felsite) and calcareous (dolomite). Furthermore, high paste volumes (> 400 l/m^3) show noticeably more deterioration in strength after exposure to elevated temperatures. It is therefore recommended that the use of concrete mixtures with excessive paste volumes or cement contents should be avoided. The use of SCMs, such as fly ash, showed higher strength deterioration compared to pure Portland cement concrete. This was attributed to the disruptive effects of the cement paste shrinkage opposed by aggregate expansion for concrete with a compact microstructure. RAC can compete with concrete made with aggregates from conventional quarries, not only under normal temperature conditions but also after exposure to high temperatures. Structural concrete can easily be manufactured where 100% coarse aggregate and 30% fine aggregate is replaced with RCA.

It was hypothesised that aggregate that contain elements and minerals that decompose at relatively low temperatures, would place less stress on the surrounding cement paste, thus reducing the damage caused to the ITZ by the thermal expansion of the aggregate. The study established that the mass loss of aggregate obtained from thermogravimetric analysis (TGA) might give an indication of the performance of concrete exposed to elevated temperatures, especially considering dry compressive strength as well as mass loss of the concrete. The results indicate that it would be possible to limit the extent of thermal damage to concrete by selecting aggregates with limited (at least 1*%*) but not excessive (less than 2*%*) mass loss at the exposure temperature. The study demonstrated that degradation of concrete due to temperature exposure is not only caused by the thermal expansion of the aggregates but also by the mass loss of aggregates. To limit the damage caused to concrete by exposure to elevated temperatures, there seems to be a balance required between the thermal expansion of the aggregate and the reduction in stress caused by the aggregate degradation as indicated by mass loss of the aggregate at the specific exposure temperature.

DECLARATION

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Megan Brink

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LIST OF ABBREVIATIONS

Abbreviation	Description
A	Andesite
AGR	Advanced Gas-Cooled Reactor
BWR	Boiling Water Reactor
C	Cement
CSH	Calcium silicate hydrate
CTE	Coefficient of Thermal Expansion
D	Dolomite
EC	Embodied Carbon
EE	Embodied Energy
F	Felsite
FA	Unclassified Fly Ash
FI	Flakiness Index
FM	Fineness Modulus
FNR	Fast Neutron Reactor
G	Granite
ITZ	Interface Transition Zone
LOI	Loss of ignition
LWGR	Light Water Graphite-Moderated Reactor
OPI	Oxygen Permeability Index

Abbreviation	Description
PHWR	Pressurised Heavy Water Reactor
PP	Polypropylene fibres
PSD	Particle Size Distribution
PWR	Pressurised Water Reactor
R	Dolerite
RAC	Recycled aggregate concrete
RCA	Recycled concrete aggregate
RD	Relative density
SCM	Supplementary cementitious material
SP	Superplasticiser
T	Tillite
TGA	Thermogravimetric analysis
w/b	water-to-binder ratio
w/c	water-to-cement ratio
XRF	X-Ray Fluorescence

1 INTRODUCTION

Concrete is not a modern material and the oldest form of concrete discovered dates back to around 7000 BC. Nowadays, the production of concrete as a construction material has significantly increased due to urbanisation and rapid demographic growth (Muigai et al., 2013). Furthermore, this growth has led to an increased demand and need for affordable and sustainable energy sources. Presently, 72.1% of South Africa's primary energy needs are provided by coal fired power stations (Eskom, 2021). However, the main challenge for coal as an energy source arises from environmental concerns. Burning coal not only leads to air pollution but also has a detrimental impact on the environment through the associated mining activities (Eskom, 2021). In South Africa, the availability of alternative energy sources, including natural gas, wind power, and hydroelectricity, is limited. Wind turbine units have restricted capacity, requiring the installation of hundreds of turbines to replace a single coal power station. The country experiences alternating wet and dry climatic cycles, affecting river flow to dams and consequently influencing water availability for hydropower plants. Given these limitations, nuclear power emerges as a valuable alternative energy source in South Africa.

Initially, the primary focus of nuclear energy was for military purposes. It was not until 1951 that the first nuclear power plant was built in the United States and nuclear power became a viable option for civilian use. By the 1980's, many countries around the world had invested in nuclear power plants. The United States have the largest fleet of operational nuclear plants, producing 20% of the country's electricity, while France gets 70% of its electricity supply from nuclear power.

South Africa is one of the countries that has been using nuclear power since the 1980's. The Koeberg nuclear power plant, located near Cape Town, provides only about 4% of South Africa's total electricity supply. The Koeberg plant is based on a well proven Westinghouse Pressurised Water Reactor (PWR) design, similar to nuclear power plants built in France. The use of concrete has played a significant role in the construction of containment structures for nuclear power plants, which ensures the safety of the surrounding environment. In its time, the Koeberg project was renowned for producing the largest local cement order in South Africa, with more than 100 000 tonnes of Portland Cement used during construction. The containment structure consists of a 0.9 *m* thick post-tensioned concrete shell for the twin 56 *m* high domed reactor towers which are lined with a 6 *mm* steel liner. The reactor vessel is enclosed by a massive 1.8 *m* thick wall of reinforced concrete to prevent radiation penetration into the containment environment. South Africa is also home to the Pelindaba nuclear research centre, which conducts research into nuclear technology and is a key player in the nuclear industry (Public relations division of Eskom, 1982; Cement Sales Digest, 1979).

From the mid 1980's, many countries around the world halted future nuclear power programs and the nuclear power industry experienced some stagnation. This could possibly be attributed to the 1986 accident at the Chernobyl-4 nuclear power plant in the Soviet Union. However, the recognition of the increasing global electricity demand, coupled with the need to restrict carbon emissions, revived the potential for nuclear power in the modern era. In recent years, South Africa has also shown renewed interest in nuclear power, with plans to develop new nuclear power plants to meet the rapid growing energy need. In addition to South Africa, other countries around the world are also looking to expand their nuclear power facilities by extending the life of existing reactors or by building new ones. These countries include Brazil, Belgium, China, France, Germany, Japan, Poland, Romania, South Korea, United Kingdom and the United States. Several African countries, including Ghana, Kenya, Namibia, Nigeria, Sudan, Tanzania, Uganda and Zambia have shown a keen interest in developing nuclear power programs as a way to meet their growing energy needs. Even as the world shifts towards renewable energy sources, nuclear power is expected to continue playing a significant role in meeting the increasing global energy demand.

1.1 PROBLEM STATEMENT

The expansion of nuclear energy as a future power source in South Africa, and the use of concrete for the containment vessels, has substantially increased the need for research in the concrete field. The production and development of environmentally friendly construction materials raise concerns about structural fire safety and thermal behaviour, as nuclear radiation shields face high levels of external heat and there is limited research on the performance of these materials at high temperatures (Stefan et al., 2020; Khaliq & Taimur, 2018; Kodur, 2014; Khaliq & Kodur, 2011). It is well known that heating concrete to elevated temperatures causes shrinkage of the hardened cement paste as well as thermal expansion of the aggregates. This can cause microcracking within the concrete, leading to degradation of the Interface Transition Zone (ITZ) between the aggregate and hardened cement paste, resulting in a reduction in the concrete strength and stiffness. Operating temperatures of nuclear power plants range between 285°C – 650°C, depending on the reactor type (World Nuclear Association, 2023). Other application such as outer shells of industrial chimneys or stacks and structures in metallurgy and chemical industry workshops, can also benefit from research on the thermal behaviour of concrete at high temperatures.

Cement is the primary contributor of CO_2 emissions in the construction industry and pressure to reduce the carbon footprint of cement and develop efficient, environmentally friendly cementitious binders has significantly increased in the last half century (Collis, 2021; du Toit, 2021; Muigai et al., 2013). Energy production in South Africa still mainly depends on coal and about 95% of the fly ash produced annually is being stored and disposed of in ash dams and landfills. Utilising fly ash as a cementitious binder may not only solve the serious problem of the abundance of waste, not otherwise utilised, but also reduce the carbon footprint of cement (Shekhovtsova et al., 2016).

As Alexander & Mindess (2005) state: "*Sources of natural aggregates are becoming increasingly scarce. The focus of concrete science and technology in the last half century has mostly been on the binder component; increasing focus on the aggregate component will now also be required in the coming decades.*" According to Salahuddin et al. (2019), the worldwide aggregate consumption by the construction industry is estimated to reach 62.9 billion tons per annum by 2024, which can lead to rapid depletion of natural aggregate resources. For many years, concrete in South Africa has been made using quartzite obtained from waste dumps adjacent to mining operations on the Witwatersrand (Alexander & Ballim, 1986; Davis et al., 1978). However, the supply of quartzite has been exhausted as mines have closed and the waste dumps consumed (Bonser & Alexander, 2021). The demand for alternative aggregate sources such as recycled aggregates and waste streams will progressively increase (Bonser & Alexander, 2021; Salahuddin et al., 2019; Davis et al., 1979). Furthermore, the rapid developing construction industry and the demolition of old and damaged structures have led to a considerable amount of construction and demolition waste. Europe produces approximately 850 million tons of construction and demolition waste per annum, while 86% of the waste is recycled and recovered. South Africa on the other hand generates 4.5 million tonnes of construction and demolition waste per annum, while only 19.4% of the waste is recycled and recovered (Dhir et al., 2019). The utilisation of recycled aggregates acquired from demolition of older concrete structures may present as an environmentally friendly alternative to preserve natural resources, decrease landfill waste, and encourage sustainability (Bonser & Alexander, 2021; Khaliq & Taimur, 2018; Xiao et al., 2013). However, according to Soutsos et al. (2011), there is a research need to determine whether recycled aggregates can compete with aggregate from conventional quarries.

Therefore, the effect of elevated temperatures on the properties of environmentally friendly concrete, i.e., concrete containing recycled aggregates, admixtures as well as blended cements is of importance in the design of concrete structures, such as concrete radiation shields.

It is hypothesised that the damaged caused by concrete exposed to high temperatures can be limited by using aggregate with a low thermal expansion coefficient and that aggregate that contain elements and minerals that decompose at relatively low temperatures, would place less stress on the surrounding cement paste, thus reducing the damage caused to the ITZ by the thermal expansion of the aggregate.

1.2 OBJECTIVES OF THE STUDY

The objectives of the study were to determine:

- the effect of elevated temperatures on the properties of conventional concrete made with varying:
    - i) water/cement ratios,
    - ii) aggregate types, and
    - iii) fine to coarse aggregate ratios.
- whether the material properties of concrete made using recycled aggregate is comparable to that of normal concrete before and after exposure to high temperatures.
- whether aggregate properties could be used to predict the extent of damage caused to concrete by exposure to elevated temperatures.
- the impact of elevated temperature on the properties of environmentally friendly concrete, i.e., concrete containing recycled aggregates, admixtures as well as blended cements.

1.3 SCOPE OF THE STUDY

The scope of the study was limited to physical testing of concrete containing locally available materials. Natural aggregates from six different South African sources were investigated which include:

- ➢ Eikenhof andesite (igneous rock) from Gauteng,
- ➢ Pietermaritzburg dolerite (igneous rock) from KwaZulu-Natal,
- ➢ Olifantsfontein dolomite (sedimentary rock) from Gauteng,
- ➢ Zeekoewater felsite (igneous rock) from Mpumalanga,
- ➢ Rheebok granite (igneous rock) from Western Cape, and
- ➢ Verulam tillite (sedimentary rock) from KwaZulu-Natal.

Dolomite aggregate was used as the reference aggregate throughout the study. Heavyweight concrete and lightweight concrete were not considered. Materials properties were experimentally determined on all the aggregates prior to designing concrete mixtures, in accordance with the relevant standards which include:

- chemical composition,
- mineral composition,
- thermogravimetric analysis (TGA),
- relative density (RD),
- water absorption capacity,
- particle size distribution (PSD),
- fineness modulus (FM),
- flakiness index (FI) and
- surface texture.

Only one source of Portland cement (CEM I 52.5R) and fly ash (unclassified) was investigated. A constant water content (230 l/m^3) was used with a water/cement ratio of 0.5 (except for mixtures with varying water/cement ratios). The admixture dosage was constant at 0.5$\%$ by weight of the cement. Concrete containing admixture had a reduced water and cement content.

The volume of all mixtures was kept constant throughout the study and fresh properties of the mixtures such as slump and heat generation were determined. All the specimen were cured in water until the day of testing. Standard material properties were determined by testing specimen in saturated condition to conform with standard procedures. Hardened concrete properties evaluated include compressive strength, indirect tensile strength (splitting tensile strength), modulus of elasticity, coefficient of thermal expansion, drying shrinkage and potential durability. Potential concrete durability evaluated were limited to Oxygen Permeability Index (OPI) and porosity. Setting time, autogenous shrinkage, chloride penetration, freeze-thaw, elastic strain and creep fall outside the scope of this study.

There are three steady-state test conditions for evaluating the strength of concrete at elevated temperatures, namely residual, unstressed, or stressed. For temperature testing, the residual steady state test method was used in this study and unstressed, or stressed testing conditions were not considered. To prevent concrete spalling all samples were placed in a 50$°C$ oven to dry for seven days before being exposed to elevated temperatures. Due to load-shedding and time constraints, only one heating rate (0.5$°C/min$), one heating duration (12 *hours*) and four exposure temperatures (50$°C$, 200$°C$, 350$°C$ and 500$°C$), were evaluated in this study. Furthermore, small specimen were used for all temperature testing and large scale testing fall outside the scope of this study. Although tests are not fully representative of nuclear use or fire scenarios, novel insights are presented regarding the high-temperature response that can add value to the body of knowledge. Thermal cycling fall outside the scope of this study. Concrete properties evaluated after temperature exposure include mass loss, compressive strength, indirect tensile strength (splitting tensile strength), modulus of elasticity as well as oxygen permeability index (OPI) and porosity. The concrete cube samples were also tested in both saturated and dry condition to establish the effect of moisture content on the compressive strength.

It is well known that the addition of polypropylene fibres in concrete mixtures can prevent explosive spalling as they rapidly melt at $160°C – 170°C$ to provide pressure relief channels and create voids. However, their influence on the relative properties of concrete after temperature exposure in comparisons to the other mixtures was of interest and an additional mixture containing polypropylene fibres was therefore cast. The polypropylene microfibre dosage was limited to only 2 kg/m^3, as recommended in literature. No other fibre types, sizes or shapes were considered.

1.4 METHODOLOGY

During the preliminary stage of the project, the behaviour of conventional concrete mixtures made with readily available South African materials, when exposed to elevated temperature conditions was studied. The work served as a reference to the main investigation on the behaviour of environmentally friendly concrete when subjected to elevated temperatures. Environmentally friendly concrete was considered in this study as concrete containing recycled aggregates, admixture or blended cement, or all of the aforementioned.

Six different natural South African aggregate sources were investigated in the study. The addition of fine and coarse aggregates changes the properties of the paste, and it was of importance to understand the behaviour of the paste itself. As such, an additional mixture was cast to determine the heat generation, coefficient of thermal expansion and drying shrinkage of the paste. Old concrete samples containing dolomite aggregate was crushed and graded to acquire recycled concrete aggregates (RCA) which was used in the main study. The suitability of RCA as a concrete material was evaluated and compared to the natural aggregates. Standard fresh and mechanical properties were investigated as mentioned in Chapter 1.3.

After the experimental analysis was completed, an environmental impact and cost assessment were performed on some of the mixtures used in this study to highlight the consequence of utilising waste materials such as fly ash and recycled aggregate as well as limiting the cement content in concrete to encourage sustainable construction.

The structural fire safety and thermal behaviour of concrete structures, such as concrete radiation shields, produced with environmentally friendly construction materials was of significant importance in this study. Therefore, to evaluate the effect of elevated temperatures on the concrete properties, various samples were subjected to four different temperatures namely, 50°*C*, 200°*C*, 350°*C* and 500°*C*.

1.5 ORGANISATION OF REPORT

The thesis consists of the following chapters:

Chapter 1 containing the problem statement, objectives, scope and methodology.

Chapter 2 presenting a theoretical framework for concrete subjected to elevated temperatures based on decades of research.

Chapter 3 providing the characterisation of all the materials as well as experimental procedures used in this study.

Chapter 4 containing the properties of conventional concrete and environmentally friendly concrete.

Chapter 5 providing environmental impact and cost assessments to emphasise the significance of using waste materials and limited cement content in concrete mixtures.

Chapter 6 indicating how the performance of environmentally friendly concrete is affected by elevated temperatures.

Chapter 7 summarising the conclusions and recommendations for future research activities based on the findings of this study.

Chapter 8 containing the list of references.

2 LITERATURE REVIEW

2.1 INTRODUCTION

Both structural and non-structural elements can be subjected to thermal conditions owing to either normal working conditions, such as radiation shields, or accidental causes like fire, which creates a need to understand the performance of concrete subjected to these temperature circumstances (Ballim & Otieno, 2021).

It is well known that concrete behaviour is significantly affected by the strength of the matrix surrounding the aggregate particles which has a substantially lower strength than either the bulk paste or aggregate (Soutsos & Domone, 2017; Davis & Alexander, 1994; Struble et al., 1980). This Interface Transition Zone (ITZ) between the two different materials with different elastic properties is considerably more porous and therefore regarded as a weak link. Due to the inherent weakness of this bond, microcracks will start to form in the ITZ when any external load is applied to the concrete before branching to the bulk paste (Soutsos & Domone, 2017; Beushausen & Dittmer, 2015; Giaccio & Zerbino, 1998; Mindess and Alexander, 1995; Alexander, 1993; Struble et al., 1980). This mechanism is dependent on the aggregate properties, especially surface texture, shape, stiffness, and their packing density (Beushausen & Dittmer, 2015). The coarse aggregate type is therefore one of the most influential variables affecting the concrete behaviour (Giaccio & Zerbino, 1998). Alexander & Mindess (2005) state that particle size distribution of the binder and its packing ability around the aggregate surface also influence the microstructure of the ITZ.

The effect of elevated temperatures on concrete differs substantially, depending on the conditions, concrete composition and the property being investigated. The micro- and meso-structure of concrete is severely damaged by elevated temperature (Ma et al., 2015). According to Fulton’s Concrete Technology (ed. Addis, 1986), the greatest consequence of heating concrete to elevated temperatures is that it causes shrinkage of the hardened cement paste (due to desiccation) and thermal expansion of the aggregates. Furthermore, the expansion of aggregates differs considerable between aggregate types (ed. Addis, 1986). This difference causes microcracking within the concrete which leads to degradation of the ITZ between the aggregate and hardened cement paste (Crispino, 1972; Campbell-Allen & Desai, 1967).

The latest Fulton’s Concrete Technology book (ed. Alexander, 2021) refers to South African concrete exposed to elevated temperatures. However, the references used, date back as far as the early 1950’s. This chapter presents a theoretical framework on concrete subjected to elevated temperatures based on decades of research while considering both normal working conditions, such as radiation shields, and accidental causes like fire. The main focus of the chapter is on the mechanical and deformation properties of concrete at various temperature conditions and how they are affected by supplementary cementitious materials (SCMs), water/cement ratios, admixtures, natural aggregates as well as recycled aggregates.

2.2 BACKGROUND

Soutsos & Domone (2017) state that "*strength is probably the most important single property of concrete, since the first consideration in structural design is that the structural elements must be capable of carrying the imposed loads*". Strength development with time and hence the degree of hydration is dependent on both the cement composition and fineness (Soutsos & Domone, 2017). Concrete strength increases with increasing age (well beyond 28-days), as the hydration reactions are never complete (Soutsos & Domone, 2017; Alexander & Ballim, 1986). However, the rate of strength development after such time will be small. Concrete strength development is also influenced by the cement type (Alexander & Davis, 1992). Mixtures containing SCMs have slower strength development than that of an equivalent pure Portland cement mixture (Soutsos & Domone, 2017; Neville, 2011). Aggregate properties such as surface texture, shape, grading, packing ability and stiffness greatly influencing concrete strength (Soutsos & Domone, 2017; Alexander & Mindess, 2005; Giaccio & Zerbino, 1998; Davis, 1975; Kaplan, 1959a). The surface texture of the aggregate affects the structure of the transition or interface zone (Giaccio & Zerbino, 1998; Struble et al., 1980). Larger maximum aggregate sizes have a reduced surface area with a weaker ITZ leading to a slight reduction in concrete strength (Soutsos & Domone, 2017). Aggregate shape affects particle packing and internal aggregate interlock which influence the water requirements of a concrete mixture and indirectly affect the concrete strength (Alexander & Mindess, 2005). Particle shape is also an influential property affecting the tensile strength. Angular crushed aggregates produce improved tensile properties compared to rounded smooth-textured aggregates (Alexander & Mindess, 2005). Crushed aggregates can increase the strength of concrete by 15% – 20% compared to uncrushed aggregates, provided all other mix proportions are the same (Soutsos & Domone, 2017). The tensile strength of concrete is directly proportional to the elastic modulus of the aggregate (Alexander & Mindess, 2005; Alexander & Ballim, 1986; Kaplan, 1959). However, work done by Beushausen & Dittmer (2015) show that aggregate type, hence aggregate stiffness, has no influence on the splitting tensile strength of concrete. According to Davis & Alexander (1994), the type of aggregate used in concrete can cause a variation in compressive strength of 10 *MPa* – 15 *MPa*. Aggregate types producing high concrete strengths are andesite, dolerite and felsite, while some granites may produce concrete with strengths below average, due to the large variations in their mineral composition (Davis & Alexander, 1994).

The strength of concrete is generally reduced when exposed to elevated temperatures (Ballim & Otieno, 2021; Malik et al., 2021; Soutsos & Domone, 2017; Ma et al., 2015; Lau & Anson, 2006; Kaplan, 1989; ed. Addis, 1986; Roux, 1972). Whereas the strength of immature (≤ 7 days) concrete subjected to high temperatures will results in above normal strengths (Ballim & Otieno, 2021). Roux (1972) found that concrete cured for shorter periods (≤ 28 days) showed increased reduction in strength. According to Fulton's Concrete Technology (Ballim & Otieno, 2021; ed. Addis, 1986), the effect on strength of mature concrete subjected to elevated temperatures are mainly influenced by the moisture content of the concrete at time of exposure. According to Ballim & Otieno (2021), good-quality mature concrete that is thoroughly dried will experience insignificant loss in strength up to temperatures of $250°C$. Standardise testing procedures as set out in *SANS 5863 (2006)* require samples to be water-saturated before testing. Research shows a $17\% - 35\%$ increase in strength when samples were tested dry compared to equivalent water-saturated specimen (Yurtdas et al., 2004; Mills, 1960; Ross et al., 1996).

Elevated temperatures can cause an increase in strength due to increased hydration of the cement paste when moisture is available in the concrete (Kaplan, 1989). According to literature, the strength of the concrete can be increased by 25% when exposed to temperatures of up to $200°C$ (Pihlajavaara, 1972). The disruptive effects of the cement paste shrinkage opposed by aggregate expansion will slowly become more noticeable as the temperature rises to $250°C$. The result is governed by a multitude of factors, not all of them identifiable, that can either cause a strength increase, no change, or significant loss in strength. A gradual loss of concrete strength is experienced for temperature exposure up to $500°C$. According to Cather (2003), the upper limit for any useful strength retention in concrete is in the range of $550°C - 600°C$. Thereafter, the loss of strength rapidly increases, with complete loss approaching $1000°C$ (Soutsos & Domone, 2017). Strength reduces even further after concrete exposed to elevated temperature is cooled before testing (Malhotra, 1956; Roux, 1974). However, research showed that the loss of strength due to elevated temperatures tends to recover over time, not only with wetting, but also with storage at normal relative humidity (Harada et al. 1972). This phenomenon is known as autogenous recovery. There is, however, little evidence available to indicate autogenous recovery in the tensile strength of concrete when stored in air or normal temperature and relative humidity after being exposed to elevated temperatures (Schneider, 1982).

The tensile strength retained in concrete structural members after exposure to elevated temperature is of importance, specifically where fire-induced spalling may occur (Babalola et al., 2021). The effect of elevated temperature on the tensile strength is greater than the effect on the compressive strength (Hager et al., 2016; Vieira et al., 2011; Kaplan, 1989). This may be attributed to the fact that tensile strength is more sensitive to microcracking than compressive strength (Vieira et al., 2011). According to Khaliq & Kodur (2011) limited research has been conducted on tensile strength of concrete at elevated temperatures. The tensile strength of concrete may be defined by the splitting tensile strength, flexural strength, or the direct tensile strength.

The deformation behaviour of concrete has been studied since the 1970's, especially concerning the performance under fire conditions and the demand of nuclear power industries (Huismann et al., 2012). The modulus of elasticity of concrete is significantly influenced by temperature (Neville, 2011). Various investigations (Babalola et al., 2021; Malik et al., 2021; Kodur, 2014; Lau & Anson, 2006; Schneider, 1988; Marechal, 1972; Roux, 1974; Campbell-Allen & Desai, 1967) indicated a decrease in the elastic modulus with increasing temperatures due to the disintegration of cement products and the degradation of the interface between the microstructure of the hardened cement paste and the aggregate. The main factor influencing elastic modulus of concrete at elevated temperatures is the type of aggregate used (Ballim & Otieno, 2021; Kaplan, 1989; Schneider, 1988, Harada et al., 1972). Concrete comprising of low-density aggregates, crushed brick, slag, etc, perform well in high temperature conditions (Ballim & Otieno, 2021; Campbell-Allen & Desai, 1967). Other factors such as water/cement ratio, cement properties, the compaction of the mix, and the presence of microcracks all affect the modulus of elasticity of concrete (Gilbert, 1988; Mehta & Monteiro, 2006; Yurtdas et al., 2004; Schneider, 1988). The modulus of elasticity is not affected significantly by the testing conditions i.e., testing specimen while hot or after cooled to ambient temperatures (Marechal, 1972).

There is limited research on the effect of elevated temperatures on the durability performance of concrete. Ballim & Otieno (2021), stated that conventional durability testing can be performed to determine the residual durability of the concrete subjected to elevated temperatures. Lau & Anson (2006) found that concrete specimen exposed to elevated temperatures yield higher porosity corresponding with lower strengths. The loss of durability may be due to the weakened ITZ between the hardened cement paste and the aggregates (Xu et al., 2001).

2.3 CONCRETE AT ELEVATED TEMPERATURE

Chemical and physical changes such as dehydration of cement paste, aggregate decomposition, colour changes, mass loss, strength and stiffness reduction, micro-cracking and surface crazing all occur due to temperature gradients induced in concrete (Babalola et al., 2021; Khaliq & Taimur, 2018). The chemical changes that occur in concrete exposed to elevated temperatures is tabulated in **Table 2-1**.

Elevated temperatures can be divided into three zones namely (ed. Addis, 1986):

i) Ambient to 90 $^\circ C$ – *newly cast concrete*.
ii) 90 $^\circ C$ to 250 $^\circ C$ – *operating temperatures of chimneys and nuclear shielding structures*.
iii) Above 250 $^\circ C$ – *in case of a fire (accidental)*.

Table 2-1: Chemical changes in concrete exposed to elevated temperature (adapted from Cree et al., 2013; Vieira et al., 2011; Matesova et al., 2006; Cather, 2003; Schneider, 1988)

Temperature range	Process
Ambient - 100°C	Release of evaporable water
70°C – 120°C	Dissociation of ettringite
100°C – 110°C	Increased capillary porosity and micro-cracking
100°C – 300°C	Release of chemical water from *CSH*[*] gel
	Internal autoclaving hydrates non-hydrated cement particle
≥ 300°C	Significant strength reduction
	Assumed that the concrete has insufficient strength
400°C	Release of capillary water
500°C	Dissociation/decomposition of $Ca(OH)_2$ - portlandite
573°C	Transformation of α-quartz to β-quartz in siliceous aggregates
600°C	Mechanical performance severely affected
700°C	Breakdown of the *CSH*[*]
750°C – 950°C	Breakdown of carbonates and severe microcracking
	Decomposition of limestone aggregates

[*]*CSH = Calcium silicate hydrate*

The degradation of concrete due to temperature exposure is caused by the following (Soutsos & Domone, 2017; Ma et al., 2015; Kodur, 2014; Huismann et al., 2012; Lau & Anson, 2006; Cather, 2003; Kaplan, 1989; Campbell-Allen & Desai, 1967):

i) The water vapour within the concrete starts to disperse at around 100°*C* and continues as the temperature increases. Spalling of concrete is caused by the build-up of pressure due to the evaporation of water that cannot disperse fast enough. Explosive spalling can occur within minutes when low permeability, high-strength concrete is exposed to fire. Polypropylene fibres can be used in concrete mixtures to prevent explosive spalling as the fibres rapidly melt (at 160°*C* – 170°*C*) and provide pressure relief channels.

ii) Shrinkage of the hardened cement paste and thermal expansion of the aggregates initiate stresses and cracking in the ITZ leading to rapid loss in strength at around 500°*C*.

iii) There is a total loss of strength at temperatures approaching 1000°*C*.

2.3.1 Fire exposure of concrete

According to Kodur & Naser (2020), *"fire presents one of the most severe environmental conditions to which structures may be subjected"*. Devastating fires throughout history have shaped fire safety measures for structural members in the design of high-rise buildings and built infrastructure. The *ACI 216.1 (2007)* indicate temperatures reached within slabs during fire tests as illustrated in **Figure 2-1**. Addis (1986) stated that the elevated temperature zone in case of a fire is greater than 250°*C*. However, **Figure 2-1** show that for a concrete slab, a temperature of 250°*C* would only be obtained in the concrete 30 *mm* from the surface after 30 minutes of exposure. Exposure up to 90 minutes is required for the concrete to reach temperatures of 500°*C*, 30 *mm* from the surface. For a carbonate concrete slab, a temperature of 500°*C* would only be obtained 50 *mm* from the surface after 4 hours of exposure.

Strength loss mechanisms and spalling governs the behaviour of concrete in a fire. The type of exposure, permeability and tensile strength of concrete all affect the extent of spalling (Khaliq & Kodur, 2011; Cather, 2003). The inclusion of a small volume of polypropylene microfibres can efficiently increase the fire resistance of concrete and prevent explosive spalling (Beushausen & Dehn, 2021; Malik et al., 2021; Maluk et al., 2017; Soutsos & Domone, 2017; Huismann et al., 2012; Matesova et al., 2006; Kalifa et al., 2001; Kitchen, 2001). The polypropylene fibres melt at approximately 160°*C* and start to degrade at around 360°*C* creating voids that provide pressure relief channels for water vapour to escape (Soutsos & Domone, 2017; Khaliq & Kodur, 2011; Cather, 2003; Kitchen, 2001). However, this significantly increases the permeability of the concrete (Huismann et al., 2012; Kalifa et al., 2001). In conventional applications, polypropylene fibres are typically used in small quantities in the range of 0.1% – 0.5% by volume and it is suggested that 2 kg/m^3 (< 0.2% by volume) can provide resistance to elevated temperatures (van Zijl & Boshoff, 2021; Huismann et al., 2012;

BS EN 1992-1-2). Maluk et al. (2017), found that polypropylene fibres with a smaller cross-section can more effectively reduce heat-induced spalling.

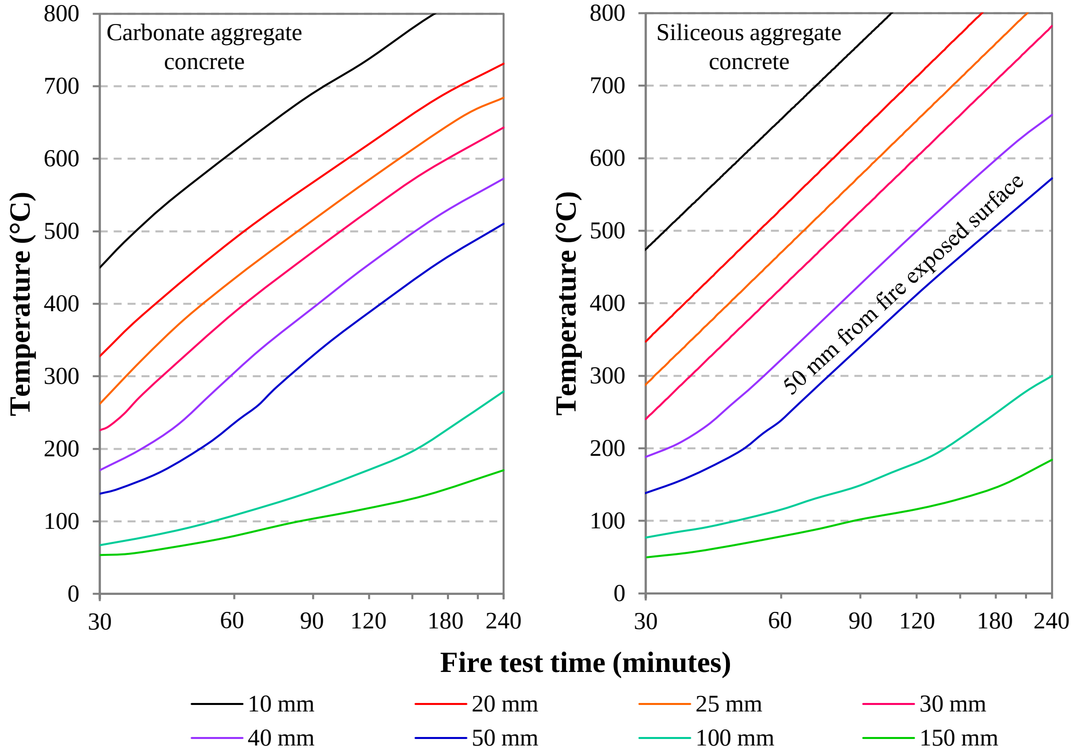

Figure 2-1: Temperatures within slabs during ASTM 119 fire tests (adapted from *ACI 216.1-2007*)

The assessment of concrete damaged by fire involves both visual and detailed aspects. Colour changes in concrete are of importance as it provides an estimation of the maximum exposed temperature and consequently the possible strength reduction (Ballim & Otieno, 2021). However, colour changes are not always visible as it is caused by the presence of iron oxide in the concrete. Colour changes, as shown in **Table 2-2**, can be expected, provided that enough iron oxide is present.

Table 2-2: Discoloration of concrete caused by elevated temperatures (Gupta & Thiruvengadam, 1993; ed. Addis, 1986)

Temperature range	Colour change
Ambient – 250°*C*	Retain its normal colour
250°*C* – 350°*C*	Pink*
300°*C* – 600°*C*	Pink to red
600°*C* – 950°*C*	Grey
≥ 950°*C*	Buff

**This colour change is of importance since below 250°C the strength is not significantly reduced.*

2.3.2 Nuclear shielding structures

It is well known that concrete is an excellent radiation shielding material (Kaplan, 1957, 1968) mainly used for containment structures in nuclear power stations, medical applications, bunkers and atomic research and testing facilities (Roxburgh & Kearsley, 2021; Mehta & Monteiro, 2006). The inner face of a reactor shield is subjected to heat transferred by convection, conduction and radiation from the reactor core (Kaplan, 1968). Operating temperatures of nuclear reactors range between $285°C$ – $650°C$, depending on the reactor type (**Table 2-3)**. Further temperature rises take place in concrete radiation shields due to neutron and gamma-radiation attenuation. The radiation shield is therefore highly influenced by temperatures and temperature gradients (Kaplan, 1958).

It is the mass of the shielding material which is important to attenuate gamma-rays and x-rays (Kaplan, 1968). The relative high density of concrete therefore makes it a suitable material, particularly if high-density concrete, containing heavyweight aggregates is used. However, high-density concrete raises construction problems. Heavyweight aggregates tend to segregate due to their high relative density. Therefore, special mixture design, shuttering and placement considerations are required when heavyweight aggregates are used (Alexander & Mindess, 2005; Kaplan, 1983; 1957). Water on the other hand is a good neutron shield, yet concrete with a high-water content may cause problems such as high shrinkage, cracking and segregation (Kaplan, 1957).

According to Kaplan (1983) the hardened cement paste shrinks when exposed to neutron radiation and the shrinkage is comparable to that caused by exposure to high temperatures. The difference in the dimensional strains between the hardened cement paste and the aggregate may cause microcracking within the concrete. Consequently, the expansion behaviour of the aggregates plays a significant role in the properties of concrete exposed to nuclear radiation (Kaplan, 1983).

Table 2-3: Operating temperature ranges of nuclear reactors (World Nuclear Association, 2023)

Reactor type	Operating temperature ($°C$)
Pressurised Water Reactor (PWR)	325
Boiling Water Reactor (BWR)	285
Pressurised Heavy Water Reactor (PHWR)	290
Advanced Gas-Cooled Reactor (AGR)	650
Light Water Graphite-Moderated Reactor (LWGR)	290
Fast Neutron Reactor (FNR)	500 - 550

2.4 EFFECT OF TEST CONDITIONS

The range of change in relative compressive strength, tensile strength, and elastic modulus of conventional concrete as a function of exposure temperature as obtained by various researchers is given in **Figure 2-2**. There is quite a high variation in the published results of concrete exposed to elevated temperatures. The effects of concrete constituents, circumstances and testing conditions when exposed to elevated temperatures are the primary reason for the large variations commonly seen in published research, which makes it difficult to compare results. According to various authors (Ballim & Otieno, 2021; Malik et al., 2021; Soutsos & Domone, 2017; Ma et al., 2015; Kodur, 2014; Cree et al., 2013; Khaliq & Kodur, 2011; Vieira et al., 2011; Lau & Anson, 2006; Zega & Di Maio, 2006; Kaplan, 1989; Schneider, 1988; Kaplan, 1983; ed. Addis, 1986), these variations can be attributed to the following conditions:

- concrete temperature at time of casting,
- mixture constituents and proportions,
- initial curing method,
- moisture content at time of testing,
- maturity of concrete at the time of exposure,
- size and shape of the element,
- exposure duration,
- “temperature history” of the concrete which includes the rate of heating and cooling,
- maximum temperature reached,
- induced temperature gradients,
- whether or not contained moisture can disperse during heating,
- testing method and condition,
- loading rate, and
- stress in the concrete during testing.

There are three steady-state test conditions for evaluating the strength of concrete at elevated temperatures, namely residual, unstressed, or stressed. The residual test method involves heating specimen to a specific temperature until a steady state is achieved. The specimen are then allowed to gradually cool down to ambient temperatures before testing. The residual strength is the post-temperature behaviour of concrete and is not a prediction of spalling. The unstressed test method involves heating the specimen to a target temperature without applying any external load. After attaining a steady state at the target temperature, the specimen is then tested until failure. In the stressed test approach, the specimen is heated to a target temperature while applying a specified preload. Once the specimen reaches a steady state at the target temperature, it is then further loaded until failure (Khaliq & Kodur, 2011; Poon et al., 2001).

Experiments performed by Roux (1972), used a heating rate of 0.41°*C/min*, while other studies used heating rates of up to 30°*C/min* (Matesova et al., 2006). Schneider (1988) state that the heating rate has insignificant influence as long as the temperature gradients in the specimen is limited to 10°*C/cm*. RILEM TC 129-MHT suggest a heating rate of 0.5°*C/min* for specimen with a diameter of 100 *mm*. Campbell-Allen & Desai (1967) maintained elevated temperatures for ten hours after reaching equilibrium temperature, while other studies only maintained the temperatures for one hour (Matesova et al., 2006; Poon et al., 2001; Xu et al., 2001). Studies maintaining equilibrium temperatures for shorter durations showed less damage to concrete. It is therefore suggested to rather maintain equilibrium temperature for a longer duration.

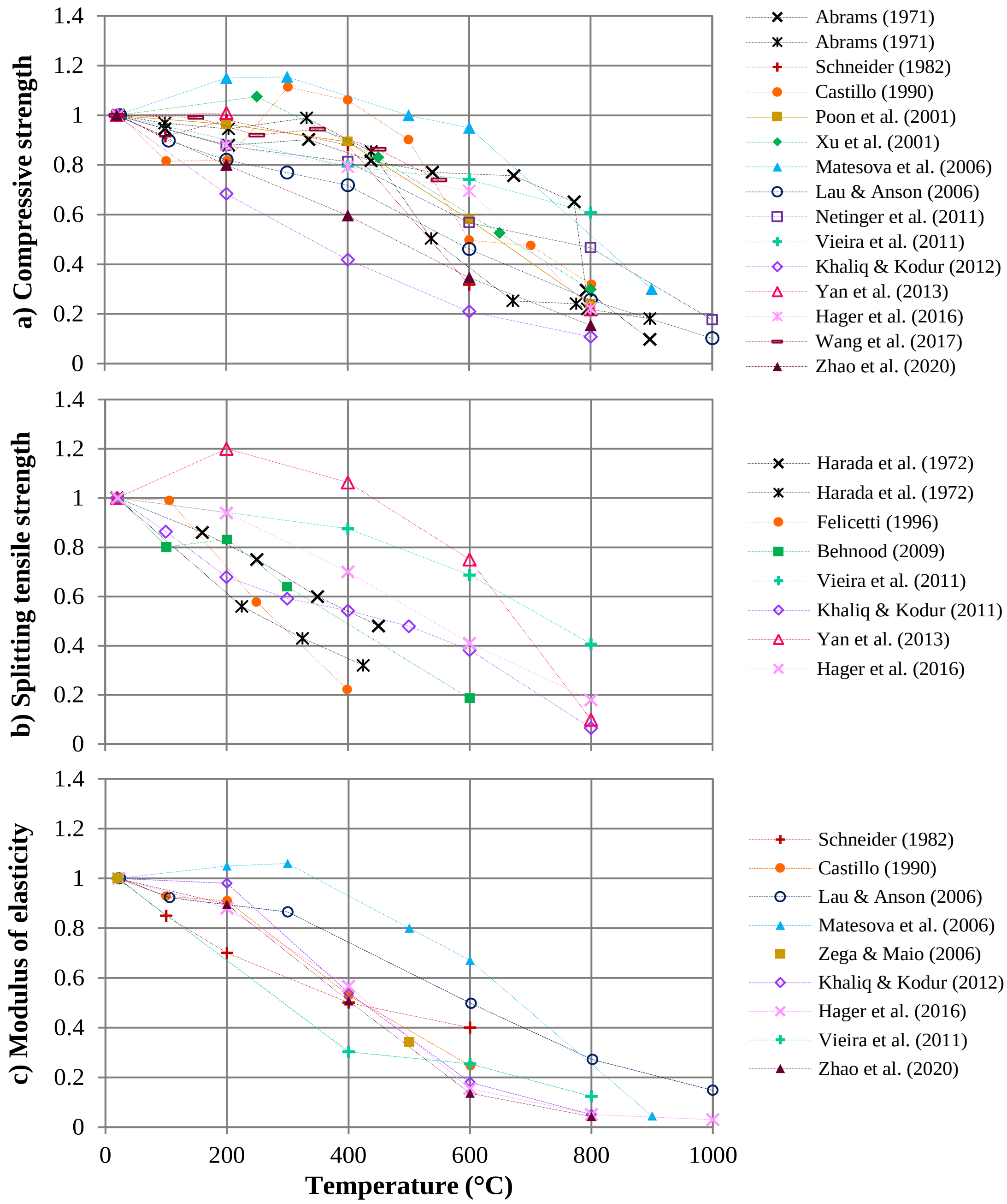

Figure 2-2: Effect of temperature on concrete strength and stiffness

2.5 SUPPLEMENTARY CEMENTITIOUS MATERIALS

Reducing the carbon footprint of cement production by using alternative binders are being investigated all over the world (Shekhovtsova et al., 2016). Supplementary cementitious materials (SCMs), such as fly ash, are beneficial in improving the sustainability of concrete by reducing the environmental footprint and lowering the amount of waste material directed to landfills (Collis, 2021; Jacobs & Kiliswa, 2021; Soutsos & Domone, 2017; Shekhovtsova et al., 2016; InEnergy 2010). Pure Portland cements accounts for approximately 90% of the total CO_2 emission per cubic metre in concrete mixtures containing no SCMs (Collis, 2021).

Fly ash, also known as pulverised fuel ash, is a by-product that can be used as a low CO_2 supplementary cementitious material (SCM) to enhance concrete properties. The fine particles of SCMs can improve not only the consistency and cohesiveness of the fresh concrete but alter the rate of hydration. Fly ash normally increases the workability of fresh concrete mixtures due to its spherical particle shape (Soutsos & Domone, 2017; Cree et al., 2013). Fly ash particles are usually smaller than 20 μm with diameters ranging from 1 μm – 100 μm (Kearsley & Wainwright, 2003). The “filler effect” of improved particle packing, aided by the spherical form of fly ash, may result in enhanced strength development (Soutsos & Domone, 2017; Kearsley & Wainwright, 2003). The addition of SCMs does however result in lower heat of hydration, which is of particular importance for mass concrete structures (Croswell & Brouard, 2021; Soutsos & Domone, 2017; Ballim & Graham, 2009; Kearsley & Wainwright, 2003; Penson & Gortzen, 1987). Subsequently, the addition of fly ash tends to reduce concrete strength up to twenty-eight days but show enhanced ultimate strength with time (Kearsley & Wainwright, 2003). Fly ash substitution for the use of site blending is normally limited to 50%, depending on the application (Jacobs & Kiliswa, 2021).

SCMs introduce additional variations into the effects of elevated temperature on concrete (Ballim & Otieno, 2021). According to Ma et al. (2015) slight variations between different cement types and their effects in concrete at normal temperatures can lead to significant differences when exposed to elevated temperatures. As mentioned, the addition of SCMs in concrete mixtures usually enhance particle packing which produce concrete with a compact microstructure and low permeability. This, however, can be detrimental to the behaviour of concrete when exposed to elevated temperatures as evaporated water cannot disperse fast enough, resulting in the build-up of pore pressures and the development of microcracks (Babalola et al., 2021). In some studies, cracking of the concrete surface was observed after exposure to elevated temperatures as a result of the rehydration of dissociated $Ca(OH)_2$ causing volume increases (Xu et al., 2001; Harada et al., 1972; Petzold et al., 1970). However, **Figure 2-3** illustrates that the addition of fly ash in concrete increases its resistance to elevated temperature up to 400°*C* (Babalola et al., 2021; Malik et al., 2021; Ballim & Otieno, 2021; Wang et al., 2017; Ma et al., 2015; Cree et al., 2013; Poon et al., 2001; Xu et al., 2001). With the addition of fly ash, $Ca(OH)_2$ is reduced in the cement paste which may lead to reduced cracking (Xu et al., 2001). Poon et al. (2001) suggests that 30% replacement of cement with fly ash is optimal to retain strength and durability after exposure to elevated temperature. However, **Figure 2-3** show that regardless of the fly ash replacement percentage, the loss of strength rapidly increases in the range of 400°*C* – 800°*C*. Furthermore, there is an insignificant difference in the resistance of the concrete to elevated temperatures after exposure to 800°*C*. Nonetheless, limited research has been conducted on the effect of elevated temperatures on the mechanical and durability properties of concrete containing SCMs (Babalola et al., 2021; Khaliq & Kodur, 2012; Xu et al., 2001).

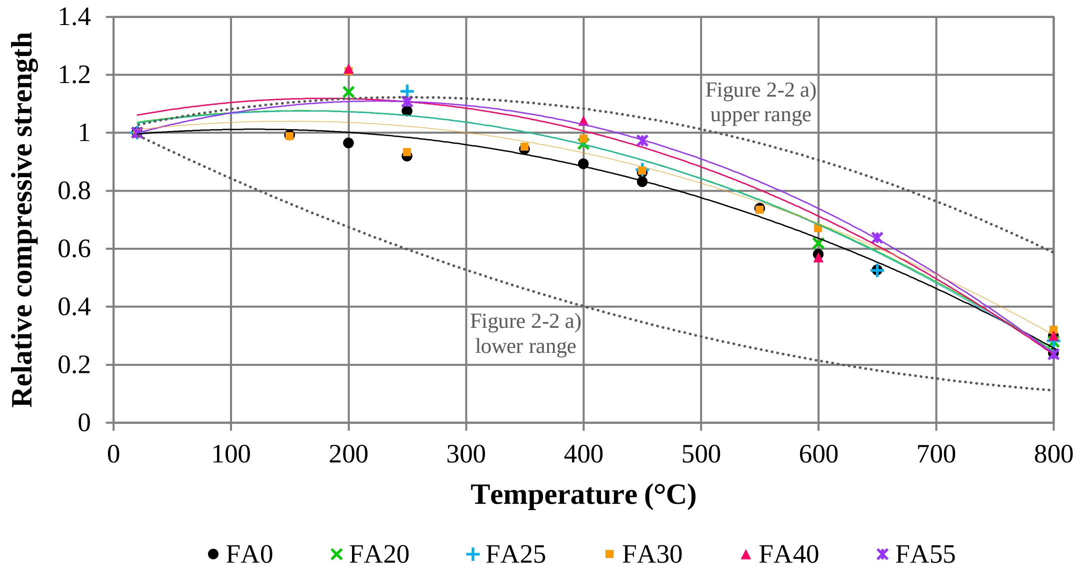

Figure 2-3: Effect of fly ash on concrete heat resistance (adapted from Wang et al., 2017; Poon et al., 2001; Xu et al., 2001)

2.6 WATER/CEMENT RATIO

Fresh water resources are becoming increasingly scarce and water usage in concrete should consequently not be considered to have a low environmental footprint (Brouard, 2021). Water/cement or water/binder ratio governs not only the strength of the concrete but influences many other concrete properties such as porosity, stiffness, and permeability (Beushausen et al., 2021; Soutsos & Domone, 2017; Davis & Alexander, 1994). According to Beushausen et al., (2021), porosity of concrete decreases with a decrease in the water/cement ratio and an increase in the degree of hydration. However, low water/cement ratios can decrease the workability of fresh concrete resulting in an increased air content. Concrete strength will decrease by 6% for each 1% of entrapped air by volume. Water/cement ratio values typically range from 0.3 – 1.0 (Soutsos & Domone, 2017). According to Soutsos & Domone (2017), at water/cement ratios lower than 0.43, maximum hydration may not occur without an available external source of water. At water/cement ratios below 0.38, hydration will stop prior to completion, even if an external source of water is available (Soutsos & Domone, 2017; Neville, 1995). Concrete mixtures with higher water/cement ratios show an increase in the hydration potential due to greater access to water and increased capillary pore volume.

The influence of water/cement or water/binder ratio on the properties of concrete exposed to elevated temperatures are more consistent throughout research (ed. Addis, 1986; Ballim & Otieno, 2021). The use of lower water/binder ratios may lead to reduced performance of concrete at elevated temperatures (Ma et al., 2015). **Figure 2-4** illustrates the limited effect of water/binder ratio (w/b) on the resistance of concrete to elevated temperature when using similar cement and aggregate types. A decrease in concrete density, due to moisture loss can take place when exposed to elevated temperatures (Kodur, 2014; Kaplan, 1989). Laneyrie et al. (2016) concluded that lower water/cement ratios result in reduced percentage mass loss at elevated temperatures.

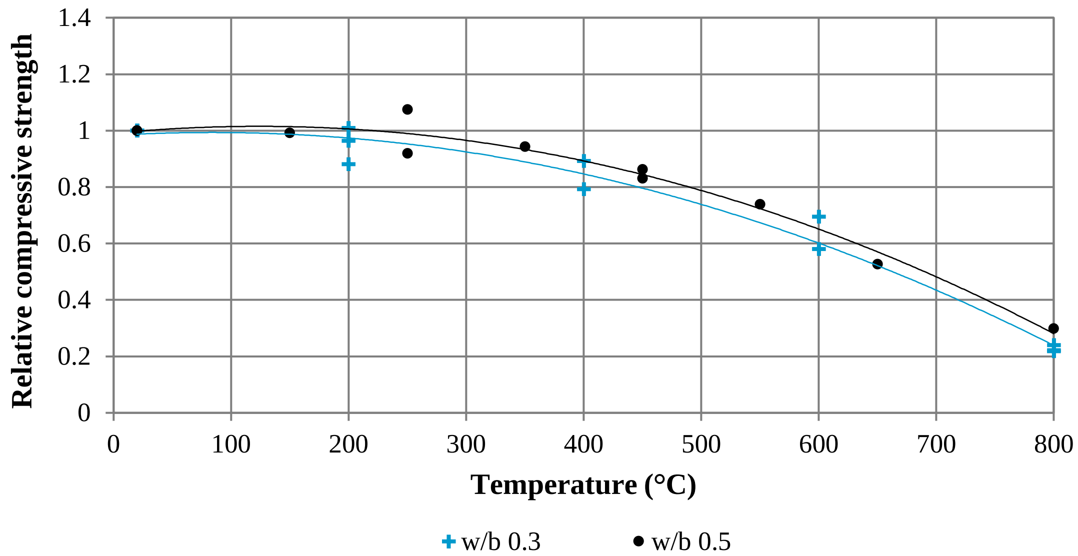

Figure 2-4: Effect of w/b ratio on concrete heat resistance (adapted from Wang et al., 2017; Hager et al., 2016; Yang et al., 2013; Poon et al., 2001; Xu et al., 2001)

2.7 ADMIXTURES

Reducing the water content of concrete mixtures by adding superplasticiser can result in a more environmentally friendly mixture. Admixtures are added to the concrete to change the fresh, early-age or hardened properties for economic gain or physical advantage (Soutsos & Domone, 2017; Cement Admixtures Association, 2012; Mehta & Monteiro, 2006). High-Range Water-Reducing Admixtures (HRWRA), also known as superplasticisers, use polycarboxylate ethers as the main chemical type. The performance of superplasticisers, usually added to concrete at rates of less than 5% by weight of the cement, depend on admixture type, binder constituents, cement composition, cement fineness and the water/cement ratio (Aitcin et al., 1994). Overdosing can result in segregation, retardation, and/or air entrainment (Brouard, 2021; Soutsos & Domone, 2017; Cement Admixtures Association, 2012). According to Aitcin et al. (1994), an optimum dosage (about 1% by weight of cement) for a cement-superplasticiser combination exists beyond which no further increase in fluidity occurs. The standard dosage of admixtures is in the range of 0.3% – 1.5% by weight of the cement, which typically reduce water content by 20% – 40% (Cement Admixtures Association, 2012). Admixtures can influence the environmental impact of concrete by (Cement Admixtures Association, 2012):

i) reducing the water content,
ii) lowering the cement content,
iii) enabling the use of recycled materials as a source of aggregate, and
iv) improving the durability.

Xu et al. (2001) showed that the addition of superplasticiser had little effect in the performance of concrete subjected to elevated temperatures (see **Figure 2-5**). Nevertheless, Babalola et al. (2021), stated that there is a need for extensive research on the effect of elevated temperatures on the behaviour of concrete containing admixtures.

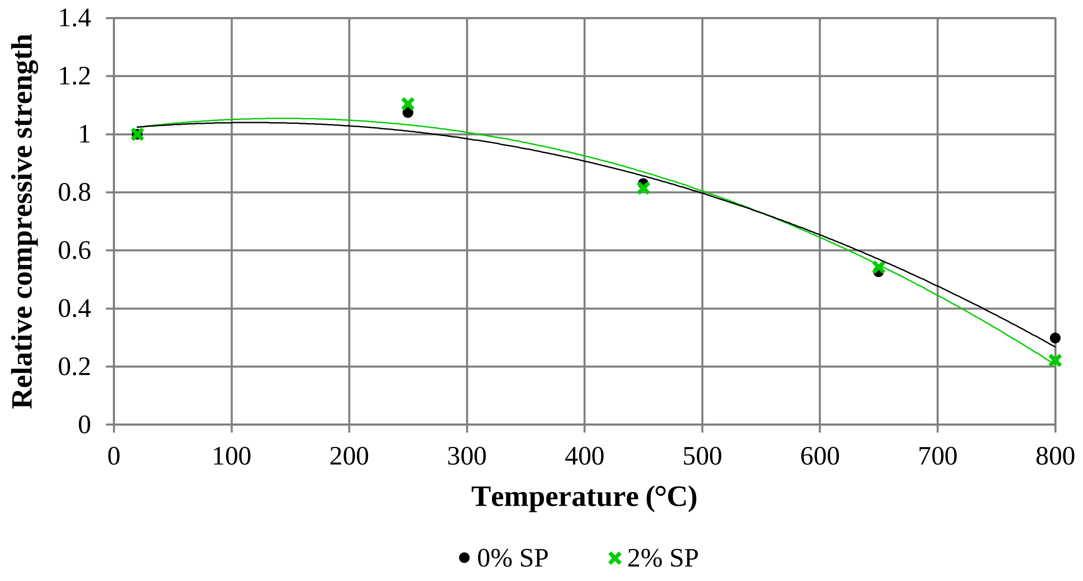

Figure 2-5: Effect of superplasticiser on concrete heat resistance (adapted from Xu et al., 2001)

2.8 AGGREGATES

Alexander & Davis (1992) stated that: "*The influence of aggregates on concrete properties was appreciated as far back as 1946 by La Rue. However, it was in many ways the work of Kaplan (1959), that brought the influence of aggregates into sharp focus*."

As coarse and fine aggregates constitute 65% – 80% of the volume of concrete, it is well known that aggregates profoundly influence the dimensional stability, properties and performance of concrete (Bonser & Alexander, 2021; Soutsos & Domone. 2017; Alexander & Mindess, 2005; Davis & Alexander, 1994; Alexander & Davis, 1992; Alexander & Ballim, 1986). The contribution of aggregates to concrete durability will continue to be of great importance for research (Alexander & Mindess, 2005). According to Bonser & Alexander (2021) aggregates can be broadly classified into three categories namely:

i) natural aggregates,
ii) manufactured aggregates, and
iii) recycled aggregates.

Aggregates acquired from natural mineral sources and only subjected to mechanical processing can be categorised as natural aggregates. Manufactured aggregates include aggregates derived from industrial by-products, such as fly ash and blast-furnace slag, as well as aggregates that are synthetically manufactured such as expanded clay and shale. Recycled aggregates are mostly derived from construction and demolition activities (Bonser & Alexander, 2021; Mehta & Monteiro, 2006).

It is important to note that the properties of the same natural aggregate type may differ significantly from one source to another and even within a single quarry over time (Alexander & Mindess, 2005; Davis & Alexander, 1994). **Table 2-4** indicates some of the available South African natural aggregates with their respective rock type and stratigraphic unit (AfriSAM TRG, 2021; Bonser & Alexander, 2021; Mehta & Monteiro, 2006; Alexander & Mindess, 2005; Davis et al., 1978). The location of these aggregates is indicated on a geological map of southern Africa in **Figure 2-6**.

The Ventersdorp Supergroup is primarily composed of an extensive collection of andesitic to basaltic lavas with related pyroclastic rocks in the form of agglomerates and tuffs with a total thickness of 1500 *m*. The Eikenhof quarry just south of Johannesburg is actively crushing these lavas of the Langgeleven Formation to generate large quantities of both coarse and fine aggregates (Brink, 1979). According to Davis et al. (1978), andesite aggregate from the Eikenhof quarry produce excellent aggregates for use in concrete.

The Great Karoo Basin is the largest and most recognised of the southern African basins. The dolerites, which stand out as a significant aspect of the Karoo basin, were intruded during a period when sedimentation in the basin had virtually ceased after the eruption of the Drakensberg basalt. Dolerite is usually fine- or medium grained in texture. The principal components of dolerite are plagioclase and pyroxene. Dolerite has been used extensively as a concrete aggregate throughout South Africa, due to its sound properties and widespread occurrence (Brink, 1983).

Substantial thicknesses of calcareous rocks, known as the dolomite Formation of the Transvaal Supergroup, are spread throughout large regions of the Transvaal. These rocks are mainly dolomitic limestones primarily composed of calcite and dolomite (Davis et al., 1978). The Chuniespoort Group mainly comprise of dolomite and chert and these together form the Malmani Subgroup with a total thickness of 1400 *m*. Dolomite have a notorious reputation to cause sinkholes and other subsidence. However, the use of dolomite (and limestone) as coarse and fine aggregate in concrete have proven advantageous, especially dolomite from the Frisco, Lyttelton and Oaktree Formation (Brink, 1979). According to Alexander & Mindess (2005), calcareous rocks of sedimentary origin (such as dolomite), produce excellent concrete aggregates. Furthermore Fulton (1961), showed that concrete containing dolomite aggregate produce high compressive and tensile strengths, low drying shrinkage, acceptable durability, high density and satisfactory adhesion between the aggregate and the cement paste.

The Rooiberg Group of the Bushveld Complex, mainly consist of red porphyritic felsite. Felsite is a very fine-grained or cryptocrystalline volcanic rock of granitic composition. Rooiberg felsite has been used extensively as a concrete aggregate in the areas around Bronkhorstspruit, Witbank, and Middleburg and in several other areas between Pretoria and Groblersdal (Brink, 1979; Davis et al., 1978). Felsite is an extremely hard and durable rock but when crushed the shape might be flaky and elongated.

Crystalline igneous and metamorphic rocks such as granite primarily consist of quartz and feldspar. Granite rocks are of widespread occurrence in South Africa (Davis et al., 1978). The Cape Granite Suite are mostly confined to the south-western Cape. The lithology of this suite is notably complex, with rocks ranging in composition from coarse-grained porphyritic biotite granites, to fines grained quartz porphyries. The Paardeberg pluton is composed of aplitic granite that ranges from fine- to medium grained in texture (Brink, 1981). According to Alexander & Mindess (2005), fresh or slightly weather granite produce excellent concrete aggregates.

The Dwyka Formation, forming the lowermost sediments of the Karroo Supergroup, consists essentially of tillites. Tillite is a highly variable glacially deposited rock, comprising of a mixture of various pebbles and boulders. Tillite is usually fine-grained in texture and primarily consisting of quartz, feldspar and clays. No general rule can be given for the behaviour tillite as a concrete aggregate and the performance of the rock in concrete depends upon the composition of the rock and the locality in which the material is quarried. However, tillite has been used extensively in the Durban region of South Africa, where it has been recognised as a satisfactory concrete material (Davis et al., 1978).

Table 2-4: Typical rock types available in South Africa (Brink, 1979; 1981; 1983)

Aggregate	Stratigraphic unit	Rock type
Andesite	Ventersdorp Supergroup	Basic igneous rock
Dolerite	Karoo Supergroup	Basic igneous rock
Dolomite	Chuniespoort Group	Sedimentary rock
Felsite	Bushveld Complex	Acidic igneous rock
Granite	Cape Granite Suite	Acidic igneous rock
Tillite	Karoo Supergroup	Sedimentary rock

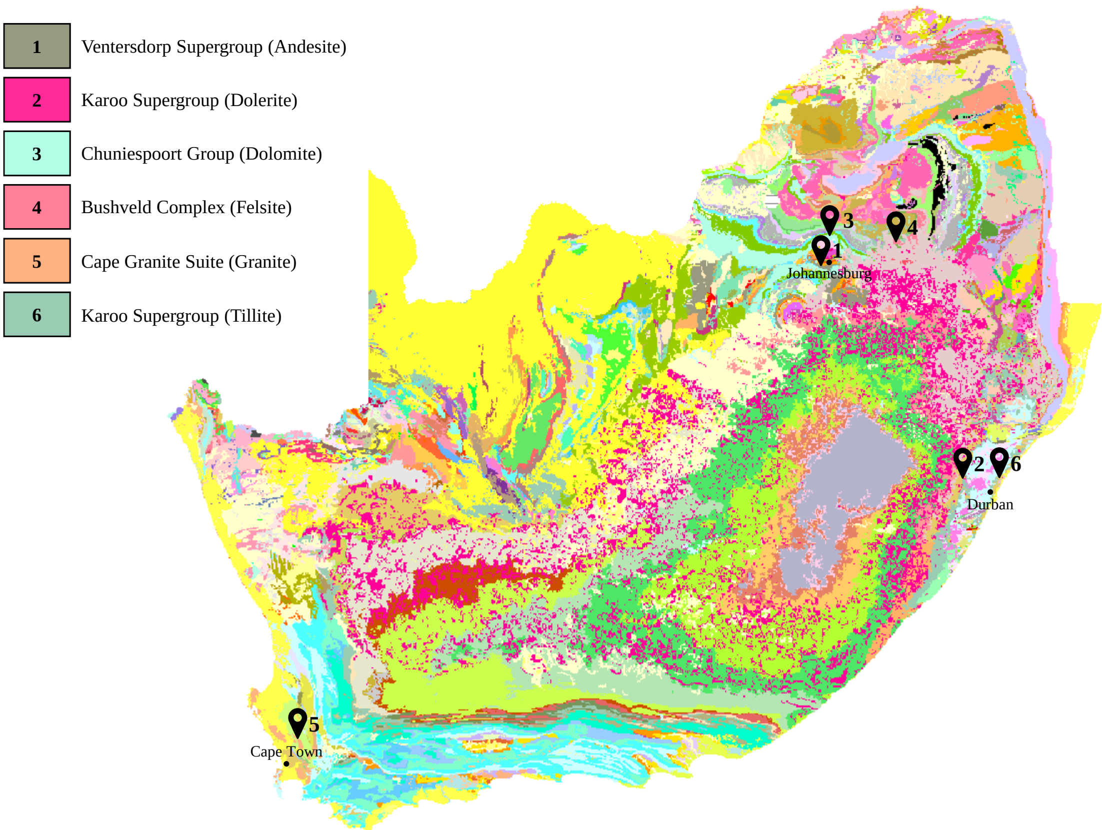

Figure 2-6: Geological map of southern Africa (Adapted from Brink, 1979; Council for Geoscience, 2023)

Construction and demolition waste typically includes concrete, bricks, metal, timber, plasterboard, asphalt, plastics, glass, rock and soil (Prajapati et al., 2023). Soutsos & Domone (2017) state that the recycling of concrete is an important aspect in ensuring sustainable construction. Recycling of concrete refer to aggregates reprocessed from crushing old concrete (Bonser & Alexander, 2021), which is known as recycled concrete aggregate (RCA). The utilisation of these aggregates in concrete can lead to sustainable construction by reducing the demand for natural aggregates (InEnergy, 2010). RCA consists of the original natural aggregate and the adhered mortar. The crushing and processing methodology to produce RCA can cause microcracks in the adhered mortar and natural aggregate itself (Prajapati et al., 2023). The use of RCA in the construction industry has been limited due the perception that its performance is inferior compared to natural aggregates. Furthermore, Prajapati et al. (2023) state that there are several challenges in the production of RCA which include:

- estimating supply of construction and demolition waste,
- on-site sorting,
- collection and transportation of waste and,
- quality control of the recycled materials.

However, it is necessary to change these negative perceptions, prioritising a purpose-driven approach over a prescriptive one.

2.8.1 Densities

Concrete density is governed by the density of the aggregates as well as the volume of entrapped or entrained air (Boshoff et al., 2021). Relative density for natural aggregates (normal-weight) ranges from 2.55 – 2.75. In special cases, aggregates with lower or higher densities are required. The density of conventional concrete produced with normal-weight aggregates is between 2200 kg/m^3 – 2450 kg/m^3. Lightweight aggregates produce lower density-concrete compared to conventional concrete. Lightweight aggregate concrete has densities as low as 2000 kg/m^3. Heavyweight aggregates have relative densities in the range of 3.4 – 6.8 and produce high-density concrete with densities exceeding 2600 kg/m^3 (Roxburgh & Kearsley, 2021; Soutsos & Domone, 2017; Mehta & Monteiro, 2006; ed. Addis, 1986). Typical relative densities of normal-weight natural aggregates readily available in South Africa are given in **Table 2-5**.

Table 2-5: Relative densities of South African aggregates

Aggregate	Source	Relative density	
		AfriSAM TRG, 2017; 2021	**Davis & Alexander, 1994**
Andesite	Eikenhof, Johannesburg	2.94	2.85 – 2.92
Dolerite	Pietermaritzburg	2.92	2.94
Dolomite	Olifantsfontein, East Rand	2.86	2.86
Felsite	Zeekoewater, Witbank	2.67	2.63 – 2.67
Granite	Rheebok, Malmesbury	2.62	2.63
Tillite	Verulam, Durban	2.69	2.68 – 2.72

2.8.2 Water absorption

The water demand of a concrete mixture is influenced by the water absorption capacity of the aggregates, and it is therefore an important property required when designing a concrete mixture. Water absorption can influence the strength, density and deformation properties of concrete. Higher drying shrinkage can be expected in concrete containing aggregates with higher water absorption values (Neville, 2011; Mehta & Monteiro, 2006; Alexander & Mindess, 2005). The water absorption for natural aggregates ranges from 1% – 3% (Soutsos & Domone, 2017), while typical South African aggregates ranges from 0.2% – 0.9% as shown in **Figure 2-7** (AfriSam TRG, 2021; AfriSam TRG, 2017; Davis & Alexander, 1994). It can be seen that the water absorption of the same aggregate type differs significantly within a single quarry over time. Consequently, these differences can influence the water demand of a concrete mixture.

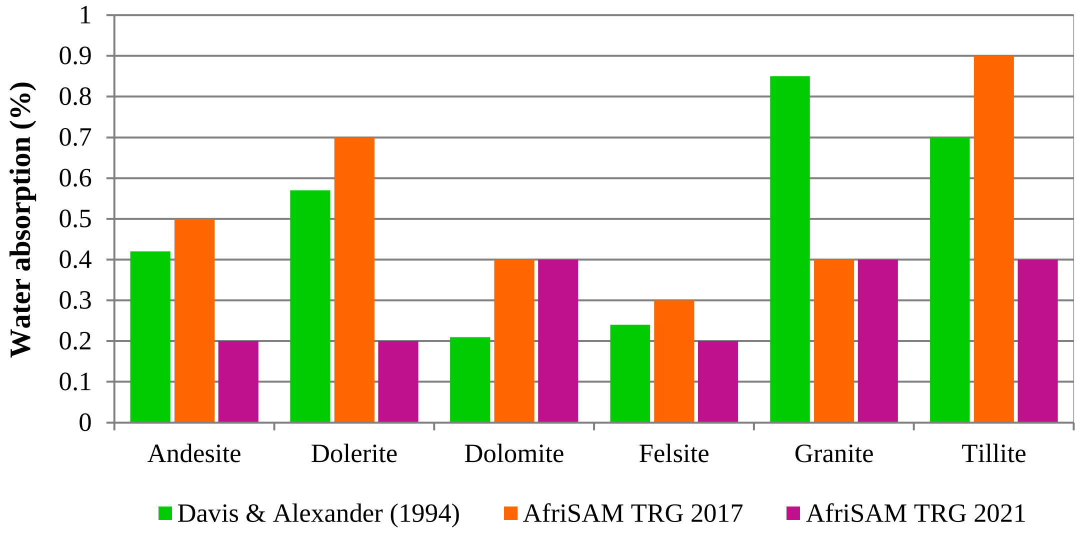

Figure 2-7: Water absorption of South African aggregates

2.8.3 Crushing strength and stiffness

According to Beushausen & Dittmer (2015), higher aggregate strength yields concrete with higher strength. Nonetheless, higher aggregate stiffness does not necessarily produce concrete with higher strength. The stiffness of aggregates does however significantly influence the elastic properties of concrete (Soutsos & Domone, 2017; Beushausen & Dittmer, 2015; Mehta & Monteiro, 2006; Alexander & Mindess, 2005; Alexander & Davis, 1992; Alexander & Ballim, 1986; Kaplan, 1959). Furthermore, Alexander & Mindess (2005), state that for conventional concrete, with compressive strength less than approximately 60 *MPa*, aggregate strength does not significantly influence the concrete strength. The aggregate crushing value (ACV) (AfriSAM TRG, 2021; AfriSAM TRG, 2017) as well as stiffness (Davis & Alexander, 1994) of typical South African aggregates are given in **Table 2-6.**

Table 2-6: Strength and stiffness of typical South African aggregates (AfriSAM TRG, 2017; 2021; Davis & Alexander 1994)

Aggregate	Source	ACV (%)	E-Value (*GPa*)
		AfriSAM TRG, 2017; 2021	Davis & Alexander, 1994
Andesite	Eikenhof, Johannesburg	4.6 – 5.9	96 – 105
Dolerite	Pietermaritzburg	11.5 – 13.7	77 – 92
Dolomite	Olifantsfontein, East Rand	12.8	109 – 118
Felsite	Zeekoewater, Witbank	9.2 – 13.3	77 – 89
Granite	Rheebok, Malmesbury	10.3 – 14.9	68 – 76
Tillite	Verulam, Durban	9.5 – 13.3	68 – 79

2.8.4 Shape and surface texture

Aggregate particle shape is an indication of the geometrical characteristics of the aggregate and can influence the void content and packing density of the aggregates in the concrete. The shape can be classified as either, round, cubic, angular, elongated or flaky. Both the coarse and fine aggregate particle shapes, significantly influence the water demand of concrete mixtures. Flaky and elongated particle shapes can entrap bleed water underneath the particles as well as increase the water requirement. Highly flaky particles can reduce the strength of concrete due to both the compaction difficulties they create as well as their greater susceptibility to fracture under stress (Bonser & Alexander, 2021; Soutsos & Domone, 2017; Mehta & Monteiro, 2006; Alexander & Mindess, 2005).

Aggregate surface texture depends on hardness, grain size, pore structure, and texture of the parent rock (Neville, 2011; Alexander & Mindess, 2005). It is difficult to define surface texture and the classification is usually based on a visual and quantitative assessment on how smooth or rough the aggregate surface has been polished or abraded. The water demand increases as the roughness of the particle surface increases due to the greater surface requiring wetting. A rougher surface texture may produce a stronger bond between the hardened cement paste and the aggregates, leading to increased strength (Bonser & Alexander, 2021; Soutsos & Domone, 2017; Mehta & Monteiro, 2006). Furthermore, the mineralogical and chemical properties of aggregates can also affect the bond. According to Neville (2011), some chemical bond may exist between the hardened cement paste and the aggregate in the case of limestone, dolomite and siliceous aggregates. **Table 2-7** indicate standard descriptions of aggregate surface texture according to *BS 812:102 (1989)*. Wright (1955) suggested that the difference of the length of the profile and the length of an unevenness line drawn as a series of chords should be determined to quantify surface texture. This method is however not widely use (Neville, 2011).

Campbell-Allen & Desai (1967) suggested that the surface texture and shape of the aggregate and the bond between hardened cement paste and aggregate (ITZ) influence the performance of concrete exposed to elevated temperatures more than what the coefficient of thermal expansion of the aggregate does.

Table 2-7: Surface texture of aggregates according to *BS 812:102* (*1989*)

Surface texture	Characteristics	Example
Smooth	Water-worn or smooth due to fracture of laminated or fine-grained rock	Dolomite, Quartzite
Granular	Fracture showing uniform size rounded grains	Sandstone, Granite
Rough	Rough fracture of fine- or medium-grained rock containing no easily visible crystalline constituents	Andesite, Basalt, Dolerite, Felsite, Greywacke
Crystalline	Containing easily visible crystalline constituents	Granite, Gabbro, Gneiss
Honeycombed	Visible pores and cavities	Brick, Clinker, Pumice
Glassy	Conchoidal fracture	Slag

2.8.5 Thermal properties

The coefficient of thermal expansion (CTE) of aggregates varies widely from about $5 \times 10^{-6}/°C$ to $15 \times 10^{-6}/°C$. The CTE of aggregate increases as their silica content increases (such as siliceous materials, quartzite and sandstone). Limestones or calcareous aggregates have lower coefficients of thermal expansion (Beushausen et al., 2021; Davis & Alexander, 1994; Bonnell & Harper, 1951). The thermal behaviour of typical South African aggregates is given in **Table 2-8**.

Table 2-8: Coefficient of thermal expansion of aggregates (Davis & Alexander, 1994)

Aggregate	Source	CTE of aggregate ($\times 10^{-6}/°C$)
Andesite	Eikenhof, Johannesburg	7.4
Dolerite	Pietermaritzburg	7.2
Dolomite	Olifantsfontein, East Rand	9.2
Felsite	Zeekoewater, Witbank	9.2
Granite	Rheebok, Malmesbury	9.7
Tillite	Verulam, Durban	6.6

According to Ballim & Otieno (2021), neither the shrinkage of cement paste nor the expansion of aggregates is directly proportional to the rise in temperature. As mentioned, the thermal expansion of different aggregate types differs significantly. Concrete exposed to elevated temperatures in service should therefore preferably contain aggregate with a low coefficient of thermal expansion. According to Addis (1986), the traditional order of preference of aggregate type for concrete exposed to elevated temperatures is:

i) calcareous (limestone, dolomite)
ii) felspathic (andesite, basalt, dolerite, gabbro)
iii) granites
iv) siliceous (quartz, quartzite)

The superior performance of calcareous aggregate concrete under elevated temperatures compared to that of siliceous aggregate concrete was confirmed by various researchers (Ma et al., 2015; Netinger et al., 2011; Cather, 2003). When concrete with a low thermal expansion is exposed to elevated temperatures, the mass loss is less significant (Kaplan, 1989). Calcareous aggregates generally have lower coefficients of thermal expansion leading to lower internal stresses on heating (Cather, 2003). However, other studies (Browne & Blundel, 1972; Campbell-Aleen & Desai, 1967) have questioned the performance of calcareous aggregates under elevated temperature and suggested that felspathic and granite aggregates should be given preference. According to Neville (2011), the loss of concrete strength upon exposure to elevated temperature for aggregates not containing silica is significantly reduced. According to Kodur (2014), mass reduction is minimal for both carbonate and siliceous aggregate concretes for temperatures up to about 600$°C$. However, carbonate aggregate concrete experience higher percentage mass loss at temperatures above 600$°C$. This is attributed to dissociation of dolomite in carbonate aggregates (Malik et al., 2021; Kodur, 2014). **Figure 2-8** illustrate the performance of concrete containing different aggregate types as a function of temperature.

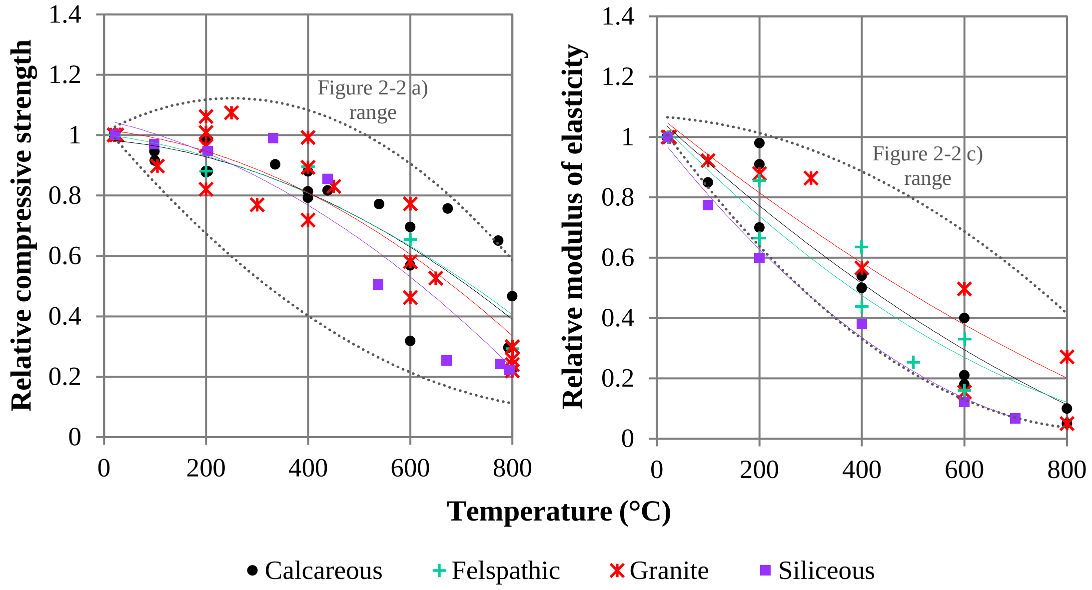

Figure 2-8: Effect of aggregate type on concrete heat resistance (adapted from Hager et al., 2016; Yang et al., 2013; Khaliq & Kodur, 2012; Netinger et al., 2011; Lau & Anson, 2006; Poon et al., 2001; Xu et al., 2001; Schneider, 1982; Abrams, 1971)

2.9 RECYCLED AGGREGATE CONCRETE

Over the last half century, various sustainable efforts have been investigated to the reduce carbon footprint and environmental impact of the construction industry. According to the International Energy Agency (2018), aggregate productions yield small quantities of CO_2 when compared to cement production. In South Africa, only 3% of the total CO_2 emissions per cubic metre of concrete is due to aggregate production (Collis, 2021). However, Collis (2014, 2021) states that the true environmental impact of aggregate production are unquantifiable by life-cycle assessments studies and environmental rating systems since new aggregate quarries are located further from cities due to urbanisation and rapid population growth. This causes increased transportation distances, which leads to increased pressure on the road infrastructure. Several alternative aggregate sources are used in concrete as a response to reduce the environmental footprint thereof (Bonser & Alexander, 2021). The use of recycled materials, from construction and demolition waste, as aggregates may present as a resource-saving alternative to preserve natural resources, decrease landfill waste, and encourage sustainability (Prajapati et al., 2023; Bonser & Alexander 2021; Laneyrie et al., 2016; Alexander & Mindess, 2005; Buck, 1977). Furthermore, according to Prajapati et al. (2023), the use of recycled materials for new concrete production promotes "*a circular economy where products at the end of their life cycle are not considered as waste, but as potentially useful resources that can be repurposed and recycled for new applications, thus reducing the environmental impact of disposal as well as contributing to economic growth*".

According to InEnergy (2010), RCA are not widely used in South Africa due to production costs involved relative to natural aggregates. However, a recent study by Ohemeng & Ekolu (2020) concluded that production of South African RCA is more cost effective (40% reduction in cost) and environmentally beneficial (97% increase) compared to natural aggregates. According to Mehta & Monteiro (2006), Michigan State Department of Transportation reported that reprocessing an existing concrete pavement was more cost effective than using new materials. RCA can thus offer a cost-effective and environmentally friendly alternative to natural aggregates, if the quality and performance of the concrete remains equivalent to natural aggregate concrete (Tosic et al., 2021; Ohemeng & Ekolu, 2020; Silva et al., 2015; Cree et al., 2013; Buyle-Bodi & Hadjieva-Zaharieva, 2002).

Natural aggregates can replace recycled aggregates in different quantities to produce RAC. These substitutions can be either by volume or by mass (Tosic et al, 2021). A mixture of RCA and natural aggregate is typically used (Soutsos & Domone, 2017). **Table 2-9** indicate the wide range of national standards and recommended values for the replacement of natural aggregate with RCA as used by different countries (Ram et al., 2019). South African standards do not yet address the use of RCA as replacement for natural aggregates. The coarse RCA replacement of natural aggregates in structural concrete is mainly limited to 20% – 25%. However, there are standards that permit 100% RCA replacement. Using 100% RCA replacement can effectively reduce demolition wastage and solve the shortage of natural aggregates (Zhao et al., 2020; Kearsley & Mostert, 2012). The replacement of natural aggregate with coarse RCA has gained wider acceptance than fine RCA. Soutsos et al. (2011) recommend that the fine aggregate replacement should be limited to less than 30%. Furthermore, the acceptable range for water absorption of coarse RCA in structural concrete is between 5% – 10%, with only Japan and China that specify water absorption percentages below 3%.

The source of the reprocessed aggregate will significantly influence the physical, mechanical and durability properties of the recycled aggregate concrete (RAC). RAC have the following properties in comparison to natural aggregate concrete (Tosic et al., 2021; Soutsos & Domone, 2017; Silva et al., 2015; Soutsos et al., 2011; Buyle-Bodi & Hadjieva-Zaharieva, 2002; McNeil & Kang, 2013; Xiao et al., 2013; Yang et al., 2008; Wickins, 2013; ed. Hansen, 1992; Buck, 1977):

i) high water absorption,
ii) increased water requirement,
iii) reduced workability with time,
iv) lower density,
v) higher porosity,

vi) higher creep and drying shrinkage,

vii) reduced compressive strength and modulus of elasticity,

viii) similar or improved splitting tensile strength, and

ix) higher air permeability, water sorptivity, chloride conductivity and carbonation with depth.

Table 2-9: Recommended replacement percentages of RCA (Adapted from Prajapati et al., 2023; Ram et al., 2019)

Country	Application	Maximum allowable replacement (%)		Water absorption (%)
		Coarse	Fine	Coarse
Australia (CCAA 2008)	Grade 1 (40 *MPa*)	30	-	-
	Grade 2 (25 *MPa*)	100	-	
China (JGJ/T 240 2011)	Structural concrete	100	-	≤ 3
Denmark (DCA 1995)	Structural concrete (50 *MPa*)	100	20	-
Germany (DAfStB 1998)	Structural concrete (35 *MPa*)	25	-	< 10
	Non-structural concrete	-	-	< 20
Hong Kong (HKBD 2009)	Structural (≥ 30 *MPa*)	20	-	≤ 10
	Non-structural (≤ 20 *MPa*)	100	-	
India (IS 383:2016)	Plain Concrete	25	25	< 10
	Reinforced Concrete	20	20	
	Non-structural (≤ 15 *MPa*)	100	100	
Japan (JIS 5021 2005)	55 *MPa*	-	-	≤ 3
Netherland (NEN-5905,2005)	Structural concrete (50 *MPa*)	100	100	-
Portugal (LNEC E-471 2006)	Structural concrete (50 *MPa*)	25	-	7
	Non-structural concrete	-	-	
South Korea (KS F 2573 2011)	27 *MPa* (Coarse)	30	-	≤ 5
	21 *MPa* (Fine)	-	30	-
Spain (EHE-08, 2010)	Structural concrete (30 *MPa*)	20	-	7
	Non-structural concrete	100	-	
Switzerland (OT – 70085 2006)	(No limit)	100	100	-

RCA tend to increase the required water demand of a concrete mixture due to their porosity, voids and/or absorption properties. To allow for the high-water absorption of RCA, aggregates should either be pre-saturated prior to mixing or additional water should be added to the mixture (Tosic et al., 2021; Kearsley & Mostert, 2012; Wickins, 2013; Xiao et al., 2013; Vieira et al., 2011; Buyle-Bodi & Hadjieva-Zaharieva, 2002). The high-water absorption of RCA as well as their irregular particle shape and rough surface textures influence the fresh and hardened concrete properties (Yang et al., 2008). The recycled sands have even higher water absorptions, which increases the water demand in the fresh concrete further (Soutsos & Domone, 2017). Recycled sands contain a considerable amount of the old cement paste which results in an increase in drying shrinkage of the new concrete (Bonder & Alexander, 2021; Alexander & Mindess, 2005). These sands should therefore be evaluated and compared with recognised aggregate standards before use in concrete (Bonder & Alexander, 2021).

Silva et al. (2015) stated that there are substantial differences in literature on the compressive strength reduction of RAC compared to equivalent natural aggregate concrete. This can be attributed to the large number of variables involved with RAC, namely aggregate type, size and quality. However, several other studies (Silva et al., 2015; ed. Hansen, 1992; Yang et al., 2008) observed RAC with parallel strength development to that of natural aggregate concrete. Tosic et al (2021), stated that reduced strengths (and workability) of RAC relative to the equivalent natural aggregate concrete are generally observed when the same effective water/cement ratio is used for both mixtures. Therefore, the strength of the RAC is mostly depended on the strength of the new cement paste and not as much on the strength of source material (Silva et al., 2015). RAC exhibits large deformation differences relative to natural aggregate concrete. This may be attributed to the lower stiffness of RCA, therefore, offering less restraint to shrinkage. Increased drying shrinkage is experienced with an increase in RCA replacement levels (Tosic et al., 2021; Yang et al., 2008). The modulus of elasticity for RAC is normally lower than that of conventional concrete, however there is a significant difference in reported results due to usage of different aggregates in each study (McNeil & Kang, 2013).

As with conventional concrete, RAC subjected to elevated temperatures experiences deterioration in its physical, mechanical and durability properties (Babalola et al., 2021; da Silva et al., 2020; Salahuddin et al., 2019; Pliya et al., 2019; Khaliq & Taimur, 2018; Gales et al., 2016; Laneyrie et al., 2016; Zega & Di Maio, 2006). The strength and stiffness of RAC as a function of temperature obtained by various researchers is given in **Figure 2-9**. It can be seen that the range of strength and stiffness results falls within the limits of normal aggregate concrete as obtained from **Figure 2-2 a)** and **Figure 2-2 c)** respectively. However, the decline in strength and stiffness with increasing temperature seems to be greater compared to normal aggregate concrete.

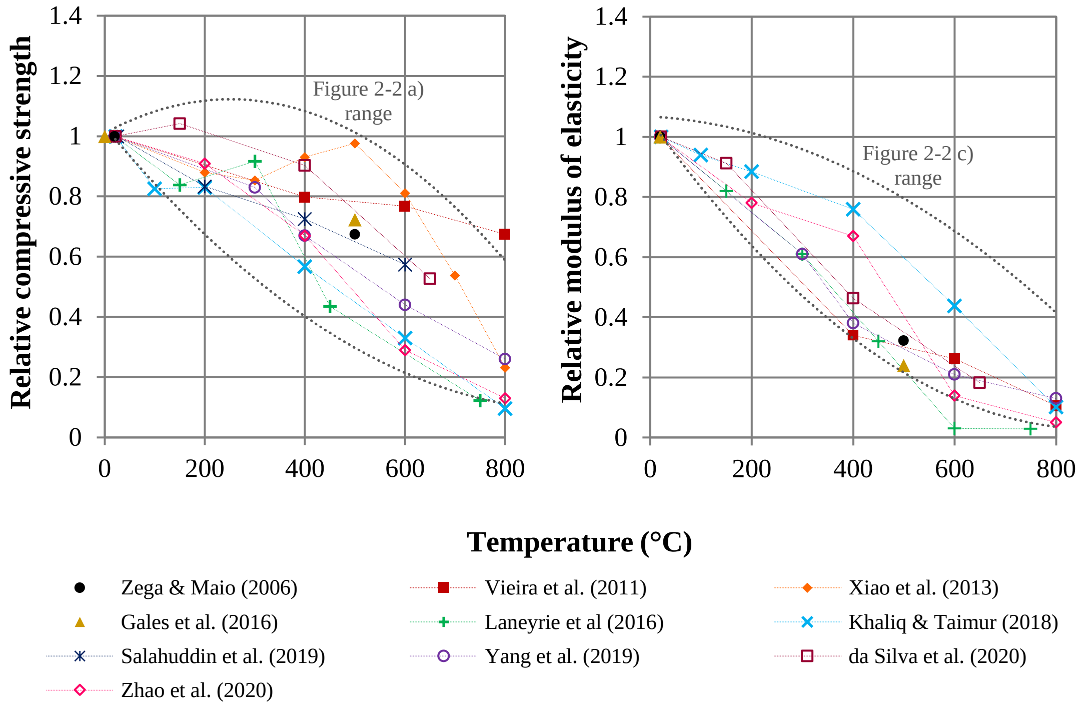

Figure 2-9: Strength and stiffness properties of RAC as a function of temperature

Research studies show opposing results when considering RAC after exposure to elevated temperatures. Furthermore, the variability of the source and nature of the RCA makes it extremely challenging to compare these results (Laneyrie et al., 2016). Some experimental studies found that the residual mechanical and durability properties of the RAC subjected to elevated temperatures are approximately the same as that of conventional concrete (da Silva et al., 2020; Salahuddin et al., 2019; Yang et al., 2019; Laneyrie et al., 2016; Vieira et al., 2011), while other studies either concluded that RAC subjected to elevated temperatures performed better relative to natural aggregate concrete (Babalola et al., 2021; Zhao et al., 2020; Khaliq & Taimur, 2018; Xiao et al., 2013) or that the performance of RAC diminished after exposure to elevated temperature (Pliya et al. 2019; Yang et al., 2019; Gales et al., 2016; Laneyrie et al., 2016). The improved performance of the RAC was attributed to the fact that the thermal expansion of the RCA is similar to that of new cement paste, subsequently lowering the formation of microcracks at elevated temperatures (Babalola et al., 2021; Zhao et al., 2020; Khaliq & Taimur, 2018). The reduced performance was attributed to the weak ITZ between the old cement paste (forming part of the RCA) and the new hardened cement paste. Other factors such as induced microcracking during sourcing and reprocessing, and the stress and loading history of the reprocessed concrete's service life may also contribute to a weakened performance (Gales et al., 2016). Even though a slight decrease in strength may occur at elevated temperatures, high replacement levels of RCA reduce the risk of explosive spalling in concrete elements (Zhao et al., 2020; Pliya et al., 2019; Yang et al., 2019). RAC also exhibits greater mass loss when subjected to elevated temperatures due to the higher water absorption of the aggregates (da Silva et al., 2020; Khaliq & Taimur, 2018; Laneyrie et al., 2016). The addition of fly ash may significantly contribute to improve the behaviour of RAC at elevated temperatures (Babalola et al., 2021).

2.10 DEFORMATION PROPERTIES OF CONCRETE

The deformation of concrete is caused by not only environmental effects (moisture movement and heat) but also by applied stress (Soutsos & Domone, 2017). This section contains a general view on the deformation properties of concrete which include elasticity, thermal movement and drying shrinkage. The effect of elevated temperatures on the elastic properties is also discussed.

2.10.1 Elastic deformation

The major factors influencing elastic modulus of concrete are (Soutsos & Domone, 2017; Alexander & Mindess, 2005; Davis & Alexander, 1994; Alexander & Davis, 1992; Grills & Alexander, 1989; Alexander & Ballim, 1986; Kaplan, 1959):

i) strength and stiffness of the paste,
ii) stiffness of the aggregate,
iii) Interface Transition Zones (ITZ).

Concretes with equal strength may yield different elastic moduli if different aggregates or different aggregate/cement ratios are used. Moreover, concrete with different mixture compositions may yield similar elastic moduli but with different compressive strengths (Alexander & Mindess, 2005). The elastic modulus of concrete will slightly increase with increasing concrete strength (Alexander & Davis, 1992; Grills & Alexander, 1989). Therefore, lower water/cement ratios and increased age both influence the modulus of elasticity (Sun & Fanourakis, 2022; Soutsos & Domone, 2017). There is thus no simple correlation between strength and elastic modulus of concrete (Soutsos & Domone, 2017; Davis & Alexander, 1994; Kaplan, 1959b).

A few authors found no significant difference in the modulus of elasticity of blended cement concretes compared to pure Portland cement concretes when proportioned for equivalent twenty-eight-day strength (Sun & Fanourakis, 2022; Ghosh & Timusk, 1981). Conversely, Alexander and Milne (1995), stated that fly ash blends commonly yield slightly lower modulus of elasticity values. Slower hydration is expected in blended cement concretes which may result in lower early-age modulus of elasticity and strengths (Wainwright and Tolloczko 1986).

Table 2-10 clearly indicates that the selection of aggregate type, hence aggregate stiffness plays a significant role in the elastic behaviour of concrete. Andesite aggregates provide higher concrete elastic modulus values than most other aggregate types, except for dolomite and some dolerites (Sun & Fanourakis, 2022; Beushausen & Dittmer, 2015; Davis & Alexander, 1994; Alexander & Davis, 1992). Dolomite aggregates produce concrete with remarkably high modulus of elasticity (Davis & Alexander, 1994), while granite concrete tends to produce a low modulus of elasticity due to the slightly weathered nature (Sun & Fanourakis, 2022; Bonser & Alexander, 2021; Davis & Alexander 1994; Alexander & Davis, 1992; Davis et al., 1978; Kaplan, 1959b). However, granite may produce concrete with an elastic modulus higher than dolerite (Davis & Alexander, 1994).

Table 2-10: Influence of aggregate type on the elastic modulus of concrete (Alexander & Davis, 1994)

Aggregate	Source	E-Value (*GPa*)
Andesite	Eikenhof, Johannesburg	37.5 – 45.0
Dolerite	Pietermaritzburg	28.0 – 36.5
Dolomite	Olifantsfontein, East Rand	35.0 – 51.0
Felsite	Zeekoewater, Witbank	31.0 – 42.0
Granite	Rheebok, Malmesbury	31.0 – 36.0
Tillite	Verulam, Durban	25.5 – 39.0

2.10.2 Thermal movement

There is a need to research the thermal movement of concrete due to the extensive use of nuclear reactors around the world (Ballim & Otieno, 2021). The thermal movement of concrete is "*the product of the temperature change times the coefficient of thermal expansion*" (Brooks, 2003).

The Coefficient of Thermal Expansion (CTE) is a measure of the strain in a material due to a unit change in temperature (Ballim & Otieno, 2021) and is generally assumed to be $10 \times 10^{-6}/°C$. The mix composition, especially aggregate type, and moisture condition at the time of temperature change ($-30°C$ to $65°C$), greatly influence the CTE (Alexander & Mindess, 2005; Brooks, 2003). The influence of other factors such as cement type, curing methods, age, aggregate volume concentration and concrete strength are much less substantial (Alexander & Mindess, 2005; Davis & Alexander, 1994; Bonnell & Harper, 1951). Ballim & Otieno (2021), stated that it is difficult to experimentally determine a single definitive value for CTE due to the complex inter-relationships of the concrete constituents. Consequently, a range of possible values are normally given.

Before discussing the influence of aggregate type on the CTE, the importance of moisture content should be considered. Roux (1972) concluded that there is a significant difference in the CTE of concrete in a partially dry state compared to concrete in a fully saturated state. Cement pastes yield a maximum CTE at some intermediate moisture content and lower coefficients in both the saturated and oven-dry state. Therefore, partially moist concretes also yield expansion coefficients higher than those in the saturated or completely dry conditions (Beushausen et al., 2021; Davis & Alexander, 1994; Bonnell & Harper, 1951). However, since the aggregate occupies 65% – 80% of the total concrete volume, the effect of moisture content on the CTE is significantly reduced in concrete. A constant approximation for the CTE of concrete is normally deemed to be sufficient over all humidities (Soutsos & Domone, 2017).

According to Alexander & Mindess (2005) aggregate significantly influence the dimensional stability and thermal properties of concrete. Typical CTE values for aggregates, hardened cement paste and concrete are given in **Table 2-11**. Aggregates have lower CTE values and therefore restrain the thermal movement of the hardened cement paste. The difference in CTE between the various constituents may lead to the development of internal stresses in the concrete. This is significant when concrete is exposed to elevated temperatures and can influence concrete durability due to development of microcracks (Alexander & Mindess, 2005). It is therefore recommended that low thermal coefficient materials should not be used in circumstances where concrete is exposed to large thermal cycles or where fire resistance is required.

Limited data is available on the coefficient of thermal expansion of South African concrete and the available values mostly date back to the early 1990's. **Table 2-12** presents CTE values for concrete containing some of the aggregate types typically used in South Africa**.**

Table 2-11: Typical thermal expansion coefficient values (adapted from Soutsos & Domone, 2017; Davis & Alexander, 1994; Davis, 1975)

Material	CTE ($\times 10^{-6}/°C$)
Aggregates [1]	5 – 15
Hardened cement paste [2]	10 – 22
Concrete [1]	4 – 12

Depending on [1] aggregate type; [2] moisture condition

Table 2-12: Effect of aggregate type on coefficient of thermal expansion of concrete (Davis & Alexander, 1994; Alexander 1990; Davis, 1975)

Aggregate	Source	CTE of concrete ($\times 10^{-6}/°C$)
Andesite	Eikenhof, Johannesburg	7.4 – 7.5
Dolerite	Pietermaritzburg	6.0 – 8.1
Dolomite	Olifantsfontein, East Rand	7.5 – 9.2
Felsite	Zeekoewater, Witbank	9 – 9.2
Granite	Rheebok, Malmesbury	6.4 – 9.7
Tillite	Verulam, Durban	6.6 – 8.5

2.10.3 Shrinkage

Concrete members undergo volume changes in both the fresh and hardened states throughout their service life. Volume changes as a result of loss of moisture leads to drying shrinkage in the hardened cement paste. The rate of moisture loss and corresponding strain response is usually time dependent (Beushausen et al., 2021). Mixture proportions, aggregate properties, constituents, element size and shape can all influence drying shrinkage of concrete.

The drying shrinkage of cement paste is much higher than that of conventional concrete, due to the restraining effect of the aggregate which is dimensionally stable under varying moisture conditions (Soutsos & Domone, 2017). Therefore, higher aggregate volume concentrations and stiffness will produce concrete with lower shrinkage (Beushausen et al., 2021; Soutsos & Domone, 2017; Brooks, 2003; Davis & Alexander, 1994; Alexander & Ballim, 1986). An 80% – 95% reduction in the paste shrinkage can be expected for conventional concrete with aggregate volume concentrations between 65% – 75% (see **Table 2-13**) (Beushausen et al., 2021; Soutsos & Domone, 2017; Alexander & Mindess, 2005). Concrete drying shrinkage values can vary between 30% – 40% of the paste shrinkage depending on the aggregate type used in concrete mixtures with similar workability and strength (Alexander 1993). However, Hobbs & Parrott (1979) found that the concrete shrinkage is effectively independent of aggregate stiffness for water/cement ratios greater than 0.4. Concrete with reduced water/cement ratios are normally more impermeable causing lower rates of moisture loss, hence reduced shrinkage (Beushausen et al., 2021; Soutsos & Domone, 2017; Brooks, 2003). Reduced water/cement ratios can however result in higher early-age shrinkage (Alexander, 1993; Alexander & Ballim, 1986). It is important to note that the original water content (by mass) of a concrete mixture is directly proportional to the shrinkage of the concrete (Hobbs & Parrot, 1979; Mears & Hobbs, 1972). Sands with a low water demand generally yield concrete with low shrinkage characteristics (Davis et al., 1979). According to Davis & Alexander (1994), dolomite concrete typically exhibits low drying shrinkage, whereas dolerite and tillite aggregates produce concrete with higher drying shrinkage. This corresponds with Brooks (2003), who stated that using igneous rocks of doleritic types may increase shrinkage of concrete substantially.

Cement composition and properties as well as the addition of fly ash have insignificant effects on the drying shrinkage of concrete, compared to pure Portland cement concrete with equivalent strengths and similar exposure conditions (Soutsos & Domone, 2017).

The geometry (size and shape) of concrete elements determines the rate of moisture loss, and hence the magnitude of drying shrinkage. Consequently, comparing drying shrinkage measurements from different research articles is considered complicated (Soutsos & Domone, 2017; Mehta & Monteiro, 2006). The size effect can be expressed as the average drying path length (volume/surface ratio). Increased volume/surface ratio would indicate decreased drying shrinkage. The shape effect is much smaller and is normally neglected (Brooks, 2003).

As mentioned, the thermal shrinkage gradients between the aggregates and the cement paste when concrete is exposed to elevated temperatures results in microcrack development. The shrinkage of the cement paste is caused by the loss of bound capillary water leading to tensile stress generation which promotes spalling behaviour. Spalling of concrete is caused by the build-up of pressure due to the evaporation of water that cannot disperse fast enough in combination with differential thermal stresses. When specimen are preheated before exposure to elevated temperature, less moisture will be present and spalling of concrete due to evaporation will not take place (Malik et al., 2021; da Silva et al., 2020; Huismann et al., 2012; Lau & Anson, 2006; Kalifa et al., 2001).

Table 2-13: Typical drying shrinkage (Bonser & Alexander, 2021)

Material	Shrinkage ($\mu\mathcal{E}$)
Hardened cement paste	2 500 – 3 000
Mortar	600 – 1 200
Concrete	300 - 800

2.11 SUMMARY

The production and development of environmentally friendly construction materials raise concerns about their thermal behaviour as nuclear radiation shields face high levels of external heat. The greatest consequence of heating concrete to elevated temperatures is that it causes shrinkage of the hardened cement paste (due to desiccation) and thermal expansion of the aggregates. Based on the literature review, limited research has been conducted on the mechanical and deformation properties of concrete exposed to high temperature, especially on how the behaviour is affected by supplementary cementitious materials, water/cement ratios, admixtures, natural aggregates as well as recycled aggregates. Hence, the main focus of this study was to investigate how different concrete constituents influence the behaviour of concrete when subjected to elevated temperatures.

Additionally, there is no consensus in published results of concrete exposed to elevated temperatures. The effects of concrete constituents, circumstances and testing conditions when exposed to elevated temperatures are the primary reason for the large variations seen in the findings of previous research conducted, which makes it difficult to compare results. It was therefore decided to test all specimen under residual steady-state testing conditions. The effect of various testing circumstances were not considered in this study.

It is well known that the strength of concrete is generally reduced when exposed to elevated temperatures. This may be attributed to the degradation of the ITZ and development of microcracks. However, based on existing literature, the strength tends to recover over time and this is known as autogenous recovery. Exploring the impact of testing specimen in dry conditions versus saturated conditions after exposure to high temperature would be an intriguing area of investigation. Good-quality mature concrete that is thoroughly dried will experience insignificant loss in strength up to temperatures of 250°*C*. The disruptive effects of the cement paste shrinkage opposed by aggregate expansion will slowly become more noticeable as the temperature rises to 250°*C*. As mentioned, the upper limit for any useful strength retention in concrete is 500°*C* – 600°*C*. Operating temperatures of nuclear reactors range between 285°*C* – 650°*C*, depending on the reactor type. Exposure up to 90 minutes is required for the concrete to reach temperatures of 500°*C*, 30 *mm* from the surface. Therefore, it would be inconsequential to investigate the behaviour of concrete exposed to temperatures beyond this range (500°*C*– 600°*C*).

Aggregate type and hence aggregate properties are one of the most influential variables affecting concrete behaviour, especially concrete subjected to elevated temperatures. The surface texture and shape of the aggregate and the bond between hardened cement paste and aggregate (ITZ) seem to influence the performance of concrete exposed to elevated temperatures more than what the coefficient of thermal expansion of the aggregate does. The mineralogical and chemical properties of aggregates can also affect the bond. Some chemical bond may exist between the hardened cement paste and the aggregate in the case of limestone, dolomite and siliceous aggregates. It would therefore be meaningful to examine the effect of elevated temperature on concrete containing different aggregate types, especially aggregates readily available throughout South Africa.

Water/cement or water/binder ratios govern not only the strength of concrete but influence the elastic modulus of concrete as well. However, according to literature, the use of lower water/binder ratios may lead to reduced concrete performance when subjected to elevated temperatures.

The addition of admixture can result in a more environmentally friendly concrete mixture. The standard dosage for admixtures is in the range of 0.3% – 0.5% by weight of the cement. Research indicated that the addition of superplasticiser had little effect on the performance of concrete subjected to elevated temperatures and that there is a need for extensive research in this field.

SCMs, such as fly ash, are beneficial in improving the sustainability of concrete by minimising waste and reducing the environmental footprint. It is widely known that concrete mixtures containing SCMs such as fly ash have slower strength development compared to an equivalent pure Portland cement mixture. However, fly ash mixtures tend to show enhanced strength development beyond twenty-eight days after casting. The addition of SCMs does result in lower heat of hydration which is advantageous in mass concrete castings. Based on literature, fly ash substitution is normally limited to 50%. The addition of SCMs also enhance particle packing which produces concrete with a compact microstructure and low permeability. This can however lead to detrimental effects on the behaviour of concrete subjected to elevated temperatures. Nonetheless, limited research has been conducted on the effect of elevated temperatures on concrete containing SCMs.

According to literature, the use of recycled materials from construction and demolition as concrete aggregates, may present as a cost-effective and resource-saving alternative to natural aggregates. However, RCA should be evaluated and compared with recognised aggregate standards before use in concrete. Literature recommends that coarse RCA replacement of natural aggregates in structural concrete should be limited to 20% – 25%. However, there are standards that permit 100% coarse RCA replacement. Using 100% RCA replacement can effectively reduce demolition wastage and solve the shortage of natural aggregates. The replacement of natural aggregate with coarse RCA has gained wider acceptance than fine RCA. Literature recommends that fine aggregate replacement should be limited to less than 30%. Furthermore, the acceptable range for water absorption of coarse RCA in structural concrete is between 5% – 10%, with only Japan and China that specify water absorption percentages below 3%. Research studies show opposing results when considering RAC at elevated temperatures.

This can be attributed to the large number of variables involved with RAC as well as the different circumstances and testing conditions as mentioned earlier.

Based on the literature review conducted, it is necessary to perform an experimental study in order to ascertain the impact of elevated temperature on the properties of environmentally friendly concrete, i.e., concrete containing recycled aggregates, admixtures as well as blended cements.

3 EXPERIMENTAL SETUP

3.1 INTRODUCTION

This chapter provides a detailed discussion on the different materials used in the study as well as the various tests performed to determine the fresh and hardened properties of concrete. Material properties such as mineral composition, chemical composition, relative density, water absorption, particle size distribution, fineness modulus, flakiness index, surface texture and thermogravimetric analysis were all evaluated. Fresh concrete properties included slump and heat of hydration, while the hardened concrete properties considered were strength, modulus of elasticity, thermal expansion, drying shrinkage and potential durability. The temperature exposure procedure followed to determine the impact of elevated temperature of the properties of conventional and environmentally friendly concrete is discussed in detail in this chapter. All experiments were carried out in the Civil Engineering laboratory of the University of Pretoria.

3.2 MATERIALS

Concrete materials which are commonly used and readily available in South Africa were used in this study. A single batch of pure Portland cement was used together with one source of fly ash as a supplementary cementitious material (SCM). Natural aggregates from six different sources were investigated and were all chosen based on locality and type (see **Figure 2-6** and **Table 3-1**).

Table 3-1: Natural aggregates

Aggregate	Denotation	Source	Classification
Andesite	A	Eikenhof, Johannesburg	Basic igneous rock (volcanic)
Dolerite	R	Pietermaritzburg	Basic igneous rock (plutonic)
Dolomite	D	Olifantsfontein, East Rand	Sedimentary rock (carbonate)
Felsite	F	Zeekoewater, Witbank	Acidic igneous rock (volcanic)
Granite	G	Rheebok, Malmesbury	Acidic igneous rock (plutonic)
Tillite	T	Verulam, Durban	Sedimentary rock (glacial deposits)

Dolomite aggregate was considered as the reference natural aggregate throughout the study. Old concrete samples (approximately six months old) containing dolomite aggregate with a water/cement ratio of 0.5 and a water content of 230 l/m^3, was crushed (by means of a jaw crusher) and graded to acquire recycled concrete aggregates (RCA). Separation processes to only utilise coarse RCA would result in additional costs, making recycling financially less feasible. Therefore, to prevent the development of a new waste stream, all particles crushed to sizes less than 5 *mm* was utilised by partially replacing the fine aggregate with fine RCA. Aggregate properties were therefore experimentally determined to evaluate the suitability of RCA as a concrete material compared to natural aggregates and previous results from literature. The maximum aggregate sizes were 14 *mm* and 5 *mm* for coarse and the fine aggregate respectively.

3.2.1 Chemical composition

The chemical composition of materials used (based on X-Ray Fluorescence (XRF) results) can be seen in **Table 3-2**. As mentioned previously concrete containing siliceous aggregates do not perform well under elevated temperatures and the chemical composition results may give an indication of the performance of concrete containing different aggregate types after exposure to elevated temperatures. Felsite and granite aggregate had the highest silica contents.

The cement conforms with the loss on ignition (LOI) requirements ($\leq 5\%$ by mass) according to *SANS 50197-1 (2013)* for common cements. For most South African fly ash, loss of ignition (LOI) is a representative measure of the carbon content (Jacobs & Kiliswa, 2021). South African fly ash consist mainly of silica and alumina, therefore making it a siliceous fly ash according to *SANS 50197-1 (2013)*.

3.2.2 Mineral composition

The mineral composition of aggregates used (based on X-Ray Diffraction (XRD) results) can be seen in **Table 3-3**. Corresponding to literature, dolerite’s primary mineral is plagioclase. Furthermore, felsite is a volcanic rock of granitic composition. Both felsite and granite consist essentially of quartz. Tillite on the other hand is generally composed of a mixture of various pebbles and boulders consisting essentially of quartz, plagioclase and clays. According to literature, concrete containing siliceous aggregates, i.e., aggregates with high quartz or silica content do not perform well under elevated temperatures. The performance of concrete containing calcareous aggregates, i.e., aggregates with high dolomite content have also been questioned. According to literature, the mineralogical properties of aggregates can affect the ITZ. From the minerology, it can be expected that chemical bond may exist between the hardened cement paste and the aggregate in the case where higher percentage dolomite is present. Therefore, the mineral composition results may give an indication of the performance of concrete containing different aggregate types after exposure to elevated temperatures. It can be concluded that felsite and granite aggregates had the highest composition of quartz, while dolomite aggregate and RCA had the highest percentage dolomite.

Table 3-2: Chemical composition and LOI (%)

	SiO_2	***Al_2O_3***	***MgO***	***Na_2O***	***P_2O_5***	***Fe_2O_3***	***K_2O***	***CaO***	***TiO_2***	***MnO***	***ZrO_2***	***BaO***	**LOI**
Andesite	54.50	14.67	4.49	3.21	0.15	11.28	1.02	7.15	0.77	0.15	0.10	0.48	1.94
Dolerite	52.53	14.31	6.21	2.42	0.18	11.18	0.64	7.56	0.90	0.28	0.07	0.65	2.77
Dolomite	20.80	1.25	18.72	<0.01	0.01	0.44	0.19	21.26	0.03	0.37	0.01	0.31	36.46
Felsite	71.67	11.61	0.15	1.85	0.03	3.55	5.23	1.77	0.24	0.08	0.29	1.13	2.26
Granite	73.37	12.87	0.27	2.63	0.05	1.61	5.13	1.18	0.16	0.03	0.10	1.01	1.50
Tillite	65.11	13.47	2.98	2.74	0.20	5.68	2.95	1.99	0.57	0.11	0.14	0.83	3.06
RCA❖	21.54	2.36	12.78	0.08	0.04	0.92	0.27	28.45	0.11	0.29	0.04	0.42	32.28
CEM I	22.05	5.62	2.00	0.15	0.12	2.65	0.09	60.94	0.38	0.14	0.06	0.55	3.38
FA*	55.29	32.26	1.06	0.09	0.45	3.13	0.64	3.70	1.39	0.03	0.24	0.96	0.06

*❖Recycled concrete aggregate (RCA); * Unclassified fly ash (FA)*

Table 3-3: Mineral composition of aggregates (%)

	Andesite	Dolerite	Dolomite	Felsite	Granite	Tillite	RCA*
Quartz	9.4	12.0	11.3	44.5	42.7	30.2	16.0
Plagioclase	34.6	40.7	0	19.5	22.7	31.6	0.7
Dolomite	3.7	0	87.6	1.9	5.2	0.9	67.2
Calcite	0	0.8	0.6	1.7	0	1.0	12.8
Chlorite	13.0	14.5	0	0	5.9	13.0	0
Microcline	9.1	0	0	21.6	18.2	13.6	0
Kaolinite	3.4	3.0	0	1.4	1.6	3.3	0
Epidote	10.6	0	0	0	0	0	0
Muscovite	0	3.4	0	8.5	0	0	0
Actinolite	16.2	3.2	0	0	0	0.9	0
Biotite	0	0	0.5	0	3.8	4.1	1.2
Smectite	0	3.8	0	0	0	1.4	0
Portlandite	0	0	0	0	0	0	1.71
Gypsum	0	0	0	0	0	0	0.45
Augite	0	12.8	0	0	0	0	0
Enstatite	0	5.2	0	0	0	0	0
Ilmenite	0	0.6	0	0	0	0	0
Magnetite	0	0	0	1.1	0	0	0

**RCA = Recycled concrete aggregate*

3.2.3 Thermogravimetric analysis (TGA)

The dehydroxylation, decomposition and phase transition temperature regions for minerals are summarised in **Table 3-4**. When heated to 573$°C$, α-quartz transforms to β-quartz in siliceous aggregates. According to Prinsloo et al. (2018), a volume increase of about 2% – 4% can be expected with the α- to β-quartz inversion. Furthermore, Földvári (2011), stated that no mass change occurs during this endothermic reaction. According to Orosco et al. (2019), no mass change occurs during TGA of microcline up to 1000$°C$. The thermal decomposition of chlorite varies depending on the specific type of chlorite mineral. The dehydroxylation of chlorite typically take place in two steps. The first step between 470$°C$ – 650$°C$ and the second step between 700$°C$ – 800$°C$ (Földvári, 2011). The thermal decomposition of actinolite and epidote can also vary depending on the specific composition of the mineral, but it generally involves the release of water vapor and the formation of oxides. Various dehydration reactions are present in hydrated cement between $100°C - 200°C$. The dehydration, dehydroxylation and decomposition temperature regions for hydrated cement are summarised in **Table 3-5**.

Table 3-4: Dehydroxylation, decomposition and phase transition regions for minerals (Bloise, 2019; Földvári, 2011)

Mineral group	**Mineral name**	**Chemical formula**	**Reaction**	**Temperature range ($°C$)**
Carbonates	Dolomite	$CaMg(CO_3)_2$	Decomposition $CaMg(CO_3)_2 \rightarrow CaCO_3 + MgO + CO_2$ $CaCO_3 \rightarrow CaO + CO_2$	750 – 800 840 – 950
	Calcite	$CaCO_3$	Decomposition $CaCO_3 \rightarrow CaO + CO_2$	> 600
Oxides	Quartz	SiO_2	Phase transition α-quartz → β-quartz	573
Chlorite	Chlorite	$(Fe,Mg,Al)_6(Si,Al)_4O_{10}(OH)_8$	Dehydroxylation	470 – 650 700 – 800
Feldspar	Plagioclase	$(Na,Ca)(Si,Al)_4O_8$	-	-
	Microcline	$KAlSi_3O_8$	-	-
Amphibole	Actinolite	$Ca_2(Mg,Fe)_5Si_8O_{22}(OH)_2$	Dehydroxylation	650 – 800
Mica	Muscovite	$KAl_2[AlSi_3O_{10}(OH, F)_2]$	Dehydroxylation $KAl_2[AlSi_3O_{10}(OH, F)_2] \rightarrow KAlSi_3O_8 + Al_2O_3 + H_2O$	820 – 920
Nesosilicates	Epidote	$Ca_2(Fe+Al)Al_2(SiO_4)(Si_2O_7)O(OH)$	Dehydroxylation	900 – 1000

Table 3-5: Dehydration, dehydroxylation and decomposition regions for hydrated cement (du Toit, 2018)

	Cement phase	**Chemical formula**	**Temperature range (°C)**
Dehydration < 300°*C*	Calcium silicate hydrates	-	100 – 200
	Gypsum	$CaSO_4{\cdot}2H_2O$	100 – 120
	Ettringite	$3CaO{\cdot}Al_2O_3{\cdot}3CaSO_4{\cdot}32H_2O$	
	Hemihydrate	$CaSO_4{\cdot}0.5H_2O$	120 – 130
	Monocarboaluminate	$4CaO{\cdot}Al_2O_3{\cdot}CO_3{\cdot}11H_2O$	150 – 170
Dehydroxylation 400°*C* – 500°*C*	Portlandite	$Ca(OH)_2$	400 – 500
Decomposition > 600°*C*	Calcium Carbonate	$CaCO_3$	600 – 800

Thermogravimetric analyses were performed on a TA Instruments Q600 SDT Thermogravimetric Analyzer (TGA) & Differential Scanning Calorimeter (DSC). Milled aggregate samples of 20 *mg* – 25 *mg* were heated from approximately 25°*C* – 1000°*C* at a heating rate of 20°*C/min* in alumina pans under a dynamic atmosphere, controlled at a flow rate of 100 *mL/min* of nitrogen (N_2). Thermogravimetric analysis of the aggregate samples can give an indication of the mineral phases present and support the data from XRD (Castleman, 2023). However, according to Castleman (2023), quantifying exactly which mineral is undergoing dehydroxylation or decomposition in a temperature range is complicated when there is a combination of minerals present. The TGA of the different aggregate samples is illustrated in **Figure 3-1** and **Figure 3-2**. The first derivative of the original curve with respect to temperature is also indicated on the graphs.

If a sample is not dried properly, the initial loss of water (< 100°*C*) from the sample is associated with the loss of surface moisture. Two distinct regions of mass loss are evident from **Figure 3-1**. The first region where mass loss occurs is between approximately 450°*C* – 600°*C*. The dehydroxylation of chlorite mineral typically occurs between 470°*C* – 650°*C*. Andesite, dolerite and tillite contains chlorite (> 13%). The second region where mass loss occurs is between approximately 600°*C* – 750°*C*. Chlorite, calcite and actinolite minerals all undergo dehydroxylation or decomposition over a similar temperature range.

One distinct region of mass loss between 650°*C* – 820°*C* is evident from **Figure 3-2 a)**. Dolomite mineral typically decomposes between 750°*C* – 800°*C* and between 840°*C* – 950°*C*. When small samples are used, both peaks of thermal decomposition occur at almost identical temperatures (Földvári, 2011). The same peak is evident in **Figure 3-2 b)**. The RCA consist of

62.7% dolomite. Two additional regions of mass loss can be observed in **Figure 3-2 b)**. The first region where mass loss occurs is just below 100°*C*. Dehydration of hydrated cement occur over a similar temperature range. The second region where mass loss occurs is between 430°*C* – 480°*C*. Dehydroxylation of portlandite typically occur between 400°*C* – 500°*C*.

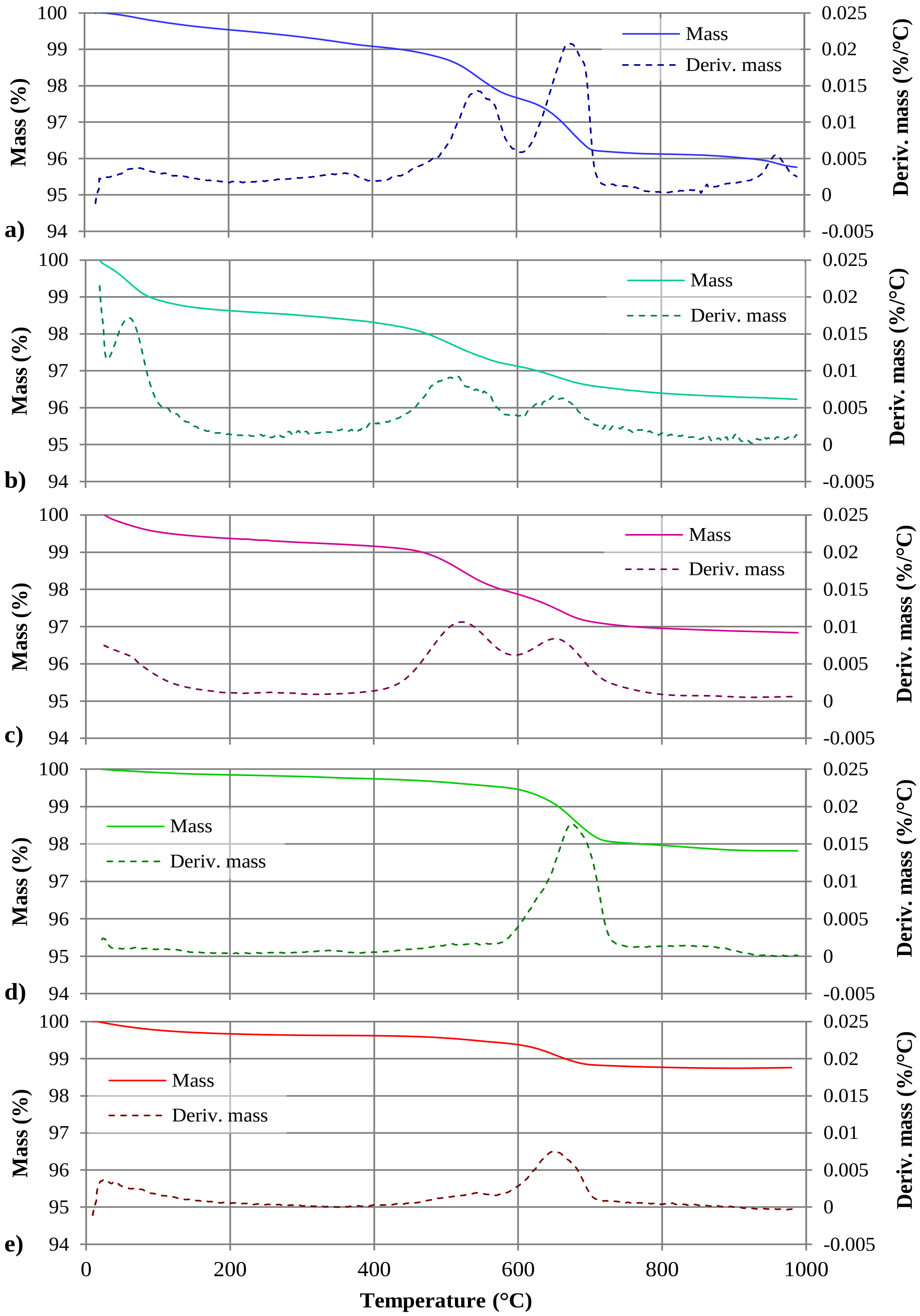

Figure 3-1: TGA of a) andesite, b) dolerite, c) tillite, d) felsite and e) granite

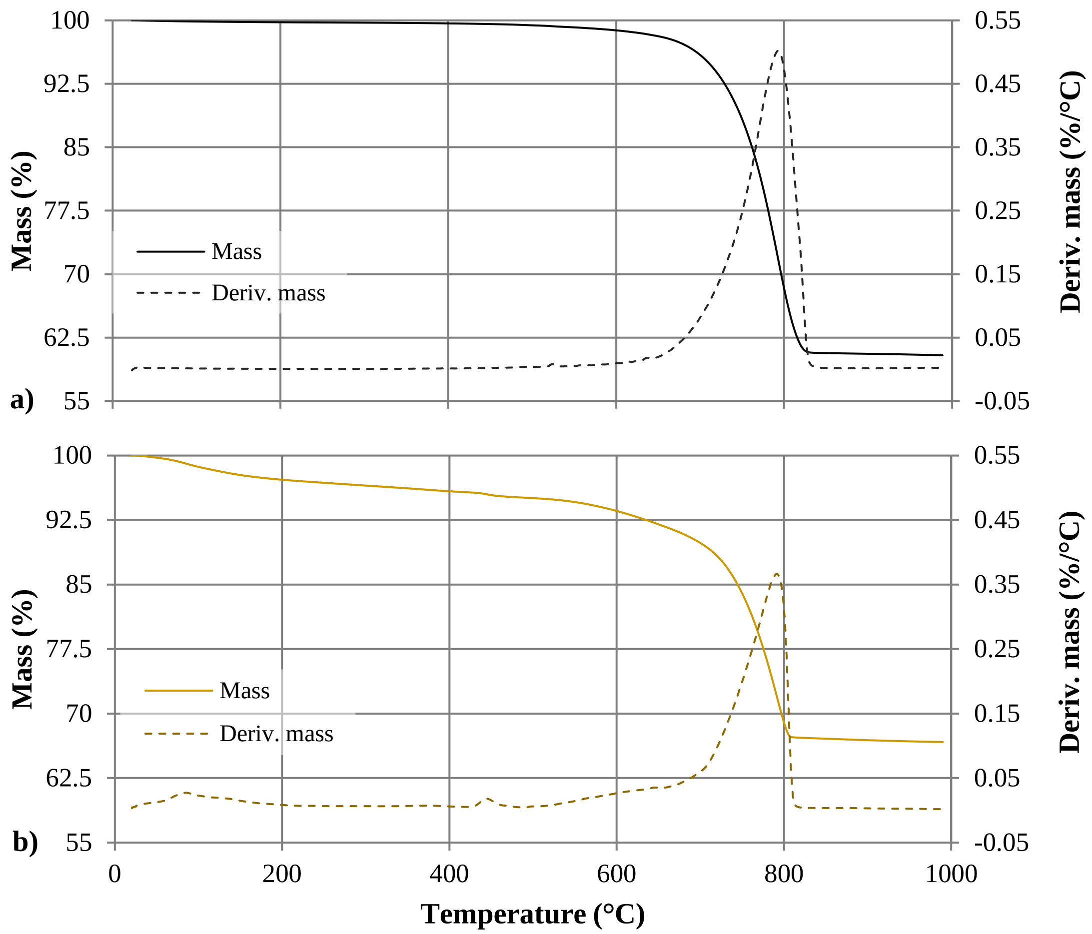

Figure 3-2: TGA of a) dolomite and b) RCA

The mass loss percentages obtained from the TGA curves at different temperatures is summarised in **Table 3-6**. The LOI values reported by the XRF analyses (**Table 3-2**) is also indicated in the table. The total mass loss of andesite and dolomite is slightly higher than the LOI value reported by the XRF analyses. The total mass loss of the other aggregates obtained from the TGA curves indicate similar values to the LOI values reported.

Table 3-6: Mass loss of different aggregates

Aggregate	Mass loss (%)				LOI
	200°C	350°C	500°C	1000°C	
Andesite	0.46	0.79	1.26	4.24	1.94
Dolerite	1.37	1.58	2.21	3.77	2.77
Dolomite	0.22	0.30	0.58	39.60	36.46
Felsite	0.15	0.23	0.35	2.18	2.26
Granite	0.33	0.37	0.44	1.24	1.50
Tillite	0.63	0.78	1.25	3.16	3.06
RCA	2.83	3.82	4.96	33.32	32.28

3.2.4 Relative densities

Relative densities (RD) of all materials were obtained by using a pycnometer and the results are tabulated in **Table 3-7**. The relative density results of the natural aggregate compared to previously published literature (AfriSAM TRG, 2021; Davis & Alexander, 1994) are illustrated in **Figure 3-3**. A fairly good correlation can be observed when comparing the experimental results to published literature. The relative density of RCA falls within the range of natural aggregate (2.64 – 2.96), as indicated in **Figure 3-4**. It is however lower compared to dolomite, and this can be attributed to the considerable amount of old cement paste in the RCA.

Table 3-7: Relative densities

Material	Source	RD	Standard deviation
Andesite	Eikenhof, Johannesburg	2.92	0.014
Dolerite	Pietermaritzburg	2.96	0.011
Dolomite	Olifantsfontein, East Rand	2.82	0.004
Felsite	Zeekoewater, Witbank	2.67	0.003
Granite	Rheebok, Malmesbury	2.64	0.009
Tillite	Verulam, Durban	2.70	0.009
RCA	-	2.66	0.023
Cement	AfriSAM, CEM I 52.5R	3.10	0.014
Fly ash	Unclassified, Lethabo	2.22	0.015
Superplasticiser	Chryso	1.06	0.009
Polypropylene fibres	-	0.90	0.010

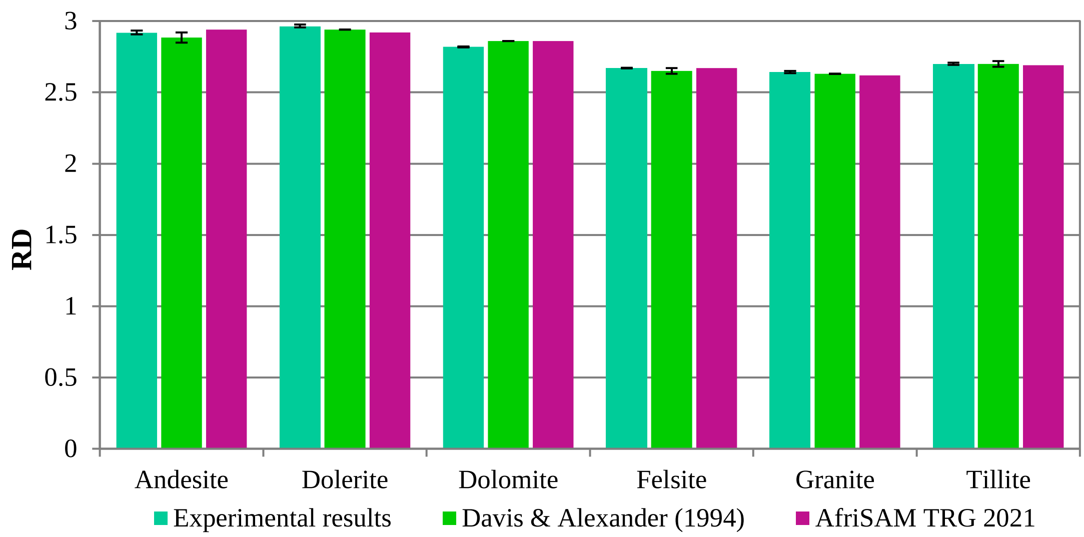

Figure 3-3: Relative density of natural aggregates

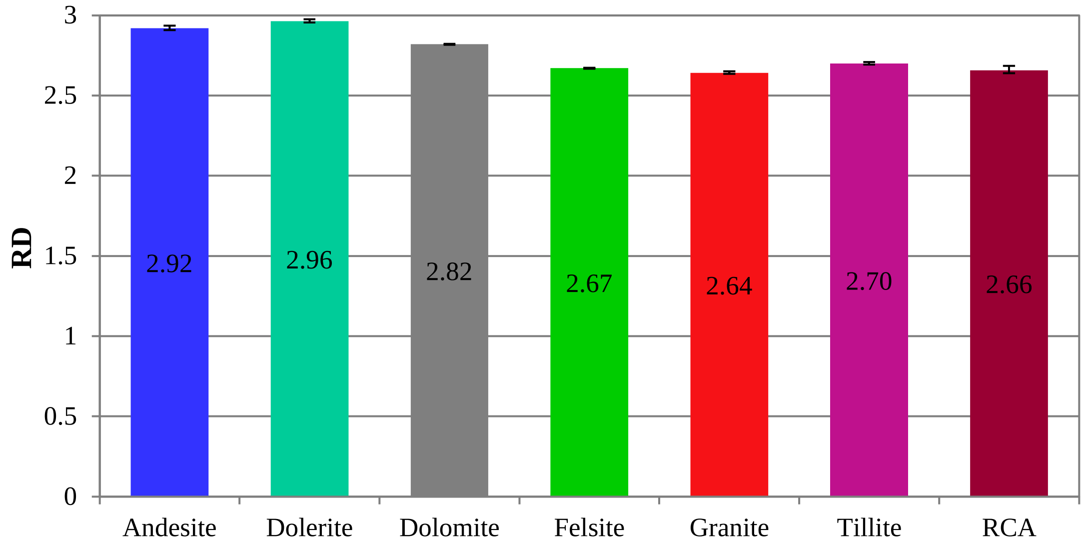

Figure 3-4: Relative density of RCA compared to natural aggregates

3.2.5 Water absorption

In South Africa, natural aggregates are generally non-porous with a water absorption of below 0.5% (Bonser & Alexander, 2021). Water absorption of the coarse aggregates was determined in accordance with *SANS 3001:AG20 (2014)*. The results can be seen in **Figure 3-5**. All the aggregates had a water absorption below 0.5% except for Tillite which yielded a water absorption of 0.57%. A fairly good correlation can be observed when comparing the experimental results to published literature (AfriSAM TRG, 2021; AfriSAM TRG, 2017; Davis & Alexander, 1994).

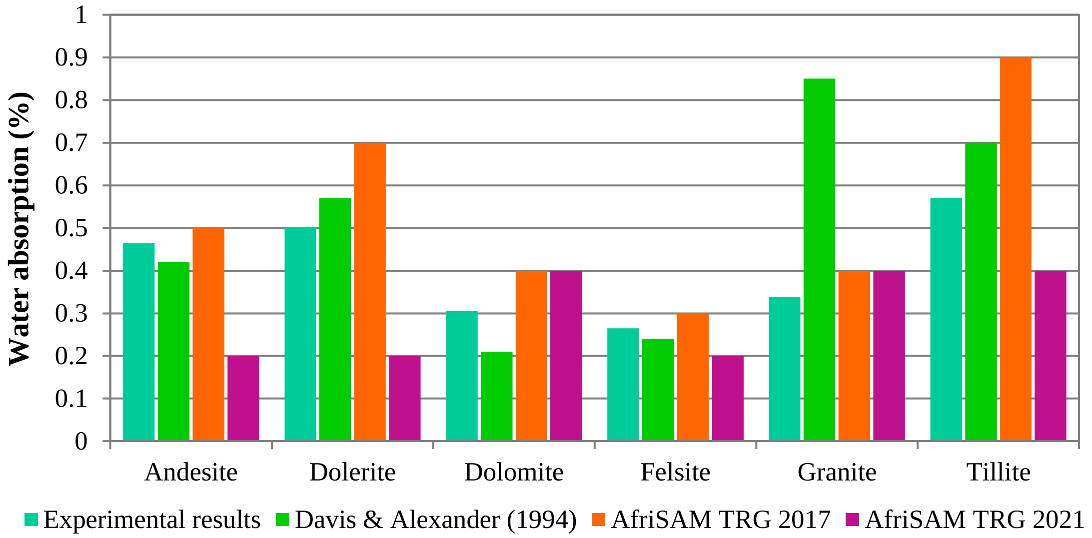

Figure 3-5: Water absorption of natural aggregates (%)

The water absorption of RCA is significantly higher than that of natural aggregates which will result in an increase in water demand for RAC (see **Figure 3-6**). Previous research suggested that coarse RCA should not be used in concrete when the water absorption is greater than 10*%*. The water absorption of the coarse RCA in this study was 4.1*%*, well below the recommended limit. Even though recycled concrete sands have higher water absorptions than coarse RCA, the water absorption of the fine RCA was assumed to be equivalent to the coarse RCA in this study.

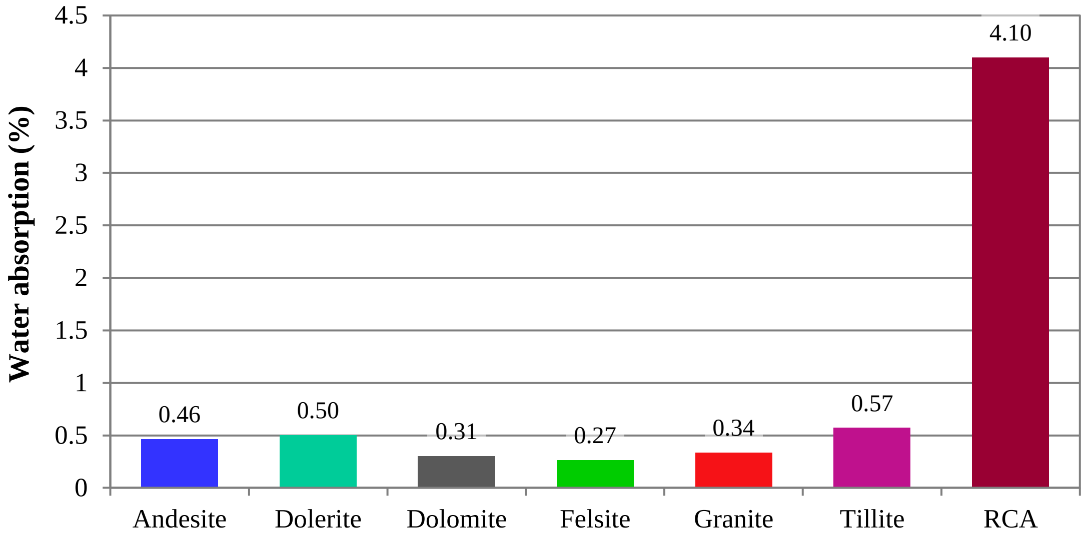

Figure 3-6: Water absorption (%) of RCA compared to natural aggregates

3.2.6 Particle size distribution

The particle size distributions were performed in accordance with *SANS 3001:AG1 (2014)* for materials greater than 75 μm, while a Malvern Instrument Mastersizer 2000 (Hydro 2000MU) apparatus was used to determine the particle size distribution of the fraction of material smaller than 75 μm. The test was repeated three times per material to ensure representative results. The particle size distributions are illustrated in **Figure 3-7** and **Figure 3-8** for the coarse (≤ 14 *mm*) and fine materials (≤ 5 *mm*) respectively. The grading for both the coarse and fine natural aggregate was as obtained from the relevant quarry. The particle size distribution for the coarse RCA is comparable to that of dolomite aggregate. The grading of all fine aggregates (sand) was fairly similar.

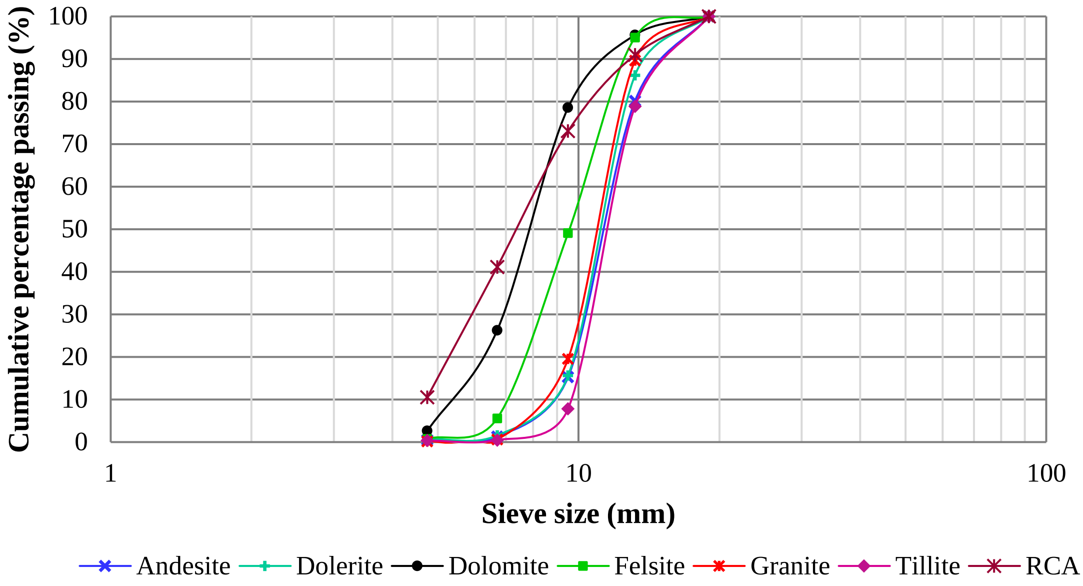

Figure 3-7: PSD for coarse material

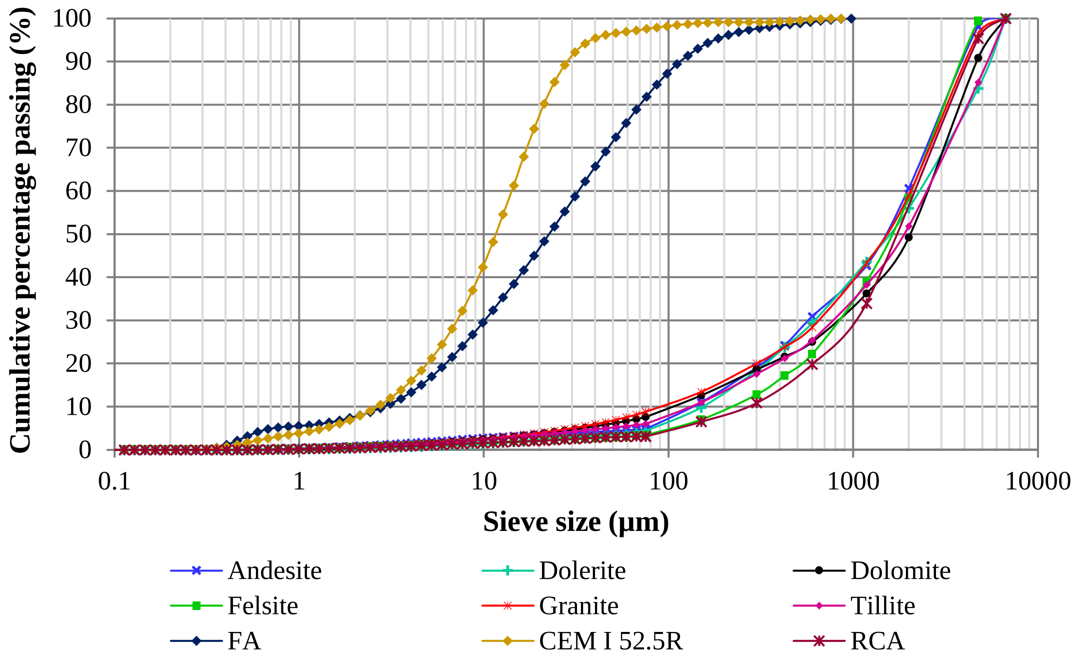

Figure 3-8: PSD for fine material

3.2.7 Fineness modulus

Fineness modulus (FM) is a dimensionless factor which categorises the average particle size of sand as coarse, medium, or fine, by adding the percentages of material retained on each of the standard sieves (starting at 150 μm and increasing in size by factors of two) and dividing the sum by 100 (*SANS 201:2008*). The FM is an indication of the water demand of the concrete mixture. FM categories are as follows (Alexander & Mindess, 2005):

i) Very fine sand: FM < 1.0
ii) Fine sand: 1.0 ≤ FM ≤ 1.9
iii) Medium sand: 2.0 ≤ FM ≤ 2.9
iv) Coarse sand: 3.0 ≤ FM ≤ 3.5
v) Very coarse sand: FM ≥ 3.5

Andesite and granite sand can both be classified as coarse sand, while all the other fine aggregates used in this study can be classified as very coarse sand (see **Figure 3-9**). The natural fine aggregates have fairly similar FM values falling in a range of 3.38 and 3.71, and their water demand should therefore be similar. The FM of the fine RCA is comparable to that of the natural aggregates.

The fineness of the fly ash was classified according to *SANS 50451:2 (2011)* which considers the material retained on the 45 μm sieve. Fly ash can be divided into two categories according to its fineness:

i) Category N: ≤ 40% by mass (variation less than ±10% from the stated value)
ii) Category S: ≤ 12% by mass

The fineness of the fly ash could therefore be classified as category N (≤ 40% by mass).

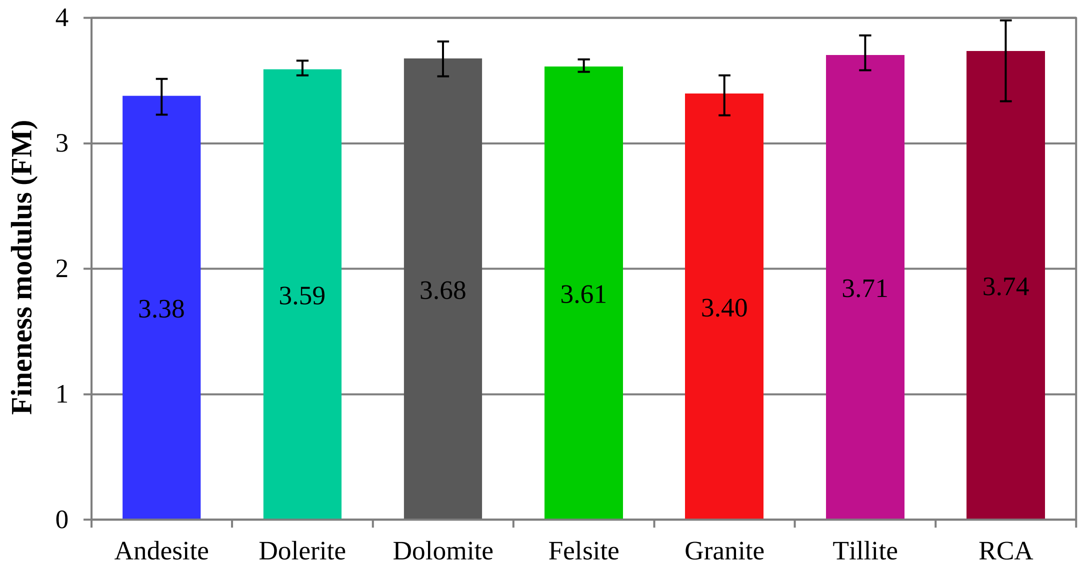

Figure 3-9: Fineness modulus (FM)

3.2.8 Flakiness Index

The flakiness index is a measure of the flatness of an aggregate and its interlocking properties. It refers to the fraction of flaky particles in a sample, by mass. As mentioned in literature, flaky aggregates tend to reduce the workability of fresh concrete and can, in some cases, reduce the strength of concrete due to both the compaction difficulties they create as well as their greater susceptibility to fracture under stress. The flakiness index of the coarse aggregates were determined in accordance with *SANS 3001:AG4 (2015)* and is illustrated in **Figure 3-10**. All aggregates, including the RCA, meet the requirements (≤ 35%) for use in concrete as stipulated in *SANS 1083 (2018)* with andesite yielding the highest flakiness of 28.5%.

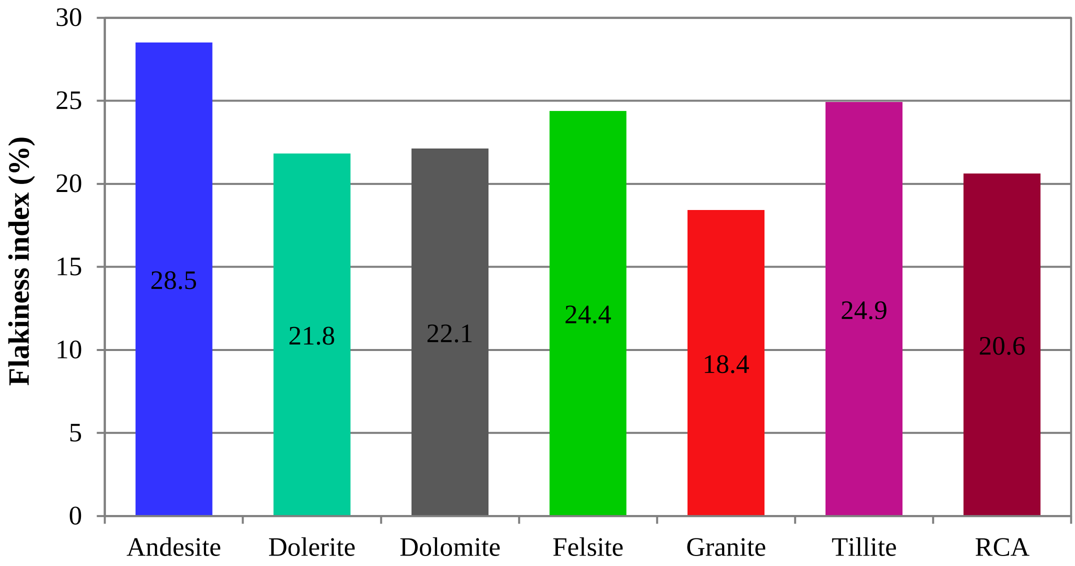

Figure 3-10: Flakiness Index

3.2.9 Surface texture

As mentioned in literature, it is difficult to define surface texture. The two independent geometric properties to describe surface texture namely surface roughness and the amount of actual surface area per unit of plane projected area was considered (Alexander & Mindess, 2005). A visual assessment of the surface texture in accordance with the standard descriptions as set out in *BS 812:102 (1989)* was performed, and the classification is given in **Table 3-8**. According to literature, dolomite's surface texture should be classified as smooth. However, according to the characteristics as described in *BS 812:102 (1989)*, the surface texture of the dolomite used in this study was classified as rough. A rough surface texture increases the specific area of the aggregate.

The specific surface of a granular material is defined by Crosswell & Brouard (2021) as the ratio of the total surface area of the material to its total absolute volume. Alternatively, it can be defined as the ratio of the total surface area of the material to its mass. Garboczi (2002) state that it is possible to mathematically model particle shape in 3-D and represent these shapes graphically. Coarse aggregate particles were modelled in 3-D by using a EinScan-Pro 3-D scanner and the models are illustrated in **Figure 3-11** and **Figure 3-12.** These scans were used to determine the surface area and volume of each aggregate. At least three coarse aggregate particles were modelled per aggregate type. The average specific surface area determined for each aggregate type is given in **Table 3-9.** The coefficient of variation for the specific surface area results were all below 0.25.

Table 3-8: Surface texture classification

Aggregate	Surface texture	Characteristics
Andesite	Rough	Rough fracture containing no easily visible crystalline constituents
Dolerite	Rough, Crystalline	Rough fracture containing easily visible crystalline constituents
Dolomite	Rough	Rough fracture containing no easily visible crystalline constituents
Felsite	Rough	Rough fracture containing no easily visible crystalline constituents
Granite	Granular, Crystalline	Fractur showing uniform size rounded grains containing easily visible crystalline constituents
Tillite	Rough	Rough fractures containing no easily visible crystalline constituents
RCA	Rough, Smooth, Honeycombed	Rough as well as smooth fracture containing visible pores and cavities.

Table 3-9: Specific surface area

Aggregate	Surface area	Volume	Mass	Specific surface	
	(mm^2)	(mm^3)	(g)	(mD^{-2}/mm^3)	(mm^2/g)
Andesite	1105.6	2116.8	6.18	0.522	178.9
Dolerite	899.8	1790.9	5.30	0.502	169.7
Dolomite	663.3	1133.4	3.20	0.590	209.3
Felsite	846.0	1735.4	4.63	0.487	182.6
Granite	738.9	1431.6	3.78	0.534	202.2
Tillite	849.0	1672.1	4.51	0.508	188.1
RCA	1154.3	2348.9	6.25	0.491	184.8

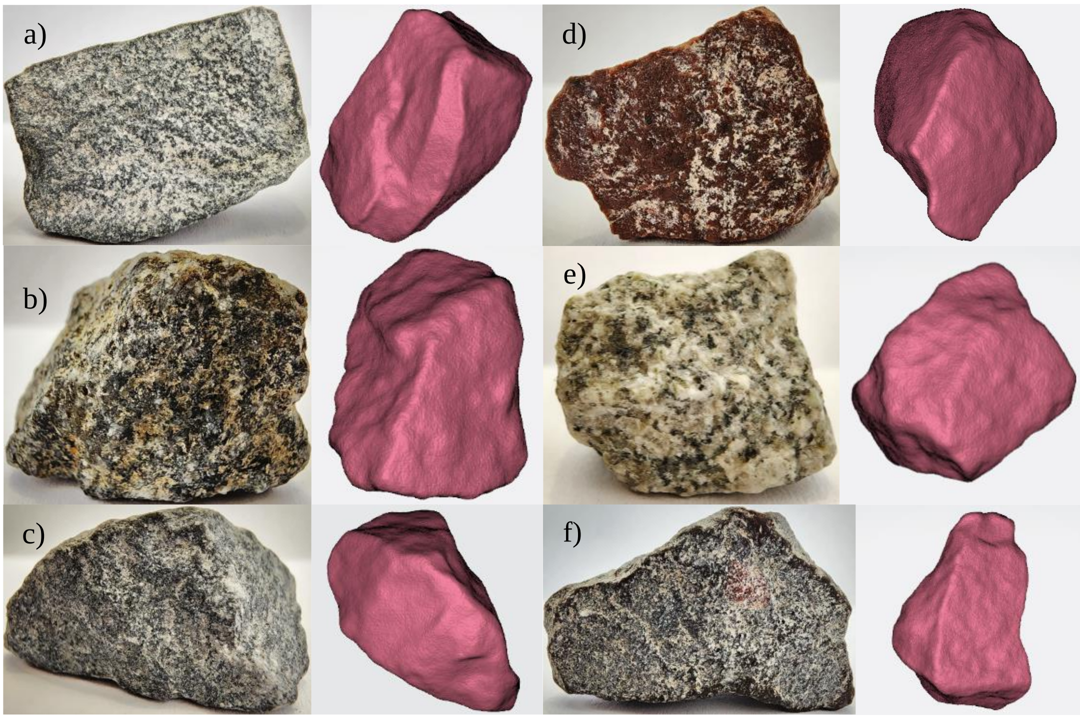

Figure 3-11: Shape and surface texture of natural aggregates - a) andesite, b) dolerite, c) dolomite, d) felsite, e) granite and f) tillite

Figure 3-12: Shape and surface texture of RCA

3.3 CEMENT CLASSIFICATION

Four different cement types were considered in this study, each containing a different percentage of fly ash replacing pure Portland cement by mass. To evaluate only the effect of fly ash as SCM, the single batch of cement was replaced with different percentages fly ash to create blended cements with:

i) 0% replacement – FA0
ii) 20% replacement – FA20
iii) 35% replacement – FA35
iv) 50% replacement – FA50

Mortar prisms were cast and tested according to *SANS 50196-1 (2006)* to classify the strength class of the different blended cements listed above in accordance with *SANS 50197-1 (2013)* (see **Table 3-10**).

Table 3-10: Cement classification and strength class according to *SANS 50197-1(2013)*

	Compressive strength (MPa)			
Class	**Early strength**		**Standard strength**	
	2 days	**7 days**	**28 days**	
32.5	-	≥ 16	≥ 32.5	≤ 52.5
32.5R	≥ 10	-		
42.5	≥ 10	-	≥ 42.5	≤ 62.5
42.5R	≥ 20	-		
52.5	≥ 20	-	≥ 52.5	-
52.5R	≥ 30	-		

The compressive strength development for mortar mixtures is illustrated in **Figure 3-13** and from the results the different blended cements were classified as follows:

- FA0 – CEM I 52.5R
- FA20 – CEM II/A-V 42.5R
- FA35 – CEM II/B-V 42.5R
- FA50 – CEM IV/B-V 32.5R

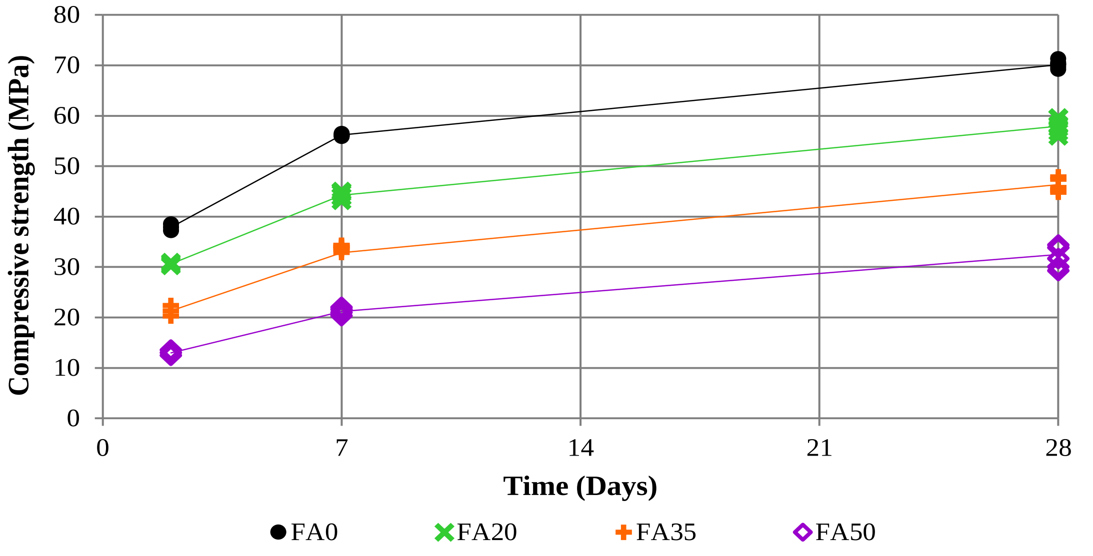

Figure 3-13: Compressive strength development for mortar mixtures

3.4 MIXTURE DESIGNS

Natural aggregates from six different South African sources were investigated in the study. Dolomite aggregate was used as the reference aggregate (coarse and fine) throughout the study. The principles set out in the BRE mix design methods for normal concrete mixes (Marsh, 1997) was used to establish the mix proportions in this study. Proportions of fine aggregate were established based on the percentage passing the 600 µm sieve resulting in a coarse to total aggregate ratio of 45% (unless specified otherwise). Only one source of pure Portland cement (CEM I 52.5R) was used. A constant water content (230 l/m^3) was used with a water/cement ratio of 0.5 (except for mixtures with varying w/c ratios). The effect of different water/cement ratios on the behaviour of concrete was investigated and the concrete mix designs are tabulated in **Table 3-11**. The water content was kept constant at 230 l/m^3. Therefore, the cement content was adjusted to obtain the desired water/cement ratio. The effect of different aggregate types on the behaviour of concrete was also investigated and the concrete mix designs are tabulated in **Table 3-12.** Additionally, the effect of varying coarse to total aggregate ratio was of interest and different fractions of 0%, 15%, 30%, 45%, 60% and 75% were investigated.

Table 3-11: Mix designs for concrete with different water/cement ratios (kg/m^3)

	D0.3	D0.4	D0.5	D0.6	D0.7
Water	230.0	230.0	230.0	230.0	230.0
Cement	766.7	575.0	460.0	383.3	328.6
Dolomite stone	663.3	741.8	788.8	820.2	842.6
Dolomite sand	810.7	906.6	964.1	1002.5	1029.9

Table 3-12: Mix designs for different aggregate types (kg/m^3)

Coarse to total aggregate ratio (%)	Water	Cement	Andesite		Dolerite		Dolomite		Felsite		Granite		Tillite	
			Stone	Sand	Stone	Sand	Stone	Sand	Stone	Sand	Stone	Sand	Stone	Sand
0	230.0	460.0	0	1815	0	1846	0	1753	0	1665	0	1646	0	1683
15	230.0	460.0	272.3	1543	276.0	1564	262.9	1490	249.0	1411	246.2	1395	251.8	1427
30	230.0	460.0	544.5	1271	552.0	1288	525.9	1227	497.9	1162	492.3	1149	503.5	1175
45	230.0	460.0	816.8	998.3	828.0	1012	788.8	964.1	746.9	912.8	738.5	902.6	755.3	923.1
60	230.0	460.0	1089	726.0	1104	736.0	1052	701.2	995.8	663.9	984.6	656.4	1007	671.3
75	230.0	460.0	1361	453.8	1380	460.0	1315	438.2	1245	414.9	1231	410.3	1259	419.6

The use of environmentally friendly concrete mix design methods such as adding superplasticiser to reduce the water and cement content, utilising blended cements or replacing natural aggregate with recycled concrete aggregates were investigated and the mix designs developed is tabulated in **Table 3-13**.

Table 3-13: Environmentally friendly concrete mixtures (*kg/m^3*)

	FA0	FA20	FA35	FA50	PP	SP	RCA	RCAF
Water	230.0	230.0	230.0	230.0	230.0	160.0	230.0	160.0
Superplasticiser	-	-	-	-	-	1.509	-	1.778
Cement	460.0	368.0	299.0	230.0	460.0	320.0	460.0	231.1
Fly ash	-	92.0	161.0	230.0	-	-	-	124.4
Dolomite stone	788.8	773.9	762.7	751.5	786.0	933.1	-	-
Dolomite sand	964.1	945.9	932.2	918.5	960.7	1140	727.5	828.2
Coarse RCA	-	-	-	-	-	-	744.1	847.1
Fine RCA	-	-	-	-	-	-	223.2	254.1
Polypropylene fibres	-	-	-	-	2.0	-	-	-

Blended cements, each containing a different percentage of fly ash replacing pure Portland cement by mass was considered. Replacement percentages of 20*%*, 35*%* and 50*%* were investigated. It is well known that the addition of polypropylene fibres in concrete mixtures can prevent explosive spalling as they rapidly melt at approximately 160*°C* and provide pressure relief channels. An additional mixture containing 13 *mm* polypropylene fibres was therefore cast and the dosage was limited to 2 *kg/m^3*, as recommended in literature (van Zijl & Boshoff, 2021; Huismann et al., 2012; *BS EN 1992-1-2*). For the mixture with denotation SP, commercially available polycarboxylate ether-based superplasticiser was used to reduce the water content with a corresponding reduction in cement content to maintain the water/cement ratio (0.5), while the sand and stone was adjusted to keep a coarse to total aggregate ratio of 45*%*. The superplasticiser dosage was kept constant at 0.5*%* by weight of the cement.

Recycled concrete aggregate was used to replace 100*%* and 25*%* of the coarse and fine dolomite aggregate respectively in the mixture with denotation RCA. These ratios were chosen so that 100*%* of the crushed concrete could be used as RCA. The coarse to total aggregate ratio was kept constant at 45*%*. Since the RCA had higher water absorption than the parent dolomite aggregate (as observed in **Figure 3-6**), the free water/cement ratio was kept constant at 0.5 by adding additional water to the mixture. The coarse aggregates were allowed to soak in the additional mixing water prior to casting. Approximately 40 l/m^3 of water was additionally added to the mixing water.

The behaviour of concrete containing a combination of superplasticiser with a reduced water and cement content, a blended cement as well as recycled aggregates was of interest. The mixture with denotation RCAF was therefore developed. The superplasticiser dosage was 0.5% by weight of the cement. A blended cement containing 35% fly ash was used together with recycled concrete aggregate replacing 100% and 25% of the coarse and fine dolomite aggregate respectively. The above-mentioned pre-soaking method was also used to keep the effective water/cement ratio constant at 0.5.

3.5 MIXING, CASTING AND CURING PROCEDURE

All the mixture constituents, including the mixing water, were stored in a temperature-controlled room (23 ± 2°*C*) for approximately twenty-four hours before casting. The volume of all mixtures was kept constant throughout the study. All the dry materials were mixed in a 100-litre pan mixer for thirty seconds whereafter water was added and the mixing continued for an additional ninety seconds. After mixing, a slump test was performed on the fresh concrete mixture in accordance with *SANS 5862:1 (2006)*. All the concrete specimen were cast in steel moulds, that were sufficiently lubricated, and were compacted on a vibrating table for approximately one minute. After casting the specimen were transferred to a temperature-controlled room (23 ± 2°*C*) and covered with a curing blanket. The specimen were left to cure for approximately twenty-four hours after casting whereafter they were demoulded and subsequently cured in water (23 ± 2°*C*) until the day of testing. Standard material properties were determined by testing specimen in saturated condition to conform with standard procedure (*SANS 5861:3, 2006*).

The various material properties determined, specimen sizes, number of specimens cast per test and testing ages are tabulated in Table 3-14. The specimen sizes conform with *SANS 5860 (2006)*, which state that the basic dimension (width and depth or diameter) of test specimens should be at least four times the nominal maximum size of the aggregate in the concrete. The number of specimen as stated in the table refer to the number of specimen tested per age for each mixture design. Some mixtures had additional strength development tests up to two hundred days. The aim of the study was to investigate mature concrete properties. Using a CEM I 52.5R cement, it was assumed that the change in concrete properties after more than twenty-eight days would be insignificant, and the results obtained for tests concluded twenty-eight days after casting was considered as mature properties. Since the concrete strength development of mixtures containing fly ash is slower than that of an equivalent pure Portland cement mixture, modulus of elasticity, splitting tensile strength and potential concrete durability for the mixtures containing fly ash were only tested after ninety-eight days of curing and not twenty-eight days as specified. This ensured equivalent strength at time of testing compared to the pure Portland cement mixture.

Table 3-14: Specimen cast

Material property	**Specimen size** (*mm*)	**Number of specimen**	**Testing age** (days)	**Temperature** (*°C*)
Compressive strength	100 × 100 × 100 (Cube)	3	7, 14, 28, 42 *(56, 98, 120, 200)	23 ± 2
Splitting tensile strength	100 × 100 (Cylinder)	4	28 *(98)	23 ± 2
Modulus of elasticity	100 × 200 (Cylinder)	2	28 *(98)	23 ± 2
Thermal expansion	100 × 200 (Cylinder)	1	90	-
Drying shrinkage	50 × 50 × 280 (Prism)	2	-	23 ± 2
Potential durability (OPI, Porosity)	± 30 × 70 (Disks cored from a 150 *mm* cube)	4	28 *(98)	23 ± 2

* *Mixtures containing fly ash*

3.6 CONCRETE TESTING PROCEDURES

3.6.1 Heat of hydration

The semi-adiabatic calorimeter test was performed on all mixtures to determine the maximum heat generated within the fresh concrete during initial hydration. The test setup was as described in *BS EN 196-9 (2010)* and the volume of the cylindrical container was approximately 0.8 *l*. The measurement of temperature was started within 10 – 15 minutes of mixing and the start temperatures were 22 ± 3°*C*. Temperature measurement in the calorimeter was continued for approximately four days after casting. The values obtained were for comparative testing and they indicate the maximum temperatures reached and the rate of heat development in freshly cast concrete.

3.6.2 Compressive strength

The compressive strength was determined in accordance with *SANS 5863 (2006)*. The specimen were weighed in air and in water to obtain the actual densities and air content before the compressive strength test was performed. The test consisted of crushing three cubes per test and calculating a mean value. Cube strength was determined at ages 7, 14, 28 and 42 days respectively. Some mixtures had additional strength tests up to two-hundred days. Standard material properties were determined by testing specimen in saturated condition to conform with the standard procedure.

According to Neville (2011), it is important to acknowledge that the definition of conventional concrete (normal strength concrete) has changed significantly over the time. In the past, 40 *MPa* concrete was regarded as high strength. Nowadays, concrete compressive strength less than approximately 60 *MPa* is normally considered as conventional concrete (normal strength concrete). Furthermore, most design codes do not consider strengths in excess of 60 *MPa* (Neville, 2011; Alexander & Mindess, 2005; Price, 2003). Soutsos & Domone (2017) state that the twenty-eight-day strength of concrete is usually used to characterise the concrete for design and specification purposes. Like with all materials, concrete exhibits an inherent variability of strength. The quality of concrete strength is therefore specified not at a minimum strength but as a characteristic concrete strength (f_{ck}). The difference between the characteristic strength and average compressive strength (target mean strength) is called the "margin" and is normally taken as 1.645 (5% defectives level) times the standard deviation (*SD*).

Equation 3-1 was used to determine the characteristic strength.

$$f_{ck} = f_{cm} - 1.645\ SD \qquad \textit{Equation 3-1}$$

Where f_{ck} is the characteristic concrete strength (*MPa*), f_{cm} is the target mean strength (*MPa*) and *SD* is the standard deviation. It is recommended by Soutsos & Domone (2017), that where less than 20 results are available, a standard deviation of 8 *MPa* should be used for concrete with characteristic strength in excess of 20 *MPa*.

3.6.3 Splitting tensile strength

The splitting tensile strength (indirect tensile strength) was determined in accordance with *SANS 6253 (2006)*. The test specimen consisted of four 100 *mm* × 100 *mm* cylinders. The splitting tensile strength was determined after twenty-eight days of curing for pure Portland cement mixes, while the mixtures containing fly ash were only tested after ninety-eight days.

Splitting tensile strengths were determined by testing specimen in saturated condition to conform with the standard procedure.

The splitting tensile strength was calculated by using Equation 3-2.

$$f_s = \frac{2P}{\pi L \emptyset} \qquad \textit{Equation 3-2}$$

Where f_s is the splitting tensile strength (*MPa*), *P* is the failure load (*N*), *L* is the specimen length (*mm*), and Ø is the specimen diameter (*mm*).

3.6.4 Elastic modulus of concrete

Elastic modulus of concrete was measured in accordance with *ASTM C469/C469M – 22*. The specimen consisted of 100 *mm* × 200 *mm* cylinders. All specimen were prepared by means of surface grinding prior to testing. The method involved loading a cylindrical specimen to 40% of the ultimate cylinder compressive strength and measuring the applied load and its corresponding deformation. Three loading cycles were performed on each specimen to minimise the hysteresis effect and stain measurements were taken over 120 *mm* (effective gauge length). According to the standard, to avoid end effects, the effective gauge length should not be more than two thirds the height of the specimen.

3.6.5 Thermal expansion

The test procedure to measure thermal expansion did not follow any standard test method and is summarised as follow:

i) A cylinder with a diameter of 100 *mm* and a length of 200 *mm* with a thermocouple imbedded in its centre was cured in water (23 ± 2°*C*) for ninety days.

ii) After curing, the cylinder was oven dried for seven days at 50 ± 2°*C*.

iii) The specimen was then allowed to gradually cool down to room temperature (23 ± 2°*C*)

iv) The length of the specimen was recorded as l_O.

v) Coiled nichrome wire, attached to a power supply, was wrapped around a stainless-steel pipe which served as the heating element. A thermocouple was placed on the surface of the pipe to measure temperature.

vi) The concrete cylinder was then placed in the stainless-steel pipe inside a thermally isolated box (see **Figure 3-14**).

vii) The displacement of the specimen was measured using two linear variable displacement transducers (LVDTs), placed on either side of the concrete cylinder.

viii) The temperature at the start of the test was 23 ± 2°C.

ix) Once the power supply was switched on, the stainless-steel pipe started heating up, which generated heat inside the box. This led to an increase in temperature in the concrete cylinder specimen.

x) The cylinder was heated until the centre of the specimen reached 60 ± 2°C, whereafter the test was ended and the specimen was allowed to cool down.

A deformation against temperature graph was plotted and a linear regression line was fitted through the data by only considering the linear portion of the graph. The slope of the linear regression line was taken as $\frac{\Delta l}{\Delta T}$ and the CTE was determined by using Equation 3-3.

$$\alpha = \frac{1}{l_O} \cdot \frac{\Delta l}{\Delta T} \qquad \textit{Equation 3-3}$$

where Δl is the change in length of an element (m) with original length l_O (m), as a result of a change in temperature ΔT (°C) and α is the coefficient of thermal expansion (/°C).

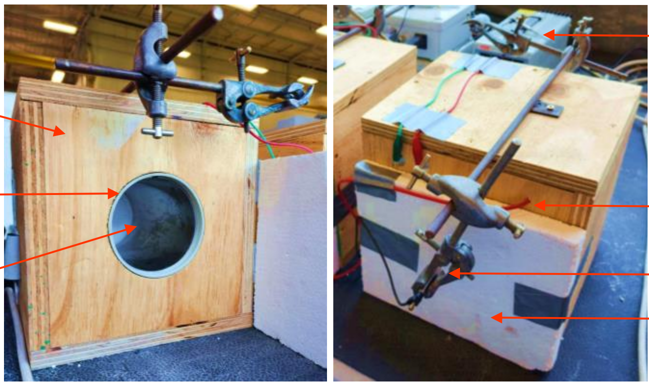

Figure 3-14: Thermal expansion test setup

3.6.6 Drying shrinkage

The test procedure to measure drying shrinkage in this study deviated from the standard procedure as set out in *SANS 6085 (2006)*. The prismatic specimen (50 mm × 50 mm × 280 mm) were removed from water after seven days of curing and the distance between the outer ends of the anvils were immediately measured by using a comparator. The specimen were then stored in a temperature- and humidity-controlled room (25 ± 2°C; 55 ± 5% RH) for the entire testing period. Drying shrinkage measurements were taken for a minimum of hundred and twenty days after casting. The results were expressed as strain (change in length in relation to the original gauge length). It is important to note that shrinkage is dependent on member geometry and dimensions.

3.6.7 Potential concrete durability

The disk specimen used for potential concrete durability testing were prepared in accordance with *SANS 3001:CO3:1 (2015)*. 150 *mm* concrete cubes were cast (one per mixture) and cured for twenty-eight days. Thereafter, a core was drilled from each of the cubes and four disk specimen where cut (30 ± 2 *mm* thick and 70 ± 2 *mm* in diameter). The disk specimen were then placed in a 50 ± 2°*C* oven to dry for seven days before testing. As mentioned, the fly ash mixtures were only tested after ninety-eight days of curing, which is not according to standard. The potential concrete durability tests performed in this study include oxygen permeability index (OPI) as well as porosity and both conformed to *SANS 3001:CO3:2 (2022)*. The OPI results were used to classify the concrete according to the recommendations by Alexander et al. (1999), as tabulated in **Table 3-15**. The porosity results were classified according to the suggestion by Moore et al. (2021), as tabulated in **Table 3-16**.

Table 3-15: Suggested durability classification for OPI values (Alexander et al., 1999)

Durability class	Oxygen Permeability Index (OPI)
Excellent	> 10
Good	9.5 – 10
Poor	9.0 – 9.5
Very poor	< 9.0

Table 3-16: Suggested durability classification for porosity values (Moore et al., 2021)

Durability class	Porosity (%)
Excellent	< 10
Good to poor	10 – 12
Poor to very poor	12 – 15
Very poor	-

3.7 TEMPERATURE EXPOSURE PROCEDURE

The residual steady state test method was used in this study and unstressed, or stressed testing conditions were not considered. One heating rate (0.5°*C/min*) as recommended by RILEM TC 129-MHT, one heating duration (12 hours) and four exposure temperatures (50°*C*, 200°*C*, 350°*C* and 500°*C*), were considered in this study. To easily compare results from different mixtures, the reported results have been expressed as a percentage relative to that of the companion specimen (23 ± 2°*C*) tested saturated after twenty-eight days curing (residual ratio). All the specimen, except the ones containing fly ash, were cured in water (23 ± 2°*C*) for twenty-eight days. As mentioned, the concrete strength development of mixtures containing fly ash is slower than that of an equivalent pure Portland cement mixture. Mixtures containing fly ash were only exposed to elevated temperatures after ninety-eight days of curing and not twenty-eight days as specified. This ensured equivalent strength at time of heating compared to the pure Portland cement mixture. In addition, the pure Portland cement mixture was also heated after seven days of water curing to determine the effect of early-age exposure to elevated temperature on the properties of concrete.

After curing, specimen were removed from water and immediately weighed. The samples were then oven dried at 50 ± 2°*C* for seven days to allow evaporation of water and prevent any concrete spalling. The temperature was then gradually elevated at a rate of 0.5*°C/min* until the desired temperature was reached. This equilibrium temperature was maintained for twelve hours to ensure steady state conditions. Afterwards, the specimen were left to gradually cool down in the oven. The specimen were then weighed again to compare the mass of the non-heat-treated specimen with its final mass after heating which is an indication of mass loss due to evaporation of water and CO_2. Autogenous recovery was investigated by means of two testing conditions namely, saturated and dry. Some specimen were placed back in water for seven days before testing as to allow saturated testing conditions. The other cube specimen were wrapped in plastic until the day of testing. This was to prevent any moisture absorption from the atmosphere. These specimen were tested in a dry condition, which is not according to standard procedures. Tests performed after temperature exposure include compressive strength, modulus of elasticity, splitting tensile strength and potential concrete durability. The various material properties determined, specimen sizes, number of specimen cast per test and testing ages are tabulated in **Table 3-17**. The number of specimen as stated in the table refer to the number of specimen tested per age for each mixture design and temperature.

Table 3-17: Specimen cast for heat testing

Material property	**Sample size** (*mm*)	**Number of specimen**	**Curing ages** (days)	**Temperature** (°*C*)
Compressive strength	100 × 100 × 100 (Cube)	6	28 *(98)	50, 200, 350, 500
Splitting tensile strength	100 × 100 (Cylinder)	4	28 *(98)	50, 200, 350, 500
Modulus of elasticity	100 × 200 (Cylinder)	2	28 *(98)	50, 200, 350, 500
Potential durability (OPI, Porosity)	± 30 × 70 (Disks cored from a 150 *mm* cube)	4	28 *(98)	50, 200, 350, 500

**Mixtures containing fly ash.*

3.7.1 Compressive strength

The compressive strength of specimen exposed to elevated temperatures was determined in accordance with *SANS 5863 (2006)* as set out in Chapter 3.6.2. Damaged infrastructure is normally tested by coring. The South African National Standard (*SANS 5865:1994*) specify that cores should be stored in water for forty-eight hours before testing. Therefore, after being exposed to elevated temperatures, half of the cube specimen were placed back in water for seven days before testing, as to allow saturated testing conditions. The other half of the cube specimen were wrapped in plastic until the day of testing. This was to prevent any moisture absorption from the atmosphere. These specimen were tested in a dry condition, which is not according to standard procedures.

3.7.2 Splitting tensile strength

The splitting tensile strength of specimen exposed to elevated temperature was determined in accordance with *SANS 6253 (2006)* as set out in Chapter 3.6.3. The splitting tensile strength test specimen were placed in water seven days before testing, as to allow saturated testing conditions (*SANS 5865:1994*).

3.7.3 Elastic modulus of concrete

The modulus of elasticity of specimen exposed to elevated temperatures were determined in accordance with *ASTM C469/C469M – 22* as set out in Chapter 3.6.4. After heating and gradually cooling, the specimen were placed back in water for seven days before testing. The specimen were tested in saturated conditions, which conforms to standard procedures (*SANS 5865:1994*).

3.7.4 Mass loss

The mass loss due to evaporation of water and CO_2 was determined by measuring the mass of specimen (100 *mm* cube and 150 *mm* cube specimen) prior to heating and comparing it to the final mass after heating.

3.7.5 Potential concrete durability

After heating, the 150 *mm* cubes specimen were wrapped in plastic and placed in a temperature-controlled ($23 \pm 2°C$) room for seven days and then only prepared for potential durability testing in accordance with *SANS 3001:CO3:1 (2015)* as mentioned in Chapter 3.6.7.

3.8 SUMMARY

Natural aggregates from six different sources as well as one source of RCA were investigated to prove the hypotheses that aggregate that contain elements and minerals that decompose at relatively low temperatures, would place less stress on the surrounding cement paste, thus reducing the damage caused to the ITZ by the thermal expansion of the aggregate.

Aggregate properties such as mineral composition, chemical composition, relative density, water absorption, surface texture, thermal expansion and thermogravimetric analysis were all considered to evaluate their effect on the residual strength and stiffness of concrete after exposure to elevated temperatures. Other factors such as paste content (water/cement ratio), coarse aggregate content and cement type were also investigated.

The residual steady state test method was used in this study with one heating rate (0.5°*C/min*), one heating duration (12 hours) and four exposure temperatures (50°*C*, 200°*C*, 350°*C* and 500°*C*). To easily compare results from different mixtures, the reported results have been expressed as a percentage relative to that of the companion specimen ($23 \pm 2°C$) tested saturated after twenty-eight days curing (residual ratio).

4 CONCRETE PROPERTIES

4.1 INTRODUCTION

This chapter provides an evaluation on the fresh and hardened material properties of conventional concrete compared to environmentally friendly concrete, i.e., concrete containing recycled aggregates, admixtures as well as blended cements. The fresh and hardened material properties considered include:

- slump,
- heat of hydration,
- compressive strength,
- splitting tensile strength,
- modulus of elasticity,
- thermal expansion coefficient,
- drying shrinkage and
- potential concrete durability.

4.2 SLUMP

In this section the slump of the different mixtures is discussed and compared. Slump was determined on the fresh concrete in accordance with *SANS 5862:1 (2006)*.

It is well known that lower water/cement ratios, hence lower water contents produce concrete with reduced slump. This correlates to the results obtained in this study as illustrated in **Figure 4-1**. The slump increased with an increase in the water/cement ratio up to 0.6, whereafter a reduction can be observed. This may be attributed to the low cement content, hence reduced paste content in the mixture with a water/cement ratio of 0.7. On the other hand, concrete mixtures with excessive paste content, such as the mixtures with a water/cement ratio of 0.3 and 0.4, was sticky and also lost workability.

The effect of coarse to total aggregate ratio together with aggregate type on the slump of concrete is shown in **Figure 4-2**. According to Crosswell & Brouard (2021), if the ratio of stone to sand is too high, concrete tend to be harsh and unworkable. This corresponds with the results obtained in this study. Mixtures with a coarse to total aggregate ratio of 75% produced substantially lower slump values than that of the 60% and 45% mixtures. On the other hand, over-sanded mixtures were sticky and less workable. The results clearly illustrate that regardless of the aggregate type, the best slump is achieved for coarse to total aggregate fractions in the range of 45% to 60%, which is within a normal mix proportion range.

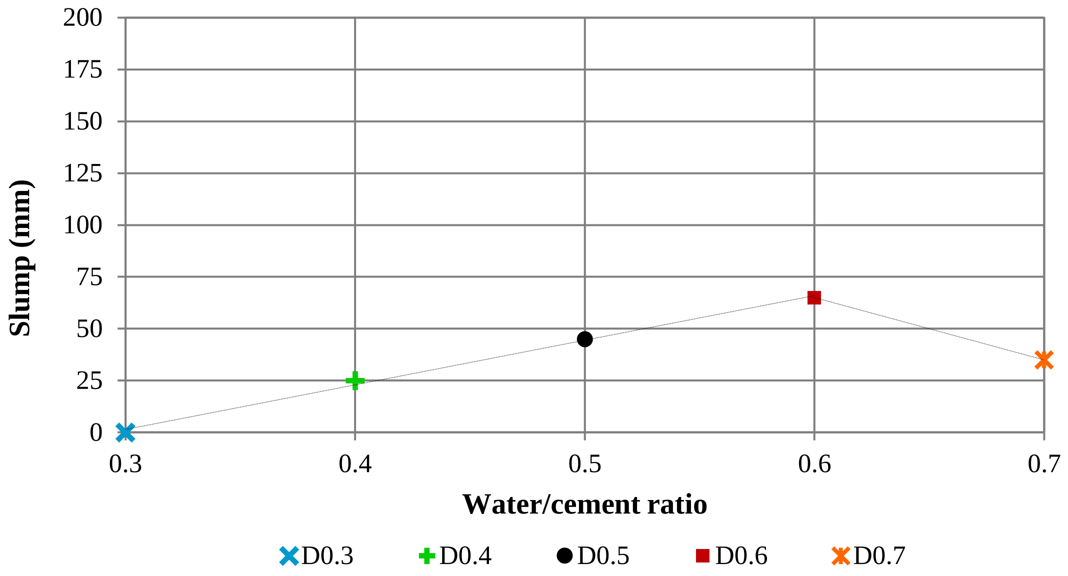

Figure 4-1: The effect of water/cement ratio on the slump of concrete

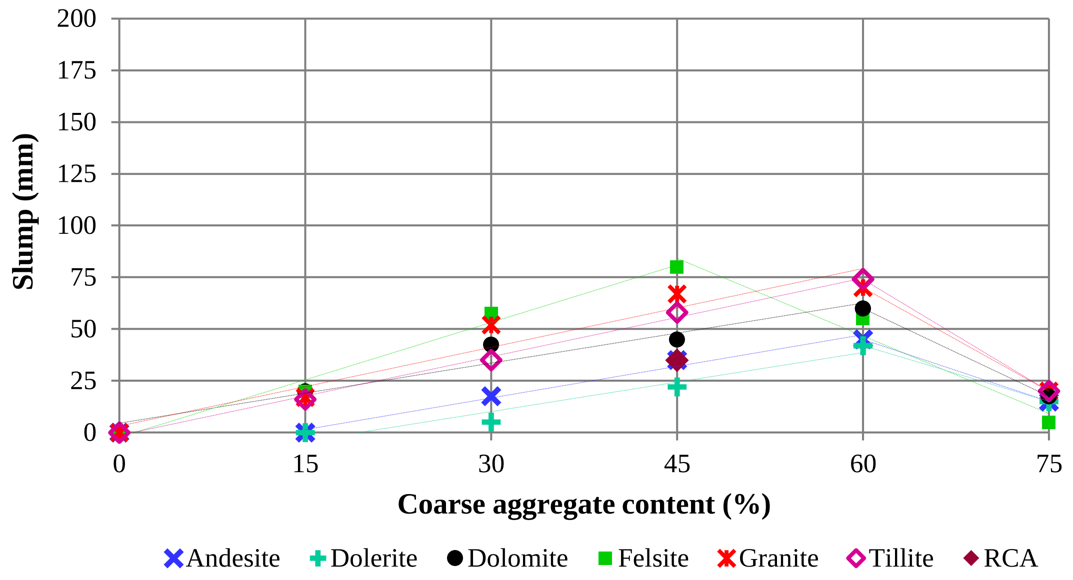

Figure 4-2: The effect of aggregate on concrete slump

The slump of the different environmentally friendly concrete mixtures is illustrated in **Figure 4-3.** It is well known that workability increases with the addition of fly ash due to the spherical shape of the fly ash particles. This corresponds with the results obtained in this study. The slump substantially increased as the fly ash content of the blended cement mixtures increased. Consequently, the water demand of the concrete can be reduced while still maintaining workability and strength which can then further contribute to reducing the environmental footprint.

The mixture containing superplasticiser with a reduced water and cement content produced a slump of 10 *mm*. The mix was however sufficiently workable once compacted on the vibrating table. The mixture with recycled concrete aggregates yielded a slump of 35 *mm*, relatively similar to that of the dolomite mixture (FA0). The difference can be attributed to either the rough surface texture and shape of the coarse RCA compared to natural aggregates or to the higher water absorption of the fine RCA. As mentioned, even though recycled concrete sands have higher water absorptions than coarse RCA, the water absorption of the fine RCA was assumed to be equivalent to the coarse RCA in this study. This can increase the water demand in fresh concrete even further. The mixture that combined RCA, fly ash as well as superplasticiser with a reduced water and cement content (RCAF) produced a slump of 10 *mm*, which was similar to the mixture containing pure Portland cement, superplasticiser and dolomite aggregate (SP).

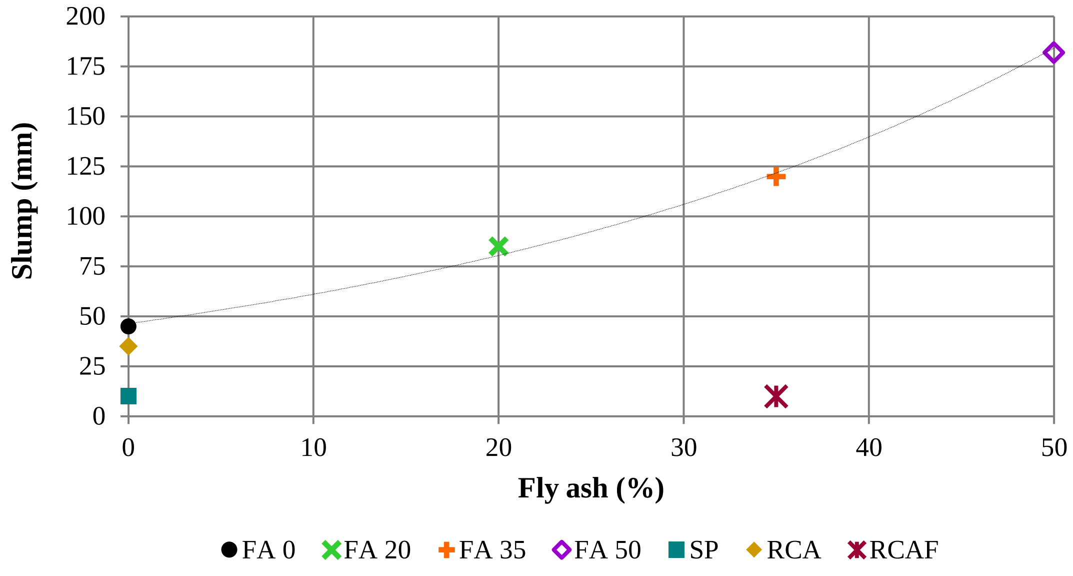

Figure 4-3: Slump of environmentally friendly concrete mixtures

As discussed, a change in slump was observed with different aggregate types. Workability can be influenced by various factors such as water absorption, shape and surface texture of the aggregate as well as the paste/aggregate volume ratio. The effect of water absorption can be ignored as all the natural aggregates had water absorption values below 1%. Despite published literature, no correlation between flakiness and slump could be obtained in this study. However, the slump improved with an increased paste (water and cement) to aggregate volume ratio for all mixtures. The effect of paste/aggregate volume ratio on the slump for the mixture with a 45% coarse to total aggregate ratio is shown in **Figure 4-4**. The other coarse to total aggregate fractions showed similar trends as indicated in **Figure 4-5**, especially when considering normal mix proportion ranges with coarse to total aggregate fractions of 45% to 60%.

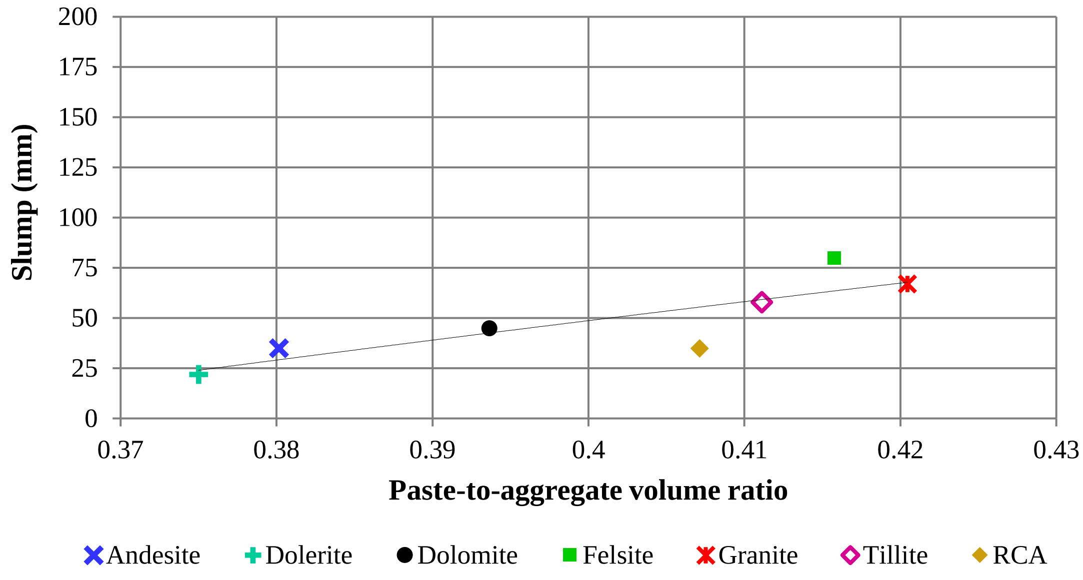

Figure 4-4: Effect of aggregate type and content on the slump of concrete

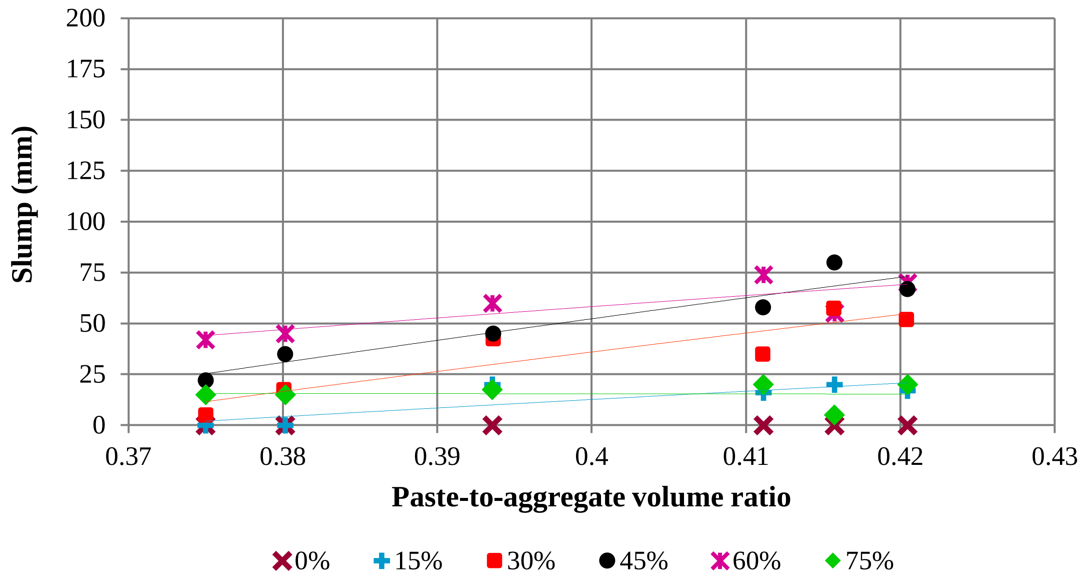

Figure 4-5: Effect of sand content on the slump of concrete

4.3 HEAT OF HYDRATION

The heat of hydration was measured by means of semi-adiabatic calorimeter testing. In this section the results of the maximum temperatures reached and the rate of heat development in freshly cast concrete are compared. It is important to note that concrete was cast over an extended period of time and the ambient temperatures at time of casting influence the temperature of the mixer itself which can affect the initial temperature of the fresh concrete.

The maximum temperature that developed as well as the rate of development for mixtures with changing water/cement ratios is shown in **Figure 4-6**. Increased water/cement ratios, hence mixtures with a reduced cement content, resulted in decreased peak temperatures and rate of development. Normal practice limits the maximum temperatures reached in mass concrete to 70°*C*. The highest peak temperature reached was almost 60°*C* for the mixture with a water/cement ratio of 0.3. However, heat exchange with the environment is not entirely prevented in the semi-adiabatic test. Therefore, maximum temperature development in mass concrete might be higher than what was measured in this study.

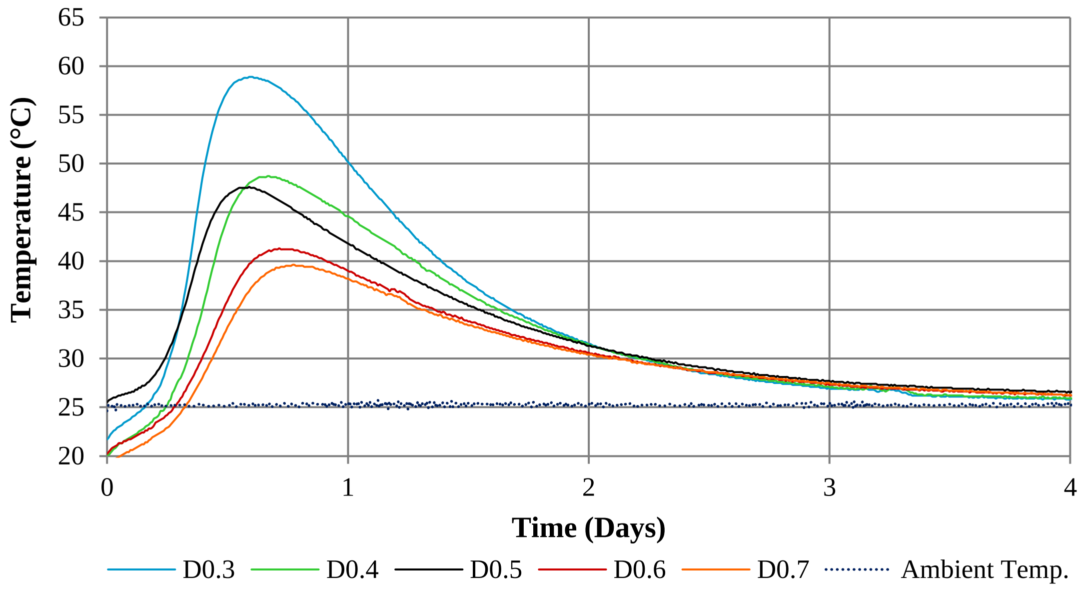

Figure 4-6: Heat of hydration of concrete with varying water/cement ratios

The addition of both fine and coarse aggregates profoundly influenced the heat of hydration of concrete, as illustrated in **Figure 4-7**. The heat of hydration of the paste alone produced a peak temperature of almost 85°*C*. This was drastically reduced to approximately 49°*C* with the addition of both coarse and fine aggregates. Furthermore, the peak temperature was reduced for mixtures containing a higher percentage coarse aggregate. The mixture with 75*%* coarse to total aggregate ratio (D75) had the lowest peak temperature of approximately 44°*C*, while the mortar mixture (D0) had the highest peak temperature of approximately 49°C.

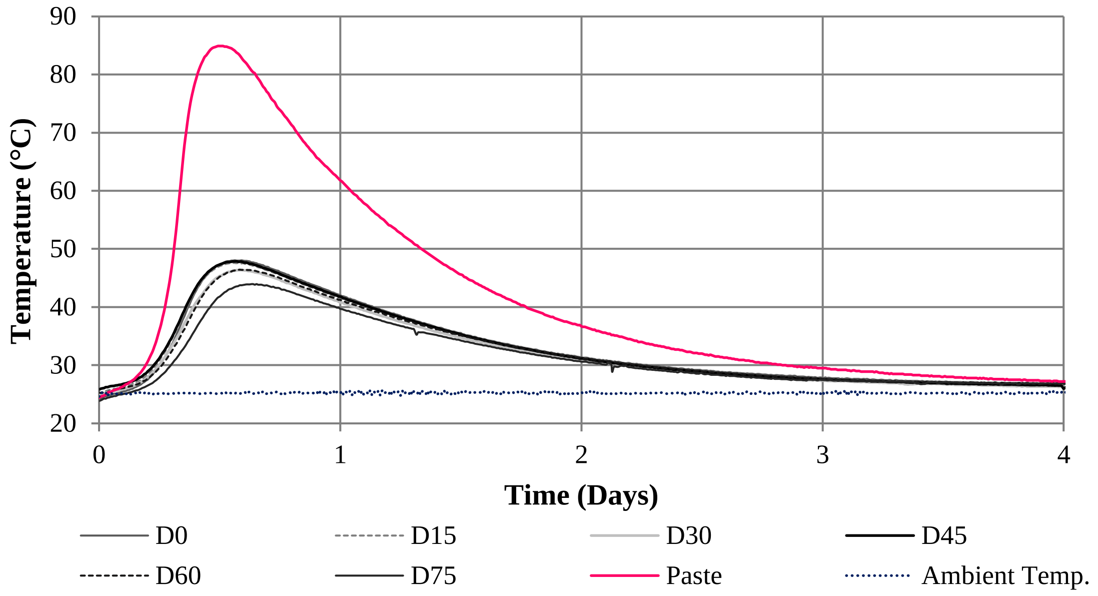

Figure 4-7: Heat of hydration of concrete with varying coarse to total aggregate ratios

The effect of aggregate type on the heat of hydration and rate of heat development of concrete is shown in **Figure 4-8** and **Figure 4-9**. The results indicate that aggregate type has a limited effect on the maximum temperature reached. However, the results highlight the importance of proper aggregate selection in mass concrete casts when considering rate of heat development. Concrete containing andesite aggregate reached its maximum temperature after 12.5 hours, while concrete containing granite aggregate only reached its peak temperature after approximately 15.75 hours.

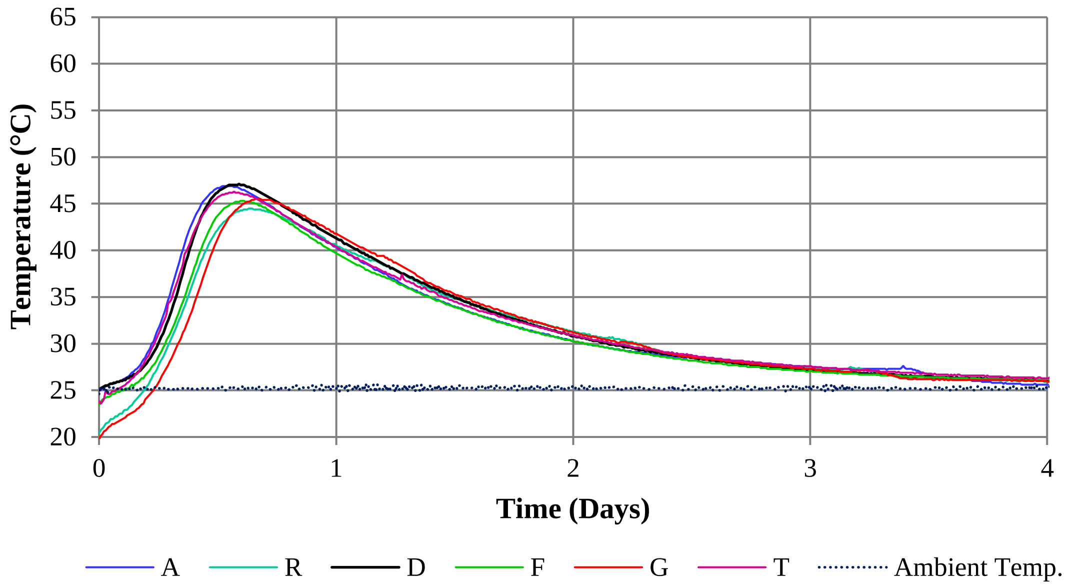

Figure 4-8: Heat of hydration of concrete containing different aggregate types

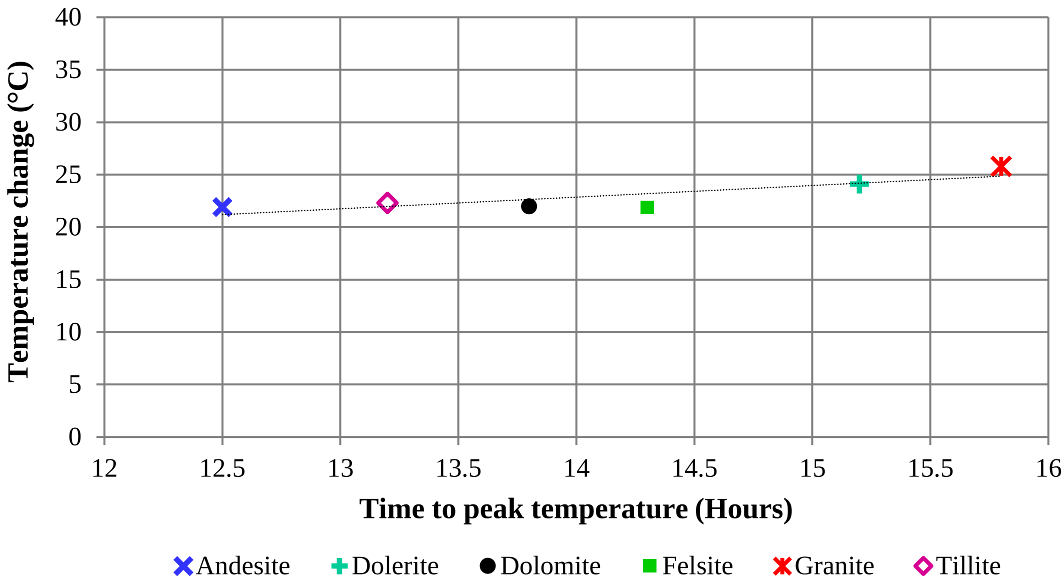

Figure 4-9: Rate of heat development for concrete containing different aggregate types

The heat of hydration of freshly mixed environmentally friendly concrete is shown in **Figure 4-10**. The addition of fly ash substantially reduced the peak temperatures reached, which is of particular importance for mass concrete casts. The water/binder ratio was kept constant for all the blended cement mixtures and the cement content was therefore reduced with an increase in fly ash replacement.

The maximum temperature reached for the mixture containing superplasticiser (SP) was substantially reduced compared to that of the dolomite mixture (FA0). The heat of hydration can therefore be reduced in mass concrete casts with the addition of superplasticiser by reducing the water content and hence the cement content of the mixture. The rate of temperature development is also decreased with the addition of superplasticiser. The results obtained reinforces the conclusion made previously that the peak temperatures of concrete mixtures increase with an increase in cement content.

The peak temperature for the RAC mixture was reduced compared to the dolomite mixture (FA0) with very little difference in the rate of development. The mixture that combined RCA, fly ash as well as superplasticiser (RCAF) had a substantially reduced peak temperature compared to the other mixtures.

The effect of cement content on the change between initial and peak temperature is shown in **Figure 4-11**. The trend illustrates that the change in temperature decrease as the cement content decrease. The results highlight the importance of reducing the cement content of mixture by either using blended cements or adding superplasticisers to reduce the maximum temperatures reached in mass concrete casts.

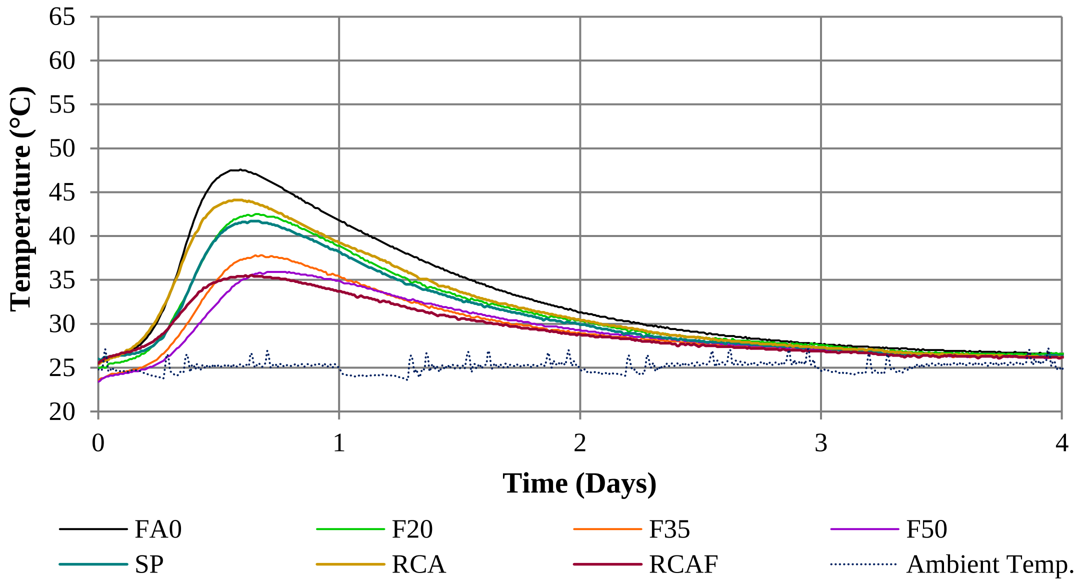

Figure 4-10: Heat of hydration of environmentally friendly concrete

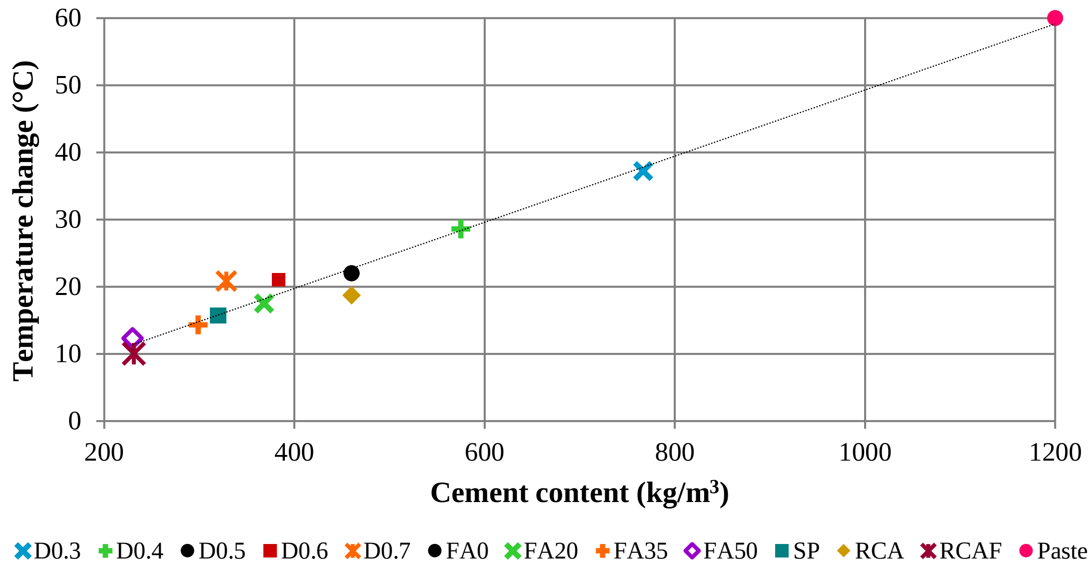

Figure 4-11: Effect of cement content on the heat development of concrete

4.4 COMPRESSIVE STRENGTH DEVELOPMENT

It is well known that lower water/cement ratios produce concrete with higher strengths. This correlates with results obtained in this study as shown in **Table 4-1** and **Figure 4-12**. The compressive strength for the mixture with a water/cement ratio of 0.7 was greater than 30 *MPa*, even after only seven-days of water curing, indicating that structural concrete can be manufactured with water/cement ratios ranging from 0.3 – 0.7.

Table 4-1: Average compressive strength of concrete with varying w/c ratios

	Time (Days)	7	14	28	42
D0.3	Compressive strength (*MPa*)	70.3	77.3	80.1	82.2
	Standard deviation (*MPa*)	5.09	0.31	4.42	0.32
	Coefficient of variation	0.07	0.00	0.06	0.00
D0.4	Compressive strength (*MPa*)	56.2	70.1	73.1	78.6
	Standard deviation (*MPa*)	4.12	1.03	5.42	0.70
	Coefficient of variation	0.07	0.01	0.07	0.01
D0.5	Compressive strength (*MPa*)	47.8	54.1	60.1	64.0
	Standard deviation (*MPa*)	3.10	0.52	1.97	1.56
	Coefficient of variation	0.06	0.01	0.03	0.02
D0.6	Compressive strength (*MPa*)	40.7	44.7	47.4	51.5
	Standard deviation (*MPa*)	2.11	0.82	5.39	0.29
	Coefficient of variation	0.05	0.02	0.11	0.01
D0.7	Compressive strength (*MPa*)	31.8	35.4	38.8	40.6
	Standard deviation (*MPa*)	1.90	0.95	0.25	0.64
	Coefficient of variation	0.06	0.03	0.01	0.02

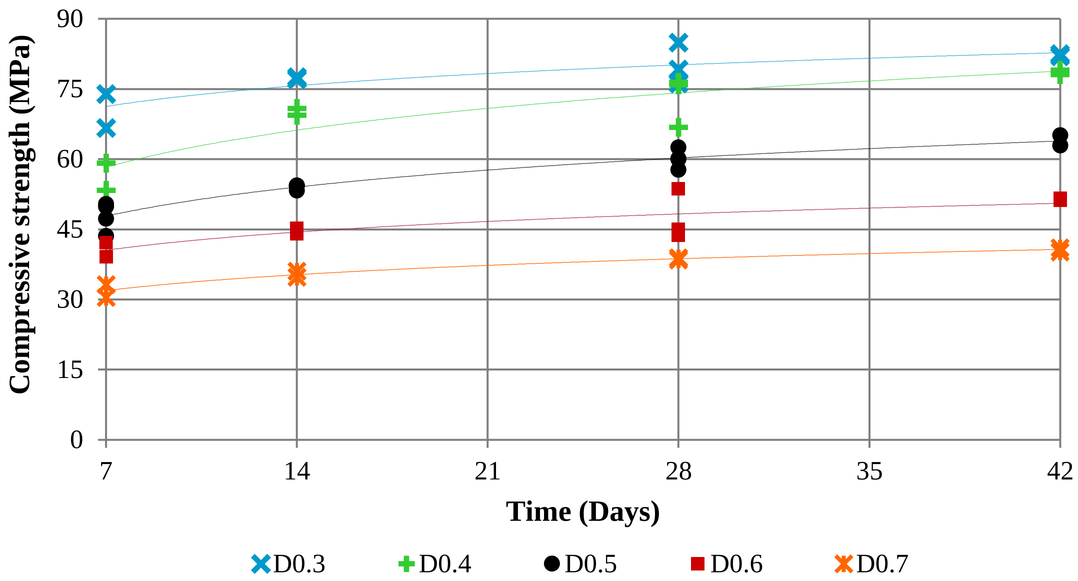

Figure 4-12: Compressive strength development of concrete with varying w/c ratios

The effect of coarse to total aggregate ratio on the strength development of concrete is indicated in **Table 4-2** and **Figure 4-14**. The graphs indicate similar strength development, as well as twenty-eight-day strengths, regardless of the coarse to total aggregate ratio. The twenty-eight-day compressive strength results had a variation of 10.3 *MPa*.

Table 4-2: Average compressive strength of concrete with varying sand contents

	Time (Days)	7	14	28	42
D0	Compressive strength (*MPa*)	49.9	56.3	63.1	65.1
	Standard deviation (*MPa*)	2.66	3.01	2.07	0.00
	Coefficient of variation	0.05	0.05	0.03	0.00
D15	Compressive strength (*MPa*)	50.7	56.9	61.6	65.4
	Standard deviation (*MPa*)	1.48	0.92	4.65	1.00
	Coefficient of variation	0.03	0.02	0.08	0.02
D30	Compressive strength (*MPa*)	49.8	54.8	60.5	63.5
	Standard deviation (*MPa*)	0.65	0.99	3.43	1.76
	Coefficient of variation	0.01	0.02	0.06	0.03
D45	Compressive strength (*MPa*)	47.8	54.1	60.1	64.0
	Standard deviation (*MPa*)	3.10	0.52	1.97	1.56
	Coefficient of variation	0.06	0.01	0.03	0.02
D60	Compressive strength (*MPa*)	48.8	54.0	60.9	63.5
	Standard deviation (*MPa*)	0.55	1.09	1.69	0.57
	Coefficient of variation	0.01	0.02	0.03	0.01
D75	Compressive strength (*MPa*)	47.0	53.6	59.2	62.9
	Standard deviation (*MPa*)	1.66	1.10	0.85	2.62
	Coefficient of variation	0.04	0.02	0.01	0.04

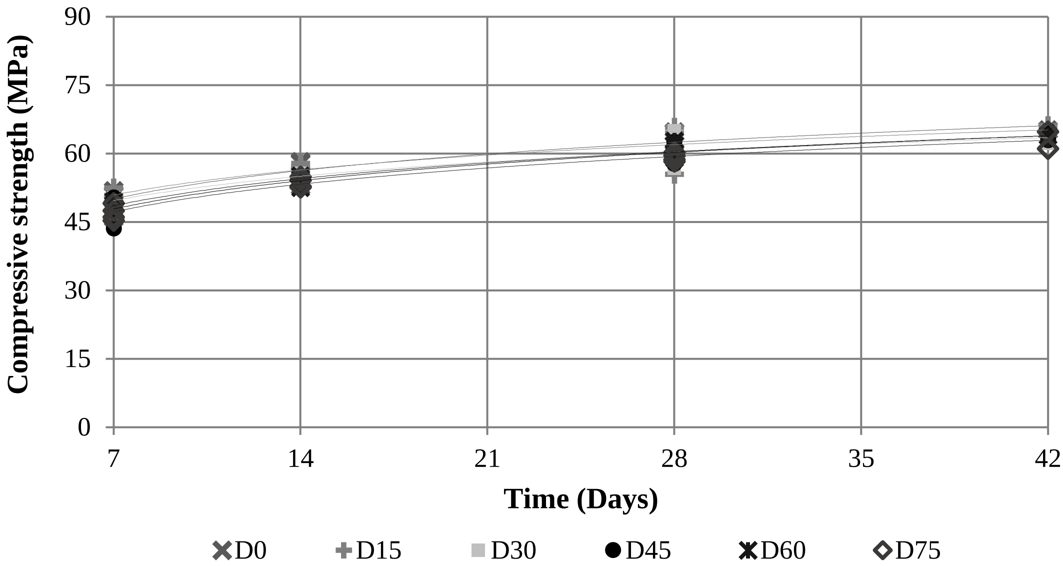

Figure 4-13: Effect of sand content on concrete strength development

The effect of aggregate type on the strength development of concrete is shown in **Table 4-3** and **Figure 4-14**. The graphs indicate similar strength development, as well as twenty-eight-day strengths, regardless of the aggregate type. According to literature, the type of aggregate used in concrete can cause a variation in compressive strength of 10.0 *MPa* – 15.0 *MPa*. The twenty-eight-day compressive strength results obtained in this study had a variation of 12.0 *MPa*. Concrete strength can be influenced by various factors such as surface texture, shape, grading and stiffness of the aggregate.

Despite published literature, no correlation between the compressive strength and the surface texture, grading or stiffness of the aggregates could be obtained in this study. It can be concluded that aggregate type and coarse to total aggregate ratio has a negligible effect on the strength of concrete for mixtures with equal water/cement ratio and water content. Water/cement ratio had a much greater effect on the compressive strength. It is important to note that aggregate type will have a greater effect on the compressive strength when considering higher strength concrete (Alexander & Mindess, 2005).

Table 4-3: Average compressive strength of concrete with different aggregate types

	Time (Days)	**7**	**14**	**28**	**42**
Andesite	Compressive strength (*MPa*)	51.6	54.8	60.0	63.9
	Standard deviation (*MPa*)	0.56	2.12	2.78	3.53
	Coefficient of variation	0.01	0.04	0.05	0.06
Dolerite	Compressive strength (*MPa*)	50.9	58.5	63.2	65.5
	Standard deviation (*MPa*)	1.26	0.01	5.98	2.65
	Coefficient of variation	0.02	0.00	0.09	0.04
Dolomite	Compressive strength (*MPa*)	47.8	54.1	60.1	64.0
	Standard deviation (*MPa*)	3.10	0.52	1.97	1.56
	Coefficient of variation	0.06	0.01	0.03	0.02
Felsite	Compressive strength (*MPa*)	48.2	57.1	59.2	63.2
	Standard deviation (*MPa*)	1.19	1.16	1.22	1.07
	Coefficient of variation	0.02	0.02	0.02	0.02
Granite	Compressive strength (*MPa*)	52.1	55.4	61.9	67.1
	Standard deviation (*MPa*)	0.19	3.31	2.68	3.37
	Coefficient of variation	0.00	0.06	0.04	0.05
Tillite	Compressive strength (*MPa*)	47.4	56.7	58.3	67.6
	Standard deviation (*MPa*)	3.60	2.38	0.47	0.98
	Coefficient of variation	0.08	0.04	0.01	0.01

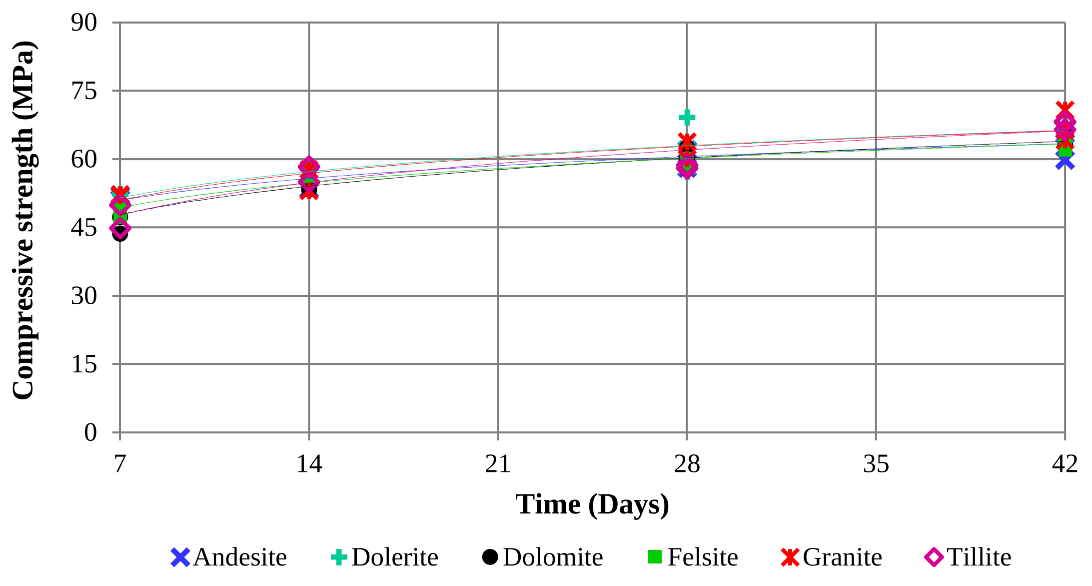

Figure 4-14: Effect of aggregate type on concrete strength development

The compressive strength development for the environmentally friendly concrete mixtures is shown in **Table 4-4** and **Figure 4-15**. It is well known that the replacement of cement with fly ash has a significant influence on the compressive strength development of concrete. This corresponds with results obtained in this study. The strength of the concrete decreased with increasing fly ash content. The water/binder ratio as well as the water content was kept constant for these mixtures. The strength reduction with increasing fly ash content could be limited by reducing the water content to maintain similar workability. The blended cement concrete containing 20% and 35% fly ash only reached a compressive strength of approximately 60 *MPa* after ninety-eight days of water curing, while the concrete containing 50% fly ash only reached a compressive strength of 57 *MPa* after two-hundred days of curing. This may be attributed to the insufficient amount of calcium hydroxide produce by the cement to react with the silica in the fly ash (Soutsos & Domone, 2017). The results do however evidently indicate that structural concrete can be manufactured with blended cements where up to 50% of the Portland cement was replaced with fly ash.

When the water and cement content in a mixture is reduced, the strength is, as expected, affected. The mixture containing superplasticiser (SP) showed higher compressive strength results compared to the dolomite mixture (FA0), while mixture containing RCA produced slightly lower compressive strengths results. The concrete mixture containing pure Portland cement, superplasticiser and dolomite aggregate (SP) reached a compressive strength of approximately 60 *MPa* after only seven-days of water curing, whereas the RCA and RCAF mixtures only reached a strength of approximately 60 *MPa* after fifty-six days of water curing. The compressive strength for the mixture containing RCA was greater than 30 *MPa*, even after only seven-days of water curing, indicating that structural concrete can easily be manufactured using RCA. The concrete containing RCA, fly ash as well as superplasticiser (RCAF) had slightly lower early-age strength compared to the concrete containing only RCA. However, the strength of the RCAF concrete surpasses that of the concrete containing only RCA after twenty-eight days of water curing. This can be attributed to the addition of both superplasticiser and fly ash in the RCAF mixture.

Table 4-4: Average compressive strength of environmentally friendly concrete

	Time (Days)	**7**	**14**	**28**	**56**	**98**	**120**
FA0	Compressive strength (*MPa*)	47.8	54.1	60.1	63.3	66.7	68.0
	Standard deviation (*MPa*)	3.10	0.52	1.97	0.74	2.01	1.79
	Coefficient of variation	0.06	0.01	0.03	0.01	0.03	0.03
FA20	Compressive strength (*MPa*)	36.9	40.5	50.1	57.1	63.3	63.8
	Standard deviation (*MPa*)	1.23	1.97	3.38	5.20	2.45	2.97
	Coefficient of variation	0.03	0.05	0.07	0.09	0.04	0.05
FA35	Compressive strength (*MPa*)	32.0	35.1	45.1	51.9	58.5	60.8
	Standard deviation (*MPa*)	1.84	0.04	3.51	5.01	2.60	3.54
	Coefficient of variation	0.06	0.00	0.08	0.10	0.04	0.06
FA50	Compressive strength (*MPa*)	19.9	26.6	33.1	40.8	47.0	51.1
	Standard deviation (*MPa*)	0.56	0.23	0.95	1.20	0.25	2.29
	Coefficient of variation	0.03	0.01	0.03	0.03	0.00	0.04
SP	Compressive strength (*MPa*)	59.1	65.7	67.0	76.1	75.0	79.9
	Standard deviation (*MPa*)	1.92	2.31	0.83	1.71	5.04	2.11
	Coefficient of variation	0.03	0.04	0.01	0.02	0.07	0.03
RCA	Compressive strength (*MPa*)	42.9	51.5	54.5	57.7	63.2	61.5
	Standard deviation (*MPa*)	2.43	3.39	0.47	1.90	0.59	3.58
	Coefficient of variation	0.06	0.07	0.01	0.03	0.01	0.06
RCAF	Compressive strength (*MPa*)	39.5	43.9	50.6	63.2	68.3	69.1
	Standard deviation (*MPa*)	0.33	1.54	7.94	0.21	2.99	0.51
	Coefficient of variation	0.01	0.04	0.16	0.00	0.04	0.01

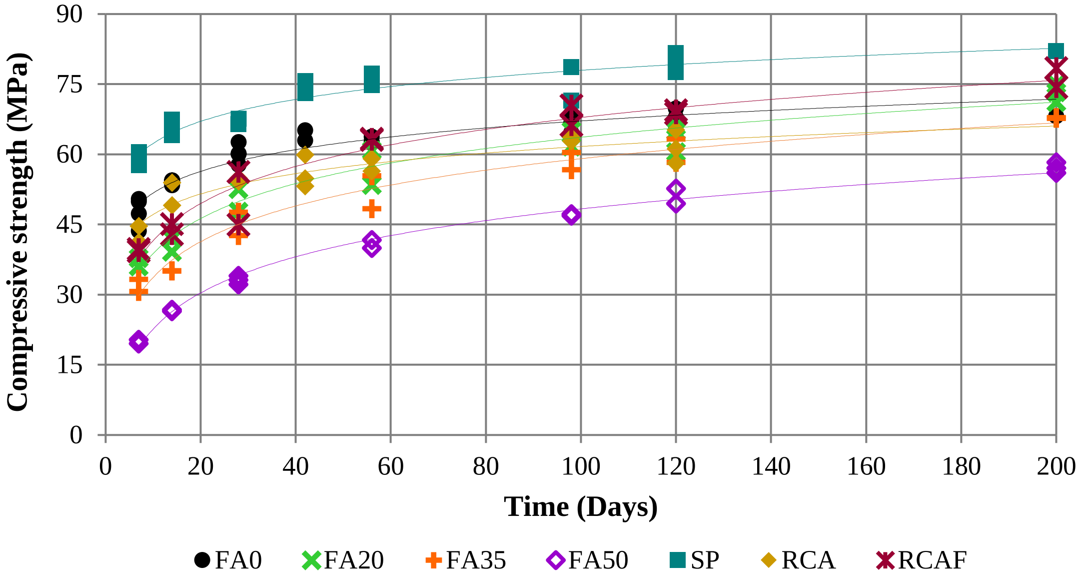

Figure 4-15: Compressive strength development of environmentally friendly concrete

4.5 SPLITTING TENSILE STRENGTH

Lower water/cement ratios produced concrete with higher splitting tensile strength as shown in **Table 4-5** and **Figure 4-16**. As mentioned previously, water/cement ratio has a substantial effect on the compressive strength of concrete. Consequently, the splitting tensile strength as a fraction of the compressive strength was also considered. According to literature, the splitting tensile strength should be in the range of 7% – 11% of the cube compressive strength (Soutsos & Domone, 2018). The results indicate the notable difference in tensile strength due to varying water/cement ratio. The mixture with a water/cement ratio of 0.3 produced an average splitting tensile strength 1.93 *MPa* higher than that of the mixture with a water/cement ratio of 0.7. However, when considering the splitting tensile strength as a fraction of the compressive strength, the mixture with a water/cement ratio of 0.7 yield the highest ratio at approximately 9%.

Table 4-5: Average splitting tensile strength of concrete with varying w/c ratios

	D0.3	D0.4	D0.5	D0.6	D0.7
Splitting tensile strength (*MPa*)	5.65	5.03	4.63	4.25	3.73
Standard deviation (*MPa*)	0.26	0.36	0.42	0.31	0.17
Coefficient of variation	0.05	0.07	0.09	0.07	0.05

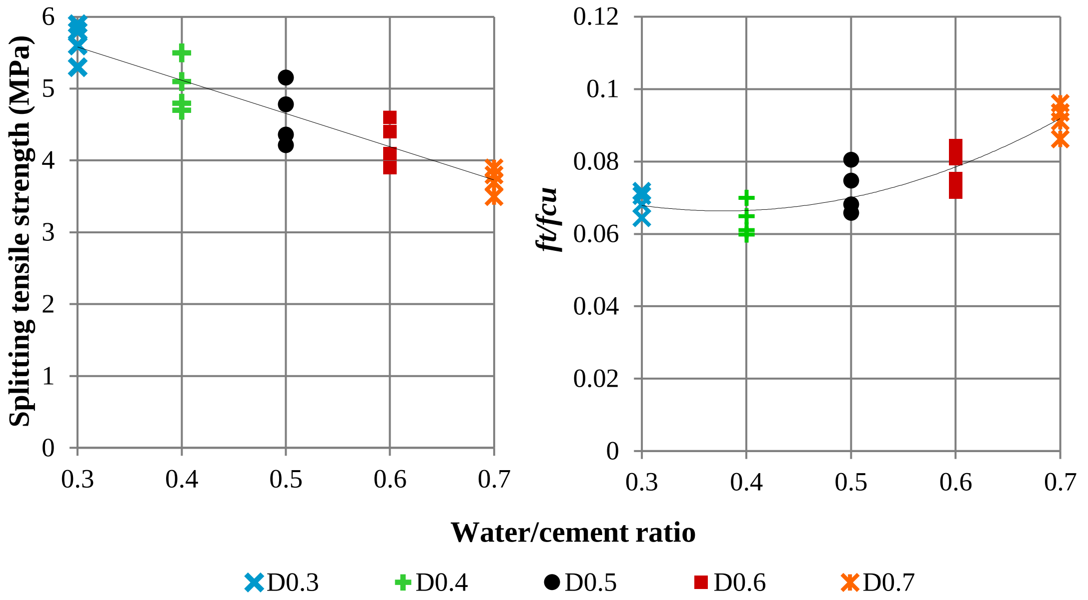

Figure 4-16: Splitting tensile strength of concrete with different w/c ratios

The effect of coarse to total aggregate ratio as well as the aggregate type on the splitting tensile strength of concrete is indicated in **Table 4-6** and **Figure 4-17**. According to literature, aggregate type has a limited effect on the splitting tensile strength of concrete. This corresponds to the results obtained in this study. The results indicate that coarse to total aggregate ratio influence the splitting tensile strength of concrete to a greater extent than aggregate type. Mortar mixtures produced higher tensile strength values compare to mixtures containing more coarse aggregate.

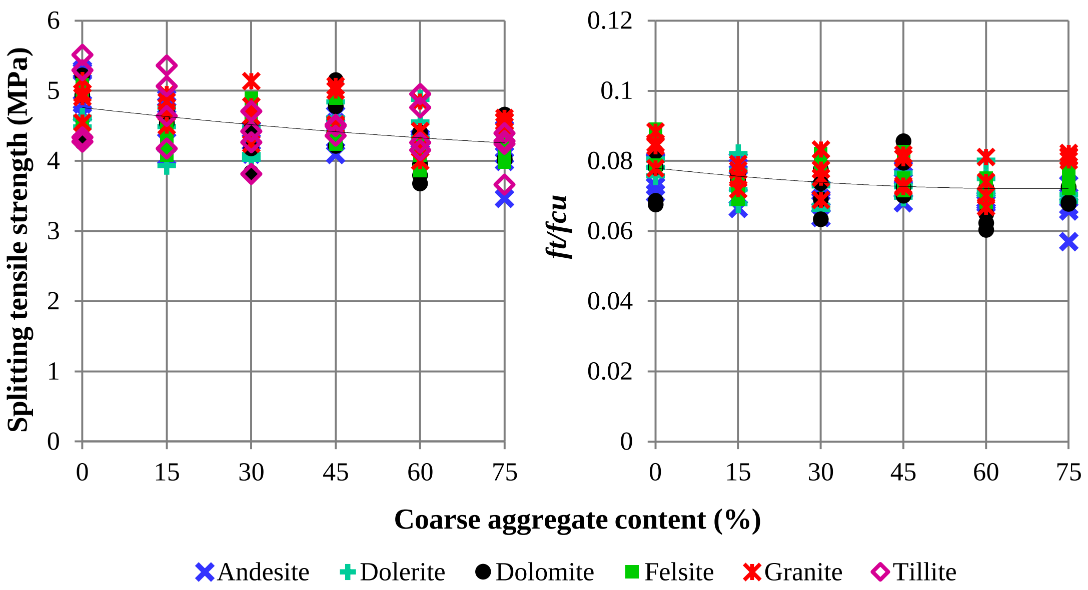

Figure 4-17: Effect of aggregate on the splitting tensile strength of concrete

Table 4-6: Effect of aggregate on the splitting tensile strength of concrete

		Splitting tensile strength (*MPa*)	Standard deviation (*MPa*)	Coefficient of variation
Andesite	**A0**	5.02	0.32	0.06
	A15	4.51	0.40	0.09
	A30	4.38	0.22	0.05
	A45	4.40	0.25	0.06
	A60	4.30	0.04	0.01
	A75	4.00	0.41	0.10
Dolerite	**R0**	4.51	0.13	0.03
	R15	4.34	0.36	0.08
	R30	4.25	0.23	0.05
	R45	4.61	0.18	0.04
	R60	4.51	0.27	0.06
	R75	4.34	0.06	0.01
Dolomite	**D0**	4.69	0.47	0.10
	D15	4.62	0.07	0.02
	D30	4.29	0.37	0.09
	D45	4.63	0.42	0.09
	D60	3.96	0.32	0.08
	D75	4.25	0.30	0.07
Felsite	**F0**	4.90	0.27	0.05
	F15	4.33	0.19	0.04
	F30	4.75	0.11	0.02
	F45	4.47	0.30	0.07
	F60	4.12	0.19	0.05
	F75	4.15	0.17	0.04
Granite	**G0**	4.89	0.25	0.05
	G15	4.75	0.20	0.04
	G30	4.70	0.37	0.08
	G45	4.77	0.31	0.06
	G60	4.37	0.37	0.08
	G75	4.56	0.05	0.01
Tillite	**T0**	4.86	0.64	0.13
	T15	4.81	0.52	0.11
	T30	4.30	0.37	0.09
	T45	4.46	0.08	0.02
	T60	4.54	0.39	0.09
	T75	4.15	0.33	0.08

The splitting tensile strength for the environmentally friendly concrete mixtures is shown in **Table 4-7** and **Figure 4-18**. It was decided to test the splitting tensile strength of the mixtures containing fly ash only after reaching an equivalent strength to that of the pure Portland cement concrete (FA0). The fly ash mixtures only reached a compressive strength of approximately 60 *MPa* after ninety-eight days of water curing. Consequently, the splitting tensile strength as a fraction of the compressive strength was also considered. The ratio increases with increasing fly ash content up to approximately 11%. The mixture containing 35% fly ash appear to be an optimum replacement percentage when considering splitting tensile strength at later ages. However, further research and testing is required to confirm these findings.

Corresponding with literature, the results indicate that the splitting tensile strength for the concrete containing only RCA fall well within the range of splitting tensile strengths for the different natural aggregate types (4.2 *MPa* to 4.8 *MPa*). Consequently, the tensile strength of concrete will not be influenced substantially when using RCA as an alternative aggregate source.

The concrete mixture containing pure Portland cement, superplasticiser and dolomite aggregate (SP) produced slightly higher tensile strength results compared to the dolomite mixture (FA0). As mentioned, the addition of 35% fly ash substantially increased the splitting tensile strength. This can also be seen in the combined mixture (RCAF), where the splitting tensile strength is higher than both the mixtures containing RCA and superplasticiser (SP). Therefore, increased tensile strengths can be obtained by reducing the water and cement content of the mixture as well as utilising blended cements.

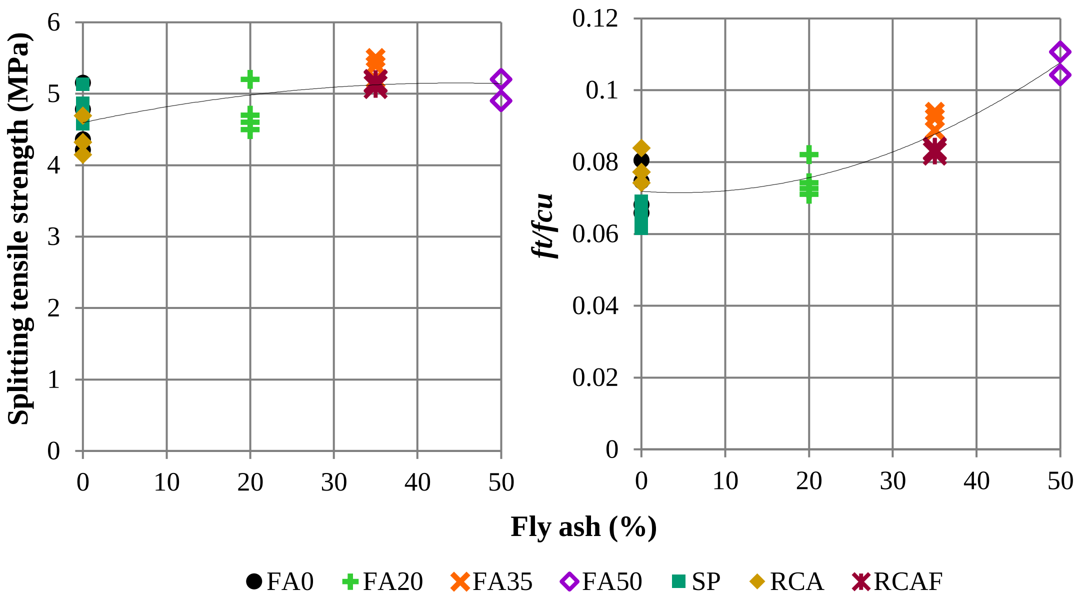

Figure 4-18: Splitting tensile strength of environmentally friendly concrete

Table 4-7: Average splitting tensile strength of environmentally friendly concrete

	FA0	FA20	FA35	FA50	SP	RCA	RCAF
Splitting tensile strength (*MPa*)	4.63	4.75	5.38	5.05	4.85	4.37	5.15
Standard deviation (*MPa*)	0.42	0.31	0.13	0.17	0.23	0.23	0.05
Coefficient of variation	0.09	0.07	0.02	0.03	0.05	0.05	0.01

There is no simple relationship between tensile strength of concrete and its compressive strength. Factors such as water/cement ratio, mix proportions and properties of aggregate affect the compressive strength of concrete in different ways. It is also important to note that these factors may not affect the splitting tensile strength of concrete to the same degree. According to Loedolff & Chambers (1994), the relationship between average compressive strength (f_{cu}) and splitting tensile strength (f_t) is $f_t = 0.185f_{cu}^{0.783}$. Eurocode 2, *BS EN 1992-1-1*, state the relationship as $f_t = 0.3f_{ck,cyl}^{0.67}$ with $f_{ck,cyl}$ as the characteristic cylinder compressive strength. The relationship for cylinder compressive strength to cube compressive strength was given as $f_{cyl} = 0.8f_{cu}$, while the characteristic strength (f_{ck}) was converted to a target mean strength (f_{cm}) by using the relationship given in Equation 3-1. The relationship between the average compressive strength and the splitting tensile strength of concrete is illustrated in **Figure 4-19**. The correlation between the experimental results and that of previously published relationships (Loedolff & Chambers, 1994) is marginally different in the lower compressive strength range. Fairly similar results can however be seen for compressive strengths between 60 *MPa* and 80 *MPa*. The relationship derived from Eurocode 2, greatly underestimates the splitting tensile strength values.

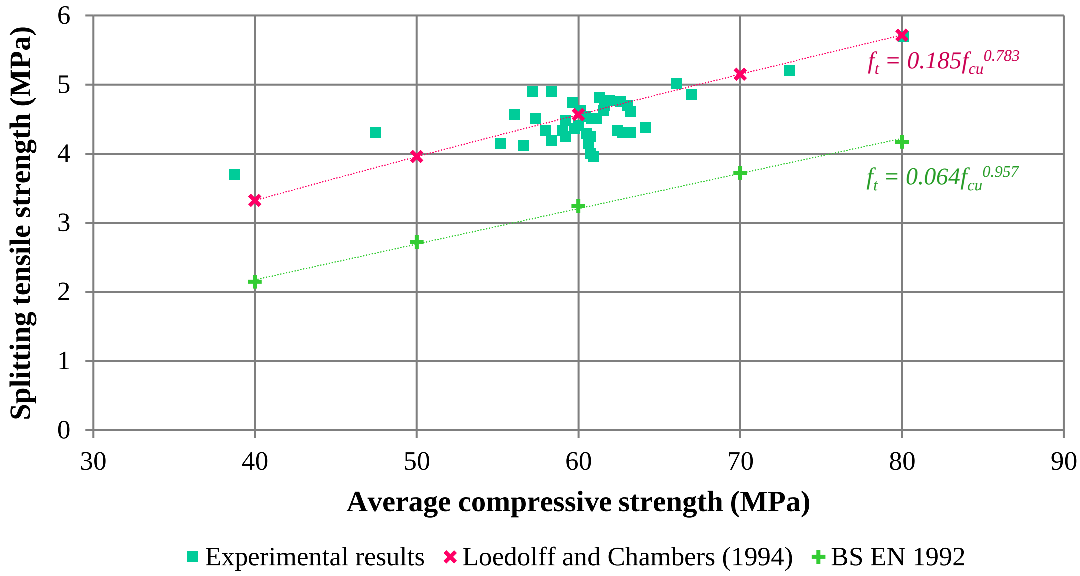

Figure 4-19: Compressive and splitting tensile strength

4.6 MODULUS OF ELASTICITY

It is well known that the elastic modulus of concrete will slightly increase with increasing concrete strength. Thus, lower water/cement ratios and increased age both influence the modulus of elasticity of concrete. This corresponds to the results obtained in this study and the effect of water/cement ratio is shown in **Table 4-8** and **Figure 4-20**. The results indicate that lower water/cement ratios, hence concrete with higher compressive strength produce concrete with higher stiffness.

Table 4-8: Average modulus of elasticity of concrete with varying w/c ratios

	D0.3	D0.4	D0.5	D0.6	D0.7
Modulus of elasticity (*GPa*)	41.4	40.5	38.6	34.6	29.5
Standard deviation (*GPa*)	0.30	1.40	0.98	2.04	0.13
Coefficient of variation	0.01	0.03	0.03	0.06	0.00

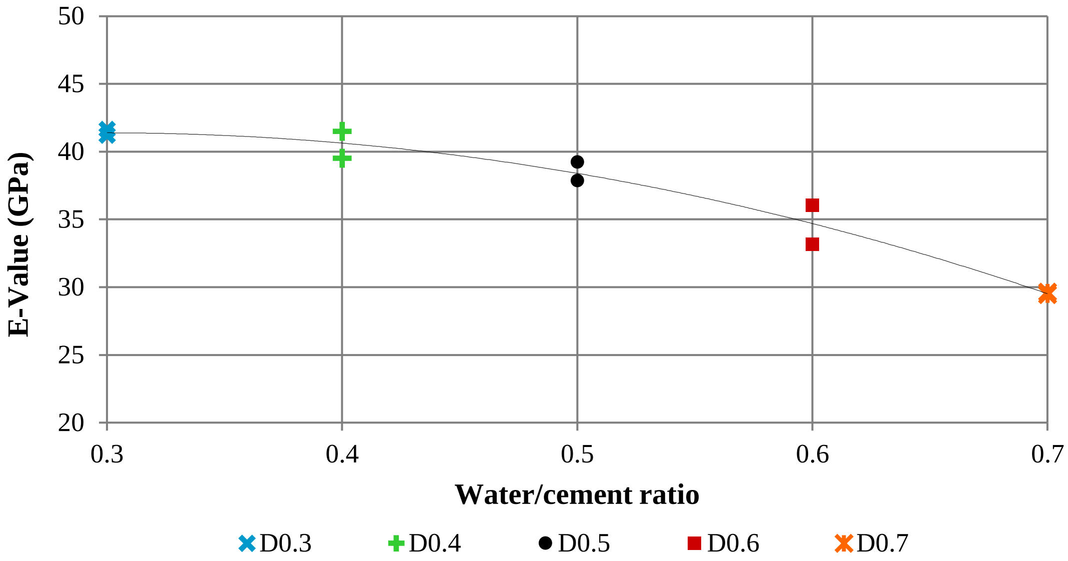

Figure 4-20: Modulus of elasticity of concrete with varying w/c ratio

It should be noted that concretes with equal strength may yield different elastic moduli if different aggregates or different aggregate/cement ratios are used. The modulus of elasticity for concrete with varying coarse to total aggregate ratios as well as aggregate types is shown in **Table 4-9** and **Figure 4-21.** A slight increase in the modulus of elasticity can be observed with increasing coarse to total aggregate ratios, expect for the concrete containing tillite aggregate. It is clear from the results that aggregate type has a notable effect on the elastic modulus of concrete. Consequently, the elastic modulus of concrete is related to the stiffness of the aggregate (**Figure 4-22**).

Table 4-9: Effect of aggregate on the modulus of elasticity of concrete

		Modulus of elasticity (GPa)	Standard deviation (GPa)	Coefficient of variation
Andesite	A0	30.3	0.91	0.03
	A15	30.9	0.79	0.03
	A30	33.2	1.77	0.05
	A45	31.8	0.14	0.00
	A60	33.9	0.16	0.00
	A75	31.7	0.19	0.01
Dolerite	R0	31.4	0.55	0.02
	R15	32.1	0.33	0.01
	R30	34.3	1.79	0.05
	R45	34.2	0.62	0.02
	R60	35.0	0.95	0.03
	R75	34.8	0.19	0.01
Dolomite	D0	35.6	0.76	0.02
	D15	36.9	0.44	0.01
	D30	38.2	0.28	0.01
	D45	38.6	0.98	0.03
	D60	38.4	2.80	0.07
	D75	40.0	2.40	0.06
Felsite	F0	30.9	0.21	0.01
	F15	31.3	0.23	0.01
	F30	31.3	1.28	0.04
	F45	34.7	0.22	0.01
	F60	35.4	1.16	0.03
	F75	34.0	1.30	0.04
Granite	G0	29.9	0.21	0.01
	G15	30.4	0.07	0.00
	G30	31.1	0.94	0.03
	G45	31.2	0.09	0.00
	G60	30.4	0.14	0.00
	G75	31.0	0.02	0.00
Tillite	T0	30.2	0.30	0.01
	T15	28.9	0.10	0.00
	T30	29.1	0.87	0.03
	T45	28.6	0.41	0.01
	T60	29.5	0.01	0.00
	T75	29.6	1.27	0.04

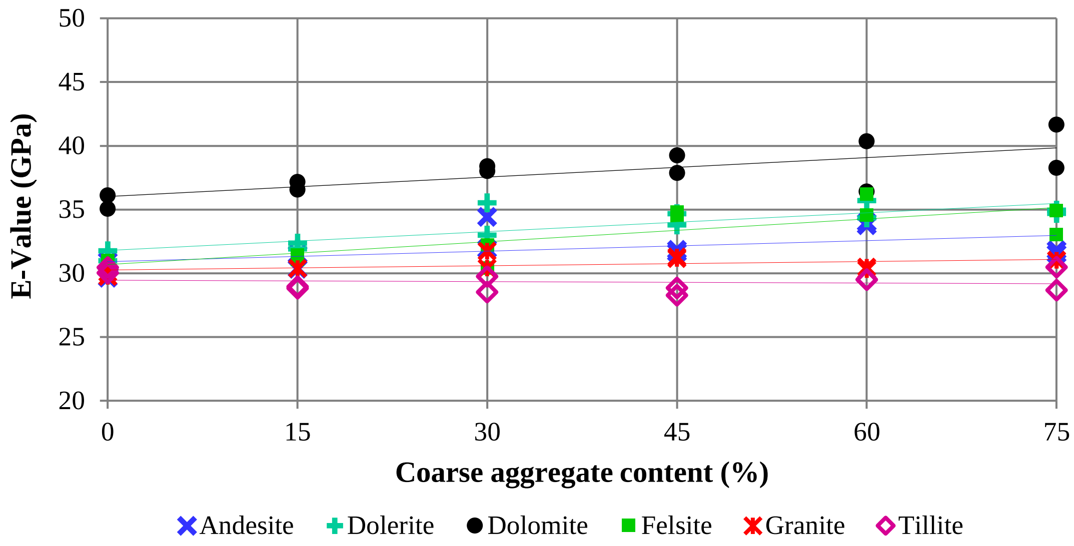

Figure 4-21: Effect of aggregate on the modulus of elasticity of concrete

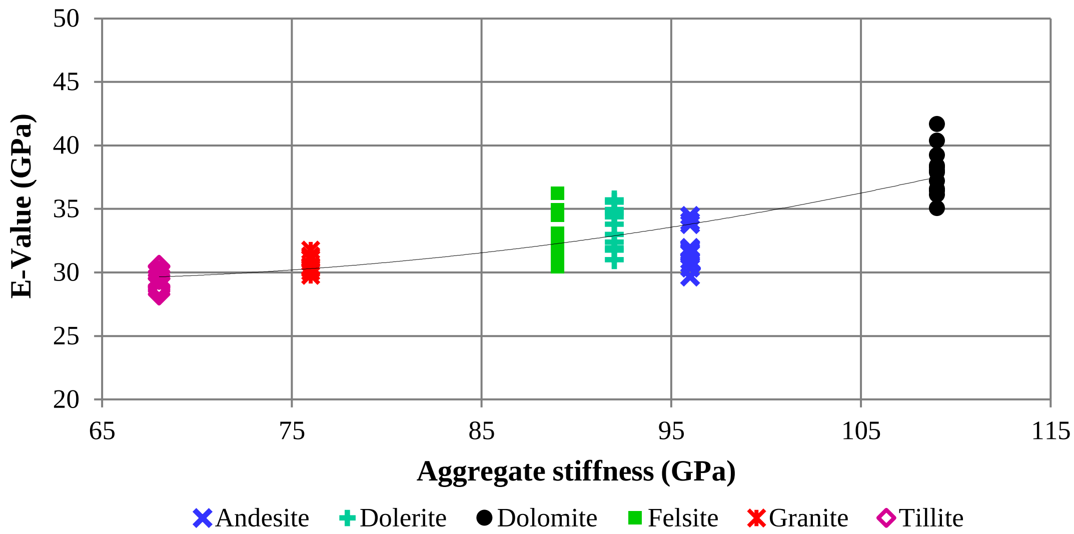

Figure 4-22: Modulus of elasticity of concrete versus aggregate stiffness

The results for 45% coarse to total aggregate ratio correlate well with previously published values, as shown in **Figure 4-23**. Previously published values obtained from Alexander & Davis (1994) indicate modulus of elasticity values for a compressive strength range of 30 – 70 *MPa*. Corresponding to literature, dolomite aggregate produce concrete with the highest modulus of elasticity (38.6 *GPa*), while granite aggregate produced concrete with a lower modulus of elasticity due to the slightly weathered nature (31.2 *GPa*). Furthermore, tillite aggregate produced concrete with the lowest modulus of elasticity (28.6 *GPa*) compared to the other aggregate types. The results highlight the importance of considering aggregate type when estimating the stiffness of concrete elements in structures.

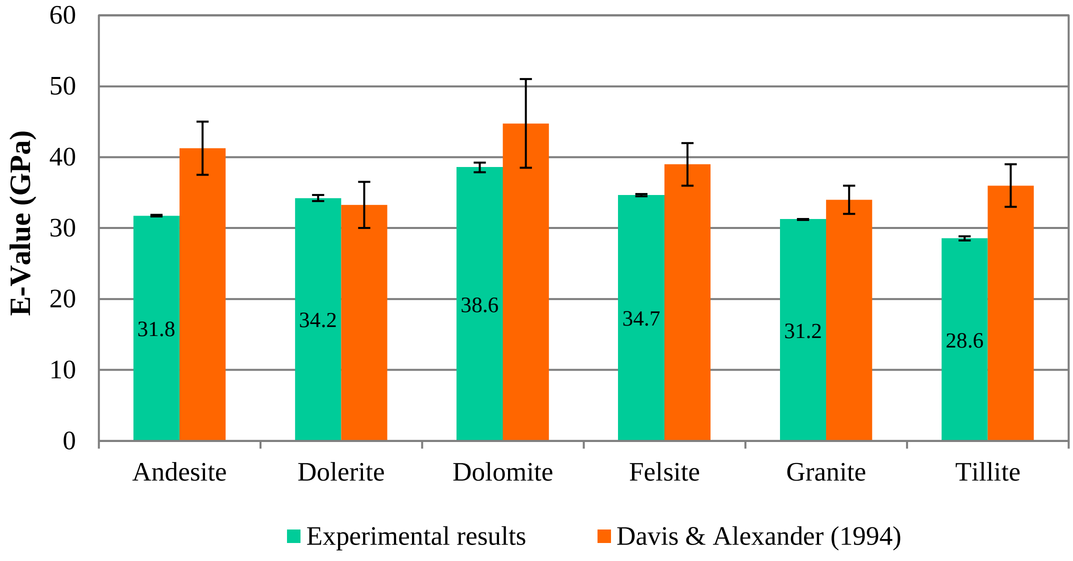

Figure 4-23: Modulus of elasticity compared to previously published results

The elastic modulus of environmentally friendly concrete is tabulated in **Table 4-10** and illustrated in **Figure 4-24**. As mentioned, the material properties of mixtures containing fly ash was only measured after reaching an equivalent strength to that of the pure Portland cement concrete (FA0). The fly ash mixtures only reached a compressive strength of approximately 60 *MPa* after ninety-eight days of water curing and the modulus of elasticity of the blended cement concrete was therefore only measured after ninety-eight days.

Table 4-10: Average modulus of elasticity of environmentally friendly concrete

	FA0	FA20	FA35	FA50	SP	RCA	RCAF
Modulus of elasticty (*GPa*)	38.6	39.5	39.1	36.2	46.1	29.5	34.6
Standard deviation (*GPa*)	0.98	1.33	0.13	0.04	1.11	0.27	0.61
Coefficient of variation	0.03	0.03	0.00	0.00	0.02	0.01	0.02

The results obtained corresponds to that of literature as there is no substantial difference in the modulus of elasticity of the blended cement concretes compared to the pure Portland cement concrete. A noticeable increase in the modulus of elasticity can be observed for the concrete containing pure Portland cement, superplasticiser and dolomite aggregate (SP). This can be attributed to the increased compressive strength of this mixtures. According to literature, the modulus of elasticity for RAC is normally lower than that of conventional concrete which may be attributed to the lower stiffness of RCA. This corresponds with the results obtained in this study. The combined concrete (RCAF) had a lower elastic modulus (34.6 *GPa)* compared to the other concrete mixtures, while concrete containing only RCA had the lowest stiffness of 29.5 *GPa*. Even though the modulus of elasticity of the concrete containing RCA is substantially lower than the dolomite mixture (FA0), it still falls within the stiffness range of 28.6 *GPa* to 38.5 *GPa* as measured for concrete containing different natural aggregates.

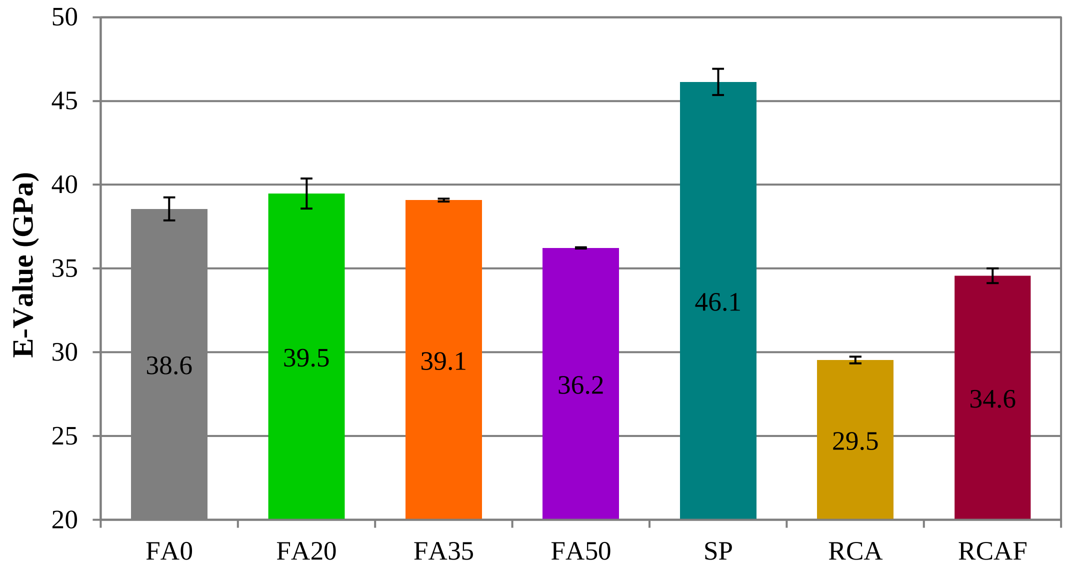

Figure 4-24: Modulus of elasticity of environmentally friendly concrete

The measured modulus of elasticity is compared against the compressive strength for all the dolomite mixtures with varying water/cement ratios and cement types, as seen in **Figure 4-25**. The general trend indicates that concrete with reduced strength produce lower modulus of elasticity values, as expected. However, the increase in modulus of elasticity seems to plateau at compressive strength higher than approximately 65 *MPa*. Known relationships between compressive strength and modulus of elasticity obtained from literature is shown with the experimental results. According to the South African National Standard, *SANS 10100-1*, the relationship between characteristic compressive strength ($f_{ck,cu}$) and modulus of elasticity (E) is $E = 20 + 0.2\ f_{ck,cu}$. Eurocode 2, *BS EN 1992-1-1*, state the relationship as $E = 10\ f_{cu}^{0.31}$, with f_{cu} as the average compressive strength, while Alexander & Mindess (2005) state the relationship between characteristic compressive strength and modulus of elasticity of concrete containing dolomite aggregate as $E = 24 + 0.45\ f_{ck,cu}$. It can be seen that the relationship derived from Alexander & Mindess (2005), greatly overestimates the modulus of elasticity values, while the South African National Standard underestimates them. Concrete compressive strength less than approximately 60 *MPa* is normally considered as conventional concrete (normal strength concrete). However, the range of compressive strengths obtained in this study was between 35 *MPa* – 85 *MPa*. The relationship derived from Alexander & Mindess (2005) is only valid for characteristic strengths in a range of 20 *MPa* – 70 *MPa*, while the South African National Standard is only valid for characteristic strengths in a range of 20 *MPa* – 60 *MPa*. Different gauge lengths and sample shapes may also influence the relationships derived from literature.

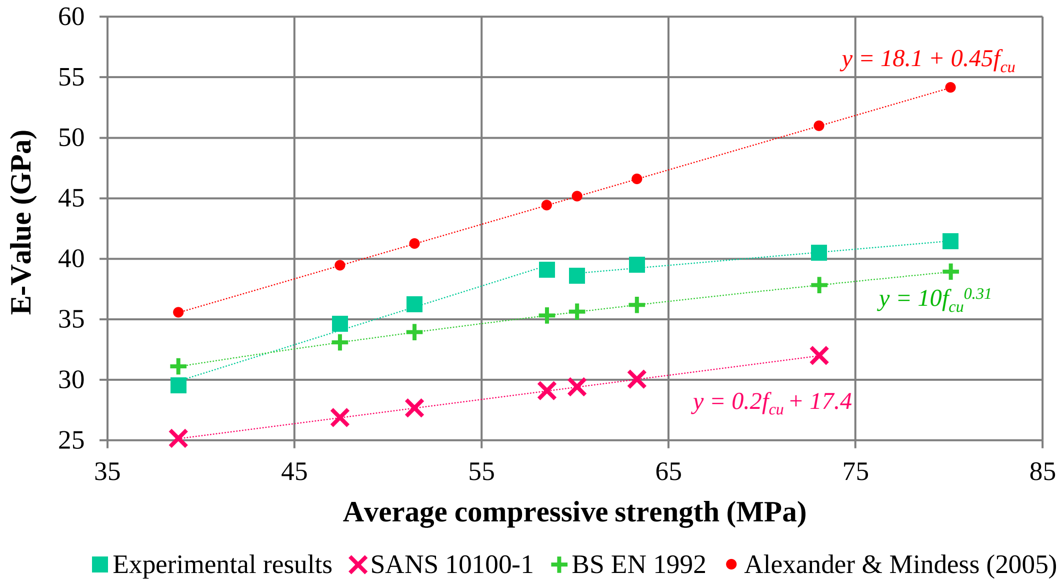

Figure 4-25: Relationship between the compressive strength and stiffness of concrete

4.7 THERMAL EXPANSION

As mentioned, limited data is available on the thermal expansion coefficient of South African concrete and most of the available literature date back to the early 1990's. According to literature, factors such as cement type, aggregate volume concentration and concrete strength should have an insignificant effect on the CTE. However, aggregate type and moisture condition at time of testing greatly influence the CTE. The moisture condition of the test specimen (saturated, air dried or oven dried) in previous literature is unclear, making it difficult to compare measured results to previous studies. The effect of varying water/cement ratios on the CTE of concrete is illustrated in **Figure 4-26**. The water content in all the mixtures were kept constant at 230 l/m^3 and the cement content was adjusted to obtain the desired water/cement ratio. The CTE decreased as the water/cement ratio increased, up to 0.5. This may be attributed to the higher cement content in the mixtures with low water/cement ratio. Therefore, concrete with higher cement content would deform substantially more than concrete with lower cement content. Unexpected CTE results were obtained for water/cement ratios higher than 0.5. Further research and testing is required to confirm these findings.

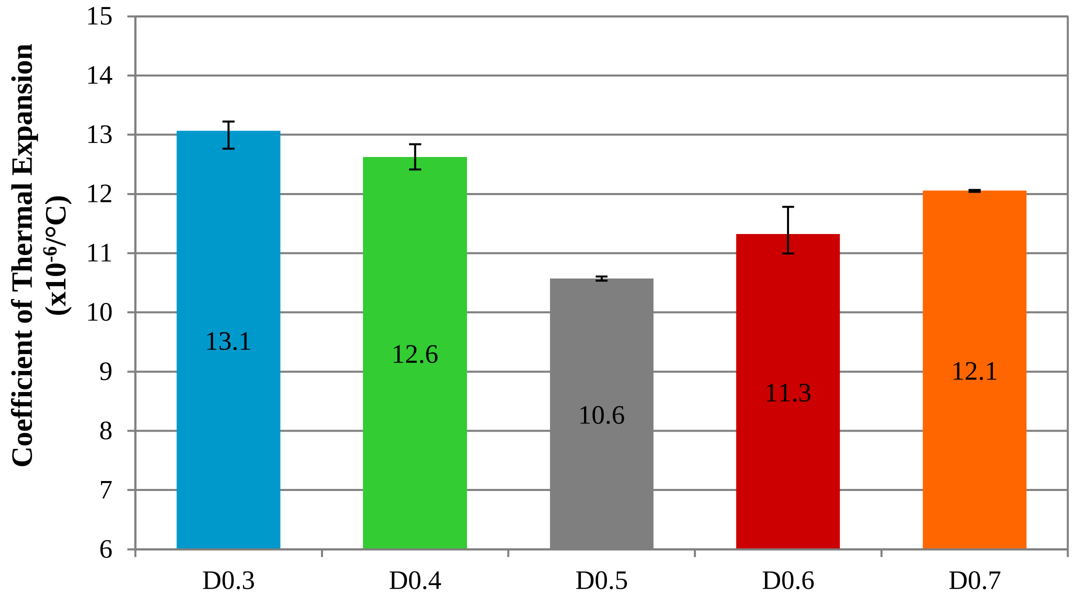

Figure 4-26: CTE of concrete with varying w/c ratio

The effect of coarse to total aggregate ratio and aggregate type on the CTE is illustrated in **Figure 4-27** and **Figure 4-28** respectively. The results indicate that the fineness of the aggregate or the coarse aggregate content have little effect on the measured CTE of concrete. Corresponding to literature, aggregate type substantially influence the CTE. Under similar temperature condition, concrete containing dolomite and granite would deform substantially more than concrete containing andesite or dolerite. The measured CTE values correlate well with the results obtained in previous studies. The only value that differs substantially from published results are that of dolomite.

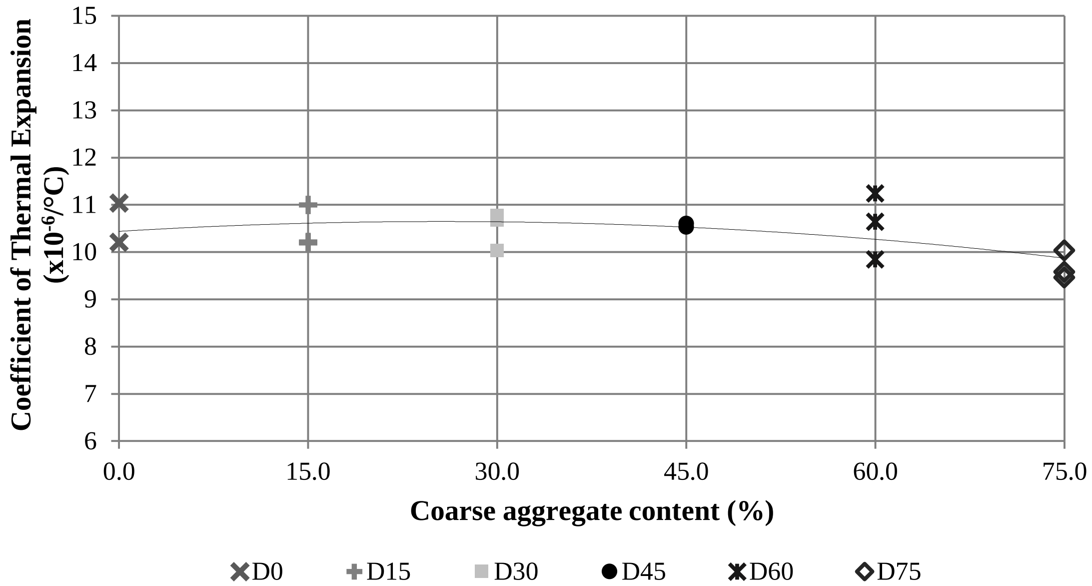

Figure 4-27: CTE of concrete with varying coarse to total aggregate ratios

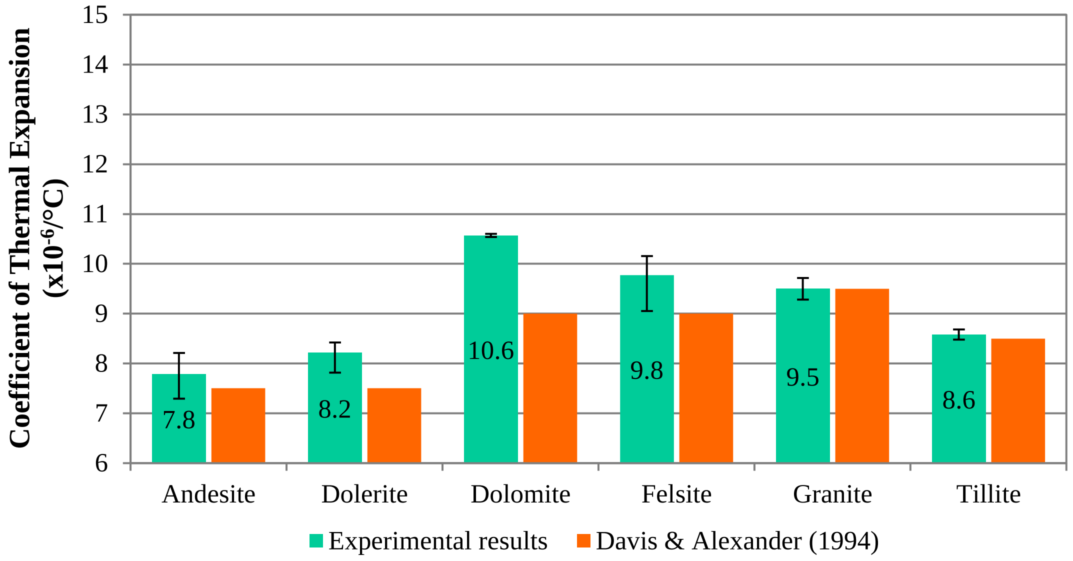

Figure 4-28: CTE of concrete with different aggregate types

The coefficient of thermal expansion for the different environmentally friendly concrete mixtures is shown in **Figure 4-29**. Corresponding to literature, cement type, hence blended cements had a limited effect on the thermal expansion coefficient. There is a negligible difference in the CTE between the dolomite concrete (FA0) and the concrete containing pure Portland cement, superplasticiser and dolomite aggregate (SP). This supports the statement made in literature, concrete strength should have an insignificant effect on the CTE. As mentioned in literature, under similar temperature circumstances, concrete containing RCA would deform substantially more than concrete containing natural aggregates. This corresponds to the results obtained in this study.

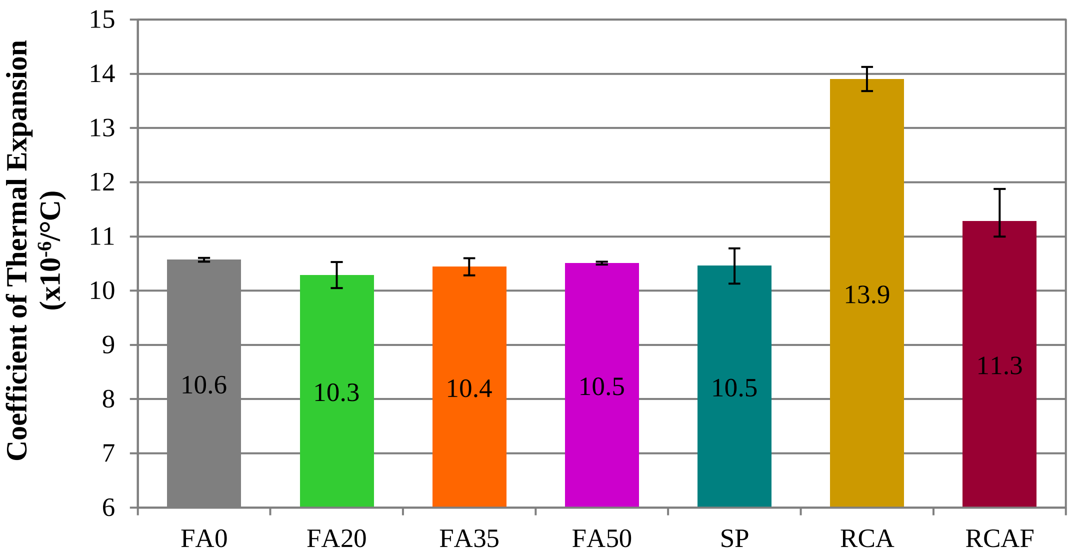

Figure 4-29: CTE of environmentally friendly concrete mixtures

It can be concluded that cement type, coarse to total aggregate ratio and concrete strength has a limited effect on the CTE of concrete, while aggregate type greatly influences the CTE.

4.8 DRYING SHRINKAGE

Concrete with reduced water/cement ratios generally have reduced shrinkage. However, it is important to note that the original water content (by mass) of a concrete mixture is directly proportional to the shrinkage of the concrete. All the mixtures had a constant water content of 230 l/m^3, while the cement content was adjusted to obtain the desired water/cement ratio. Therefore, mixtures with lower water/cement ratios had higher cement content and consequently higher paste content which resulted in an increase in drying shrinkage as illustrated in **Figure 4-30**. It would therefore be recommended to rather limit the cement content and reduce the water content while incorporating admixtures to maintain similar workability when designing higher strength concrete mixtures.

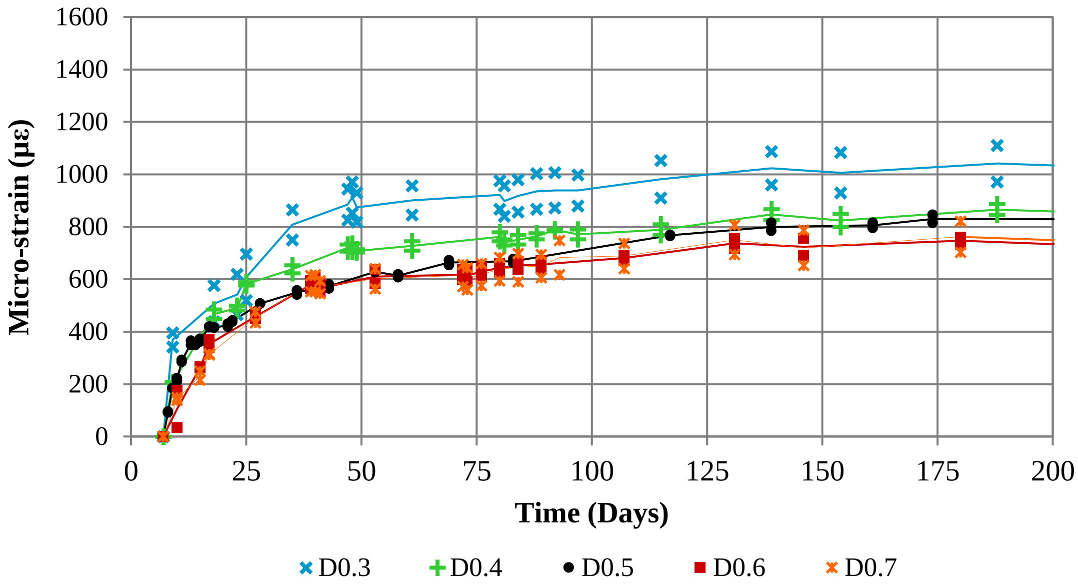

Figure 4-30: Drying shrinkage of concrete with different w/c ratios

As mentioned, the addition of both fine and coarse aggregates masks the paste behaviour in concrete. The drying shrinkage of paste compared to both mortar and concrete can be seen in **Figure 4-31**. According to literature, an 80% – 95% reduction in the paste shrinkage can be expected for conventional concrete. The drying shrinkage of the paste alone after two hundred days was approximately 4500 *micro-strain*. This was significantly reduced to just below 1000 *micro-strain* with the addition of aggregates. A reduction of approximately 80% in the paste shrinkage was obtained with the addition of aggregates.

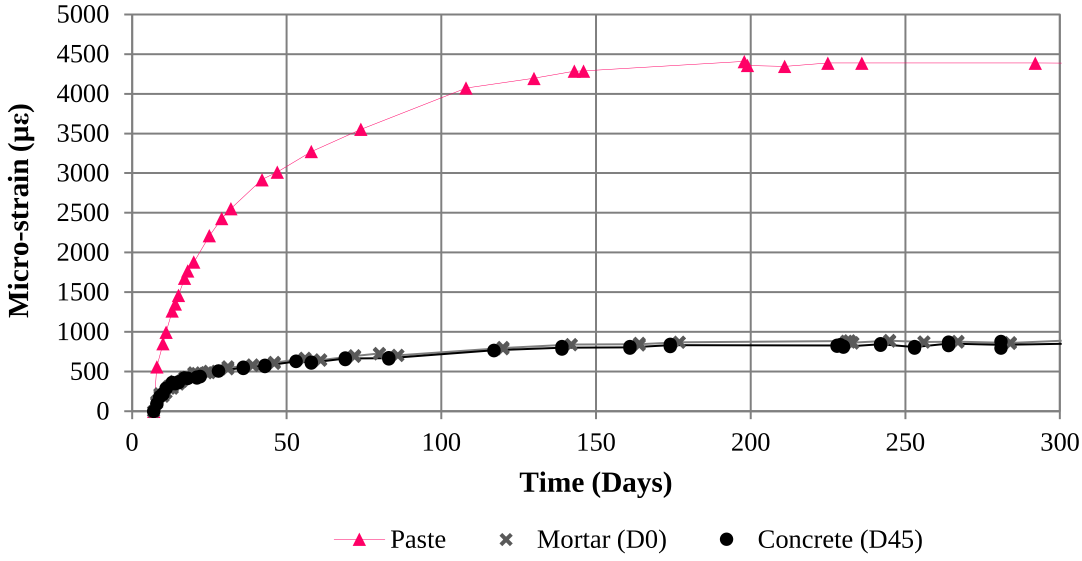

Figure 4-31: Drying shrinkage of paste compared to mortar and concrete

The effect of varying coarse to total aggregate ratio on the drying shrinkage of concrete can be seen in **Figure 4-32**. An increase in the coarse to total aggregate ratio resulted in a slight reduction in drying shrinkage. It can therefore be concluded that the fineness of the aggregate or the coarse aggregate content have little effect on the measured drying shrinkage of concrete.

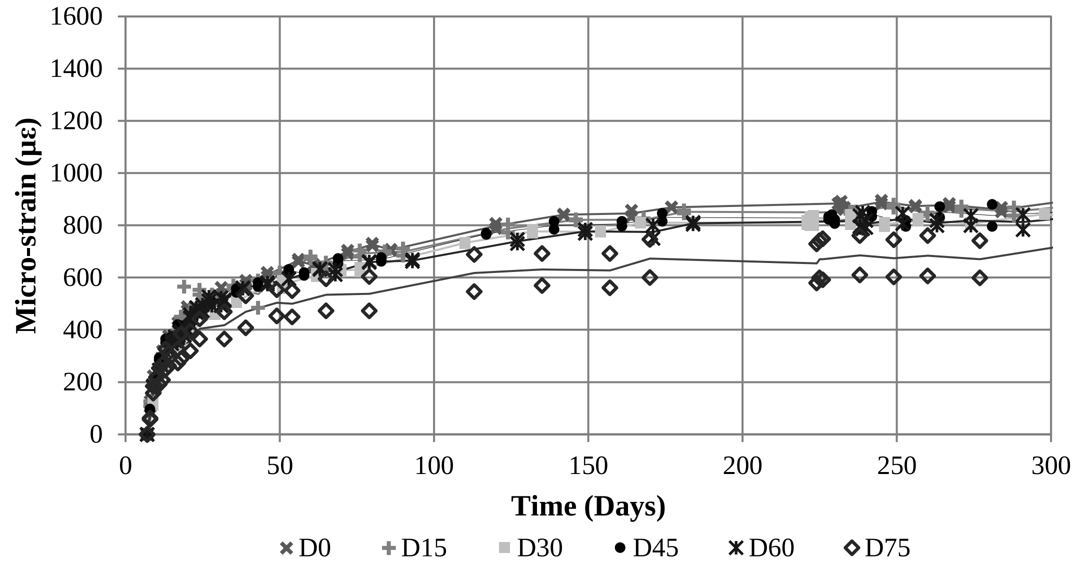

Figure 4-32: Drying shrinkage of concrete with varying coarse to total aggregate ratio

It is well known that aggregates with higher stiffness generally provide greater resistance to volume change in concrete. Furthermore, according to literature concrete containing dolomite aggregate typically exhibit low drying shrinkage compared to concrete containing dolerite and tillite. This corresponds to the results obtained in this study as illustrated in **Figure 4-33**. The results show that concrete made with andesite, dolomite and felsite have similar drying shrinkage, with dolerite and granite slightly higher. Concrete made with tillite showed nearly double the drying shrinkage compared to concrete containing andesite, dolomite and felsite. The drying shrinkage of concrete produced with most of the aggregates after three hundred days was approximately 800 to 900 *micro-strain*, while the tillite specimen experienced drying shrinkage of up to 1400 *micro-strain*. The results highlight the importance of proper aggregate selection as aggregate type can significantly influence the deformation behaviour of concrete structures.

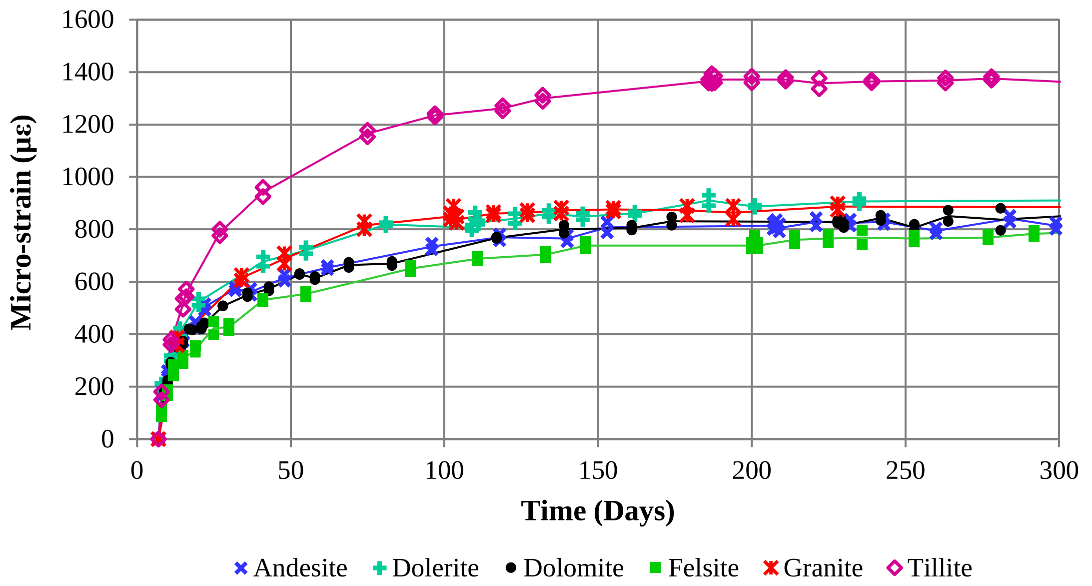

Figure 4-33: Drying shrinkage of concrete containing different aggregate types

The measured drying shrinkage for the different environmentally friendly concrete mixtures is shown in **Figure 4-34**. According to literature the addition of fly ash has little effect on the drying shrinkage of concrete, compared to pure Portland cement concrete. This was confirmed by the results obtained in this study. The insignificant difference can be attributed to the constant water content (230 l/m^3) in all mixtures. If the water content was reduced to keep similar workability, the use of fly ash blended cements might result in a reduction in measured drying shrinkage. The drying shrinkage was substantially reduced compared to the dolomite concrete (FA0), when superplasticiser was added to the mixture, and the water and cement content reduced. Consequently, drying shrinkage can be reduced by lowering the paste content in a mixture. According to literature an increase in drying shrinkage can occur due to the considerable amount of old cement paste in the recycled sand. The lower stiffness of the recycled aggregates may also contribute to less restraint to shrinkage. This corresponds with the results obtained in this study as the recycled aggregate concrete measured substantially higher drying shrinkage compared to the dolomite concrete (FA0). After hundred and twenty days, the recycled aggregate concrete (RCA) still had lower drying shrinkage compared to concrete containing Tillite. The combined concrete (RCAF) experience similar drying shrinkage compared to that of the blended cement concretes. The results prove that RCA can be used to produce structural concrete.

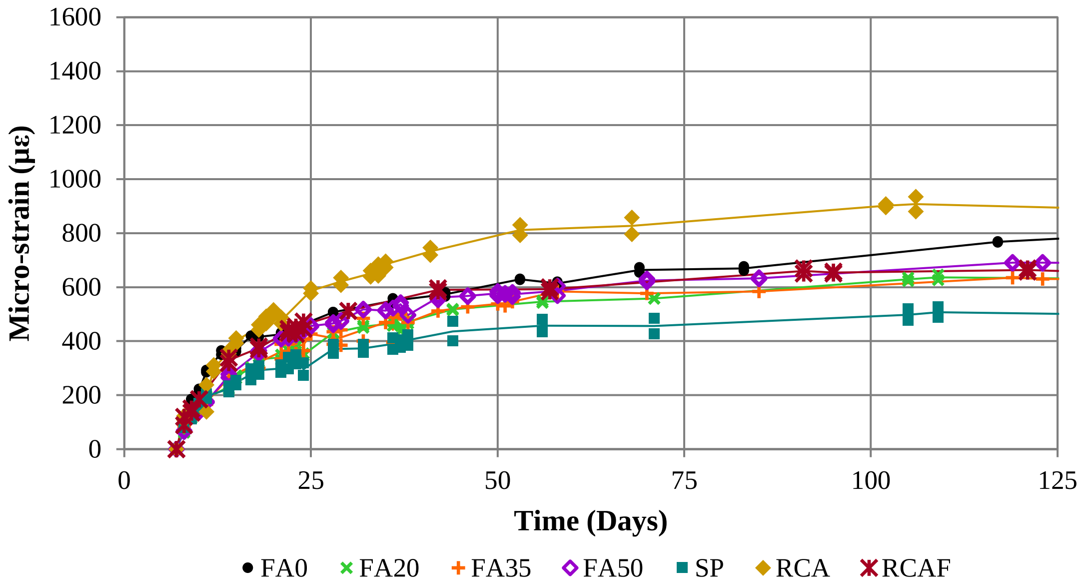

Figure 4-34: Drying shrinkage of environmentally friendly concrete

4.9 POTENTIAL CONCRETE DURABILITY

The potential durability i.e., Oxygen Permeability Index (OPI) and porosity of concrete containing different aggregate types, varying coarse to total aggregate ratios, blended cements as well as superplasticiser is discussed and classified according to recommendations in literature (Alexander et al., 1999; Moore et al., 2021). The effect of coarse to total aggregate ratio and aggregate type on potential concrete durability is illustrated in **Figure 4-35**. The measured OPI values decreased as the coarse aggregate content increased. The porosity results on the other hand show improved performance with an increase in coarse to total aggregate ratios. According to the suggested durability classification for OPI values by Alexander et al. (1999), the mixture with a 75% coarse to total aggregate ratio produced good durability concrete whereas all the other fractions were classified as excellent durability concrete. Notable variations in the measured OPI values were obtained between the different aggregate types. Concrete containing granite, dolomite and tillite all produced excellent durability concrete, while dolerite, felsite and andesite were classified as only good durability concrete. All mixtures were classified as poor to very poor durability concrete when considering the suggested durability classification for porosity values by Moore et al. (2021).

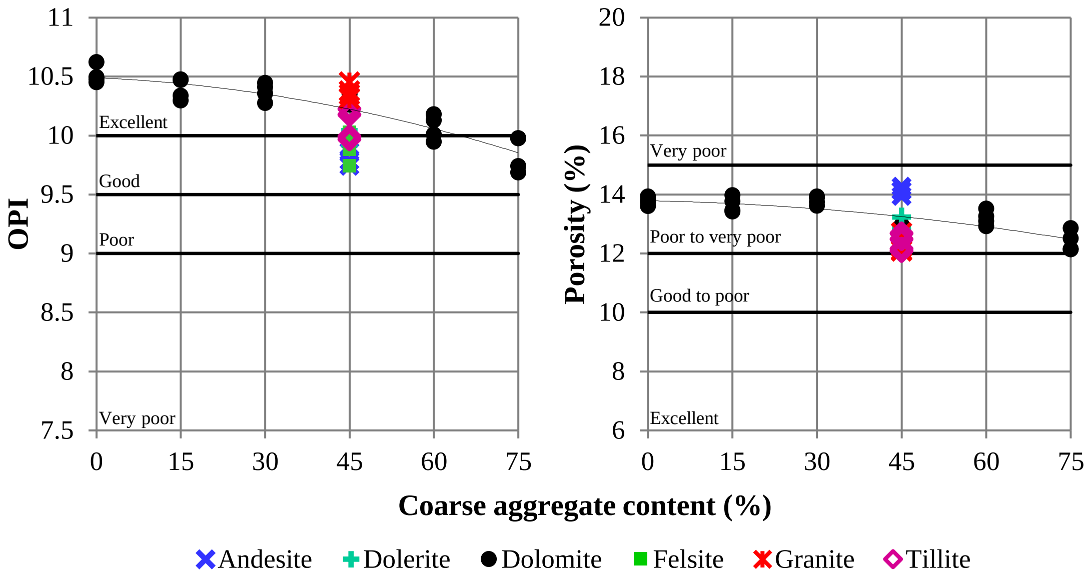

Figure 4-35: Effect of aggregate on potential concrete durability

The Oxygen Permeability Index (OPI) and porosity of the environmentally friendly concrete is illustrated in **Figure 4-36**. According to literature, the addition of fly ash in concrete usually enhances the particle packing of the mixture due to the spherical shape of the fly ash particles which then produce concrete with a compact microstructure. This corresponds with the results obtained in this study as the addition of fly ash improved the OPI and reduced the porosity of concrete, compared to the pure Portland cement concrete. The results indicate that the replacement percentage of cement with fly ash insignificantly influenced the potential concrete durability, as fairly similar OPI and porosity values were obtained for the blended cement concretes. The addition of superplasticiser together with reduced water and cement content, increased the potential concrete durability of the concrete compared to the dolomite concrete (FA0), especially when considering the porosity of the concrete. Mixtures containing superplasticiser (SP) as well as fly ash (FA20, FA35, FA50) produced excellent durability concrete when considering the suggested durability classification for porosity values by Moore et al. (2021).

The use of RCA as an aggregate source had an insignificant effect on the measured OPI and porosity compared to the dolomite concrete (FA0). According to the suggested durability classification for porosity values (Moore et al., 2021), the dolomite concrete (FA0) as well as the concrete containing RCA produced poor to very poor durability concrete. The combined concrete (RCAF) measured an OPI similar to the blended cement concretes. Whereas the porosity measured for the combined concrete (RCAF) was higher than the blended cement concretes as well as the concrete containing superplasticiser (SP). All the concrete mixtures were classified as excellent durability concrete when considering the suggested durability classification for OPI values by Alexander et al. (1999). The results illustrates that environmentally friendly concrete produced by utilising waste materials, can be sufficiently durable.

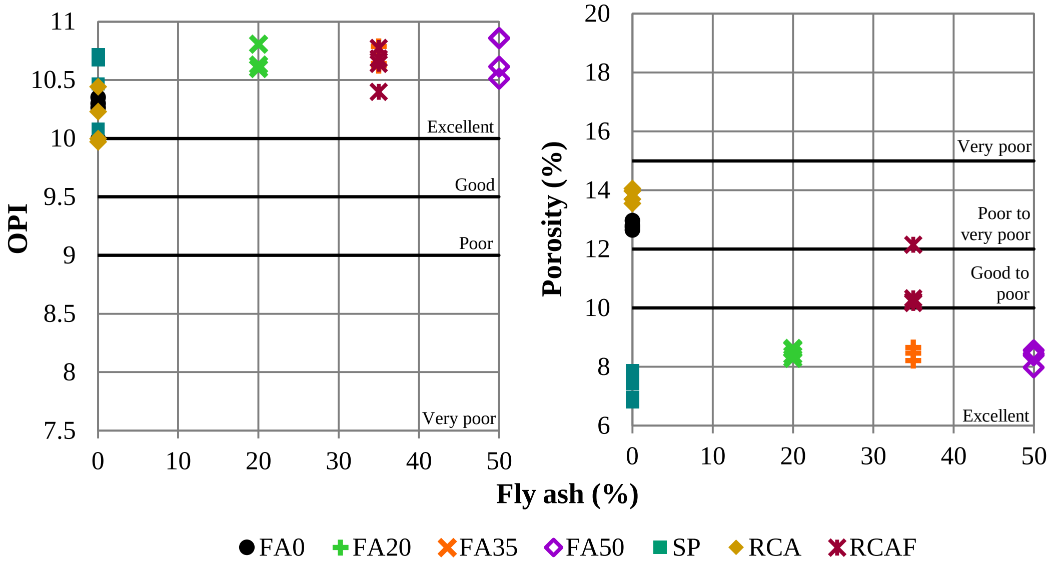

Figure 4-36: Potential durability of environmentally friendly concrete

4.10 SUMMARY

Based on the experimental study, structural concrete can be manufactured with water/cement ratios ranging from 0.3 – 0.7. Corresponding with literature, lower water/cement ratios produce concrete with increased maximum hydration temperature, strength, modulus of elasticity and drying shrinkage. Lower workability can however be expected.

The experimental study highlights the importance of proper aggregate selection when designing concrete structures. Aggregate type has a negligible effect on the strength of concrete for mixtures with equal water/cement ratio and water content. Factors such as water/cement ratio as well as cement type had a much greater effect on the strength. No simple relationship between tensile strength of concrete and its compressive strength could be obtained. Furthermore, aggregate type had a notable effect on the heat development of freshly cast concrete as well as the deformation behaviour of the hardened concrete which included:

- elastic modulus of concrete,
- coefficient of thermal expansion (CTE), and
- drying shrinkage.

Regardless of the aggregate type, higher slump is achieved for coarse to total aggregate fractions in the range of 45% to 60%, which is within a normal mix proportion range. A higher coarse to total aggregate ratio produce concrete with increased modulus of elasticity. Reduced splitting tensile strength, drying shrinkage and potential concrete durability can however be expected. The fineness of the aggregate or the coarse aggregate content have little effect on the heat development of freshly cast concrete. Furthermore, coarse to total aggregate ratio had an insignificant effect on the compressive strength as well as the CTE.

Based on the experimental study, reducing the cement content of mixtures by using blended cements resulted in a reduction in the maximum temperatures reached during cement hydration, which could be beneficial in mass concrete casts. As expected, the slump substantially increased as the fly ash content of the blended cement mixtures increased. Furthermore, utilising blended cements had an insignificant effect on the modulus of elasticity, CTE and drying shrinkage of concrete compared to the pure Portland cement concrete while a reduction in the compressive strength was observed. The strength reduction could be limited by reducing the water content of the mixtures to maintain similar workability, which will further contribute to reducing the environmental footprint. In addition, by reducing the water content, a decrease in the measured drying shrinkage could be obtained. The experimental study concluded that structural concrete can be produced with blended cements where up to 50% of the Portland cement is replaced with fly ash. The water content of the concrete and hence the cement content can be reduced further by incorporating admixtures. Consequently, the heat development of freshly cast concrete can be reduced. The addition of superplasticiser, with a reduced water and cement content, can substantially improve the strength, modulus of elasticity, drying shrinkage as well as the potential durability of concrete.

The experimental study proofed that recycled aggregate concrete (RAC) can compete with concrete made with aggregates from conventional quarries. RAC produced compressive strength greater than 30 MPa, even after only seven-days of water curing, indicating that structural concrete can easily be manufactured where 100% coarse aggregate and 25% fine aggregate is replaced with RCA. Even though RCA produce concrete with lower modulus of elasticity, it still falls within the acceptable range of natural aggregate concrete. RAC have the following properties in comparison to natural aggregate concrete:

- similar slump,
- reduced heat of hydration,
- lower density,
- reduced compressive strength,
- similar splitting tensile strength,

- lower modulus of elasticity,
- increased CTE,
- increased drying shrinkage, and
- equal potential durability.

5 ENVIRONMENTAL IMPACT ASSESSMENT

The chapter highlights the importance of not only utilising waste materials such as fly ash and recycled aggregate but also limiting the cement content in concrete mixtures to ensure sustainable construction. Embodied Energy (EE) and Embodied Carbon (EC) content relate to the amount of raw materials required, transported and processed as well as the extraction and preparation processes. The values are based on the production of the material and is defined by Soutsos & Domone (2018) as follows:

- Embodied energy: "*The amount of energy required to mine, collect, crush, refine, extract, synthesise and process the materials into the form that we can use.*"
- Embodied carbon: "*The amount of carbon dioxide emitted during the above processes but taking into account the source of the energy and its impact on the environment.*"

The values for EE and EC of concrete results from those of its constituent materials and mixture design. Typical EE and EC values of materials are given in **Table 5-1** (Hammond & Jones, 2011). The cost per kilogram of each material is also indicated in the **Table 5-1**. It is important to note that costs will differ significantly around the county depending on the cement, fly ash, superplasticiser and aggregate source as well as the transport distances. The United Kingdom imposed a tax levy in 1996 on waste disposed to landfill. In 2008, a levy equivalent to R58.30/tonne (£2.50/tonne) applied to inert waste disposal. This led to an increase in recycling and repurposing of waste materials such as fly ash and recycled aggregate (Soutsos & Domone, 2018). The cost for fly ash was taken as zero in this study, seeing that it is an unclassified waste product. According to Soutsos & Domone (2018), EE, EC and cost values for RCA may be slightly higher than that of natural aggregates and they state that there is no advantage in using RCA to reduce carbon footprint. As mentioned in literature, the true environmental footprint of RCA production is not considered by environmental impact assessment studies as the use of recycled materials, from construction and demolition, not only act as a resource-saving alternative but also lower the amount of waste material going to landfills. Nevertheless, the EE, EC and cost for recycled aggregates was assumed to be the same as that of natural aggregates.

Typical values as given in **Table 5-1** were used to determine the environmental impact, calculated per cubic meter of concrete, of the various mixtures as shown in **Figure 5-1** and **Figure 5-2**. Based on the results, the use of RCA had an insignificant effect on the EE/m^3 as well as the EC/m^3. However, as stated this does not account for the other advantages such as avoiding further extraction of natural aggregates and reducing the amount of waste material going to landfills. Furthermore, the addition of blended cements and reducing the water and cement content by utilising superplasticiser can further limit the environmental impact of concrete.

Table 5-1: Typical EE and EC values (Hammond & Jones, 2011)

	EE *(MJ/kg)*	**EC** *(kgCO$_2$/kg)*	**Cost** (R/kg)
Water	0.1	0.001	0.02
Cement	5.5	0.93	1.78
Fly Ash	0.10	0.008	-
Aggregate	0.083	0.0048	0.24
Superplasticiser	16	1.9	1.66

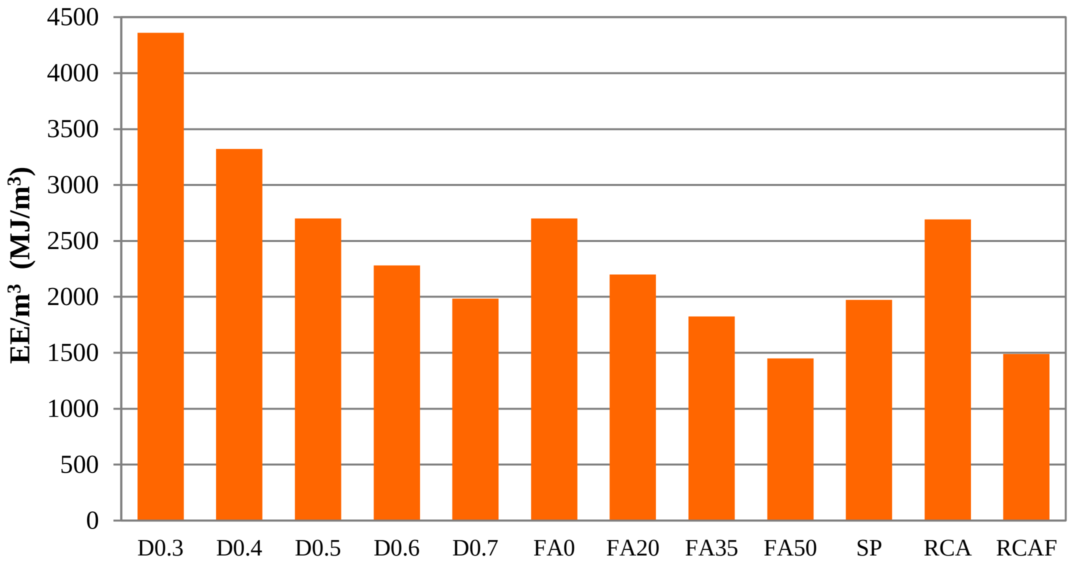

Figure 5-1: Embodied energy per cubic meter of concrete

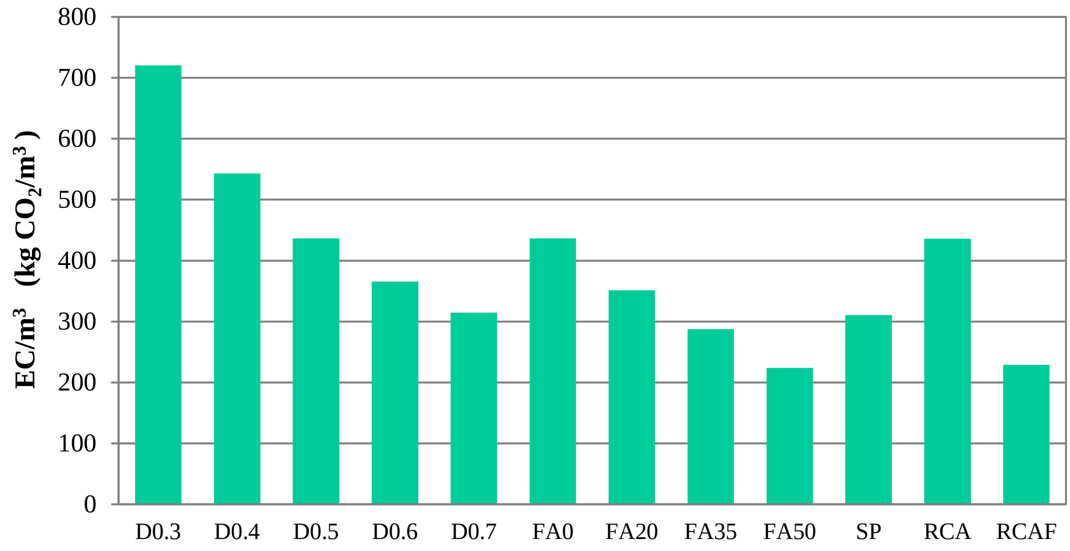

Figure 5-2: Embodied carbon per cubic meter of concrete

As concluded in Chapter 4.4, concrete constituents and mix proportions have a notable effect on the strength of concrete. Consequently, the embodied energy and carbon as a fraction of the compressive strength was also considered.

The embodied energy and carbon per unit strength of each concrete mixture is shown in **Figure 5-3** and **Figure 5-4** respectively. The effect of water/cement ratio on the embodied energy and carbon illustrate an interesting trend. Concrete with a water/cement ratio of 0.5 had the lowest values for both the EE and EC per unit strength. Concrete with a water/cement ratio of 0.3 was the least environmentally friendly mixture even though it produced substantially higher twenty-eight-day compressive strengths. The advantage of using blended cements on the embodied energy and carbon is also illustrated in the figures. When considering the EE and EC per twenty-eight-day compressive strength, the environmental impact of the pure Portland cement concrete (FA0) was further improved by replacing cement with fly ash. The embodied carbon per unit compressive strength reduced to only 4.6 $kg\ CO_2/m^3/MPa$, when reducing the water content by using superplasticisers. The influence of RCA on the embodied energy and carbon is also illustrated in the figure. Higher EE and EC per twenty-eight-day compressive strength values were obtained compared to the dolomite concrete (FA0). The combined concrete (RCAF) has substantially lower EE and EC per twenty-eight-day compressive strength values compared to the other concrete mixtures. The results obtained highlights the necessity to reduce water content, hence cement content of concrete mixtures by utilising admixtures. Additionally, using waste materials such as fly ash and RCA can further contribute to improve the sustainability of future construction.

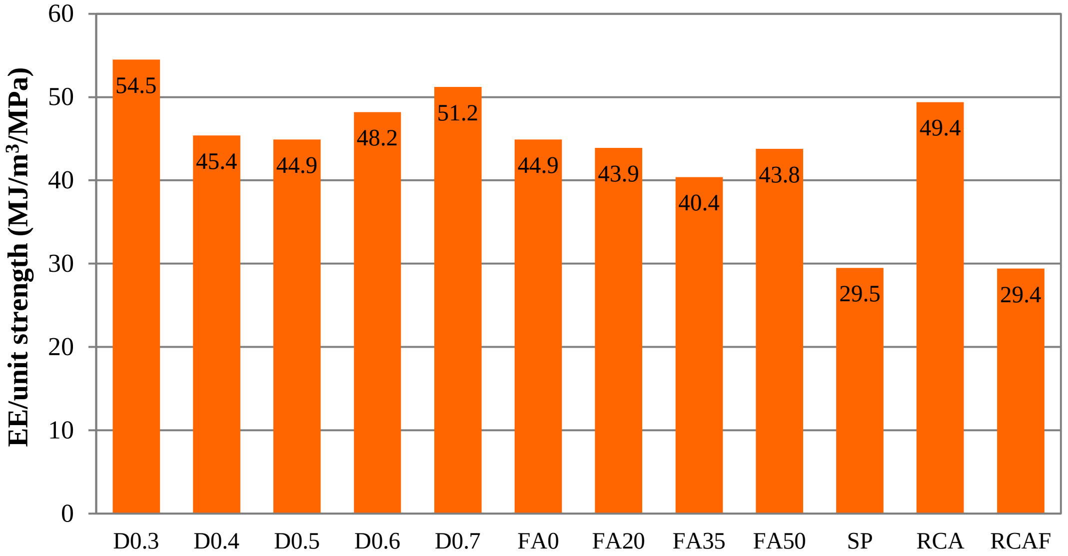

Figure 5-3: Embodied energy per unit compressive strength

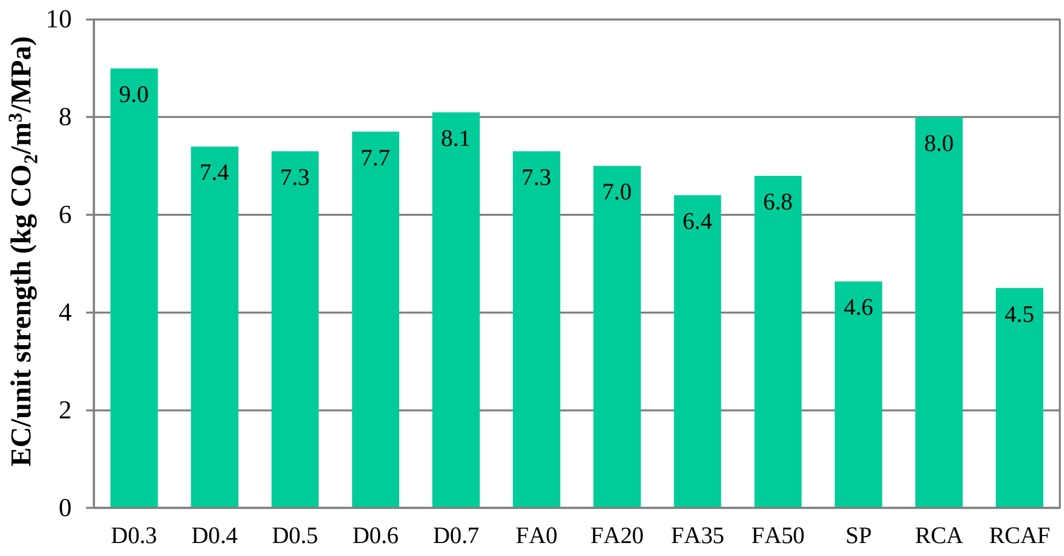

Figure 5-4: Embodied carbon per unit compressive strength

The cost per cubic meter of concrete for each mixture is illustrated in **Figure 5-5.** The results follow fairly similar trends to that obtained with the environmental impact calculated per cubic meter of concrete.

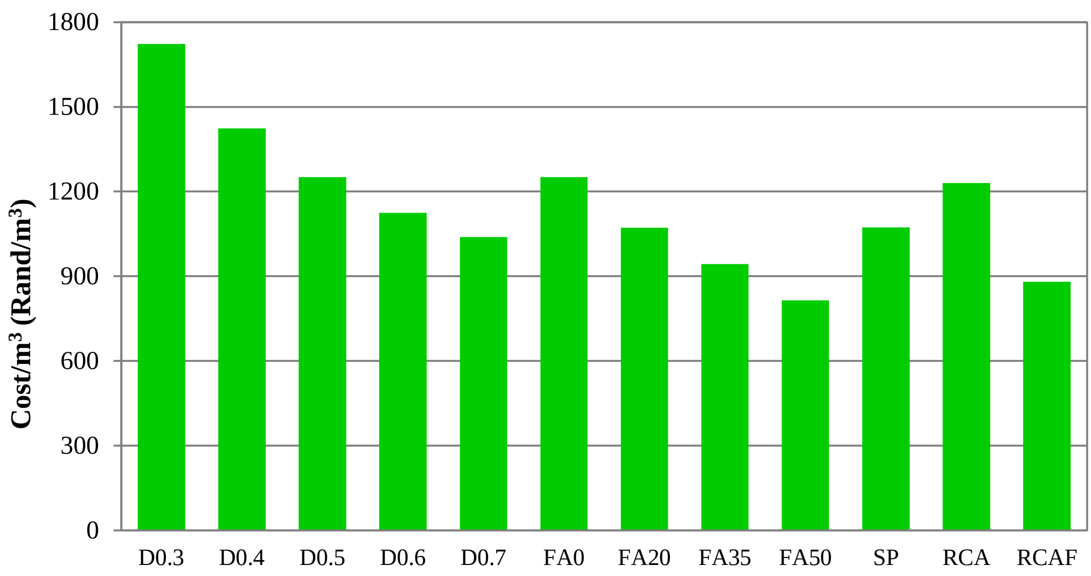

Figure 5-5: Cost per cubic meter of concrete

The cost per unit compressive strength for each mixture is illustrated in **Figure 5-6.** The results follow fairly similar trends to that obtained with the EE and EC per unit strength. The cost per unit strength for the concrete mixture containing pure Portland cement, superplasticiser and dolomite aggregate (SP) was substantially less than that of the other mixtures. Consequently, reducing the water content, hence cement content of concrete can greatly reduce the cost. This can be attributed to the high cost of cement. The cost for acquiring RCA was assumed to be the same as normal aggregates and the cost per unit strength for the RAC slightly increased compared to the dolomite concrete (FA0). An economical RAC can however be obtained by using blended cements together with superplasticiser to reduced water and cement content.

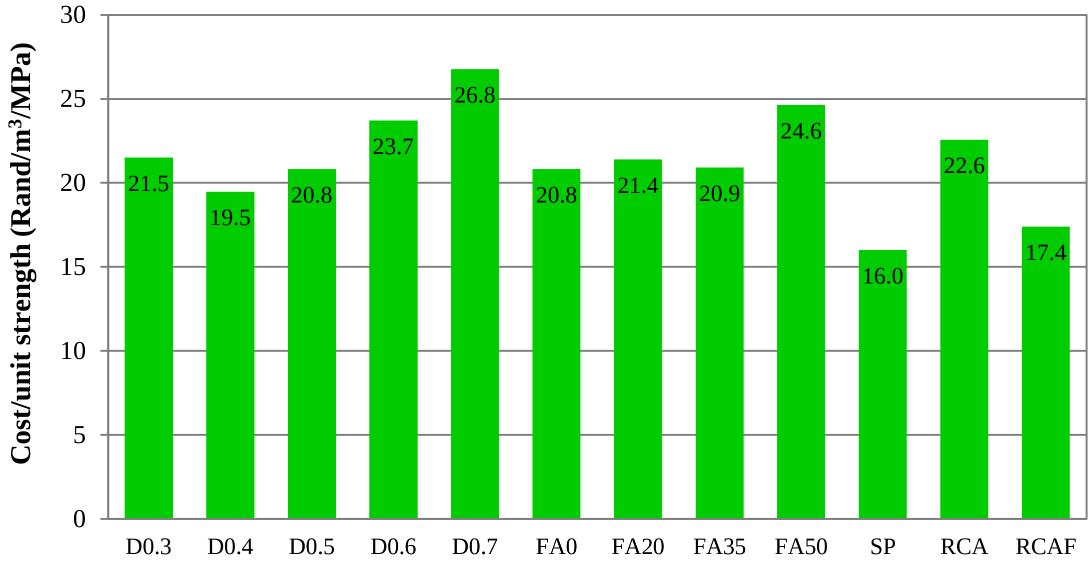

Figure 5-6: Cost per unit compressive strength

In conclusion, limiting the cement content in a concrete mixture is of great importance to ensure cost effective and environmentally friendly construction. Using blended cements in concrete can reduce the environmental footprint and lower the amount of waste material going to landfills. The advantage of recycling is not taken into account by environmental impact assessment studies as the use of recycled materials, from construction and demolition, not only act as a resource-saving alternative but also reduce the burden on to landfills. Furthermore, the use of recycled materials promotes a circular economy in the construction industry.

6 ELEVATED TEMPERATURES

6.1 INTRODUCTION

The behaviour of conventional concrete made with readily available South African materials, when exposed to elevated temperature conditions was studied. The work served as a reference to the main investigation on the behaviour of environmentally friendly concrete, i.e., concrete containing recycled aggregates, admixtures as well as blended cements, when subjected to elevated temperatures. This chapter provides an in-depth examination on the properties of conventional and environmentally friendly concrete after exposure to elevated temperatures such as compressive strength, splitting tensile strength, modulus of elasticity, mass loss and potential concrete durability.

6.2 COMPRESSIVE STRENGTH

The reduction in compressive strength of concrete exposed to elevated temperatures is discussed in this section. The influence of two different testing conditions were also considered, i.e., testing the specimen saturated according to standard test methods or testing the specimen in a dry state.

The effect of early age temperature exposure (7-days after casting) on the saturated compressive strength is illustrated in **Figure 6-1 a)**. Corresponding with literature, the relative compressive strength results indicate superior behaviour when concrete was exposed to elevated temperature at an early age compared to the performance after twenty-eight days of water curing. However, when considering the actual compressive strength at the various exposure temperatures, an insignificant difference was obtained. The results indicate a gradual loss in concrete compressive strength up to 350°*C*. However, strength loss was recovered at 500°*C* and according to literature this phenomenon is known as autogenous recovery. The effect of early age temperature exposure on the dry compressive strength is illustrated in **Figure 6-1 b)**. The same trend as with the saturated testing conditions was observed, except that the compressive strength gradually decreased up to 500°*C* with no autogenous recovery. Autogenous strength recovery only took place with the specimen placed back in water and tested saturated. According to literature, portlandite decomposes at approximately 500°*C* and with the reintroduction of water after temperature exposure, the decomposed portlandite may rehydrate, causing a recovery in compressive strength.

As mentioned, the inclusion of polypropylene fibres in concrete mixtures can prevent explosive spalling as they provide pressure relief channels after melting. The effect of polypropylene fibres on the residual properties was therefore of interest and can be seen in both figures. The inclusion of fibres did not improve the residual compressive strength and equivalent results to the concrete without fibres were obtained. From the results, it can be concluded that the addition of polypropylene fibres can prevent explosive spalling but will not improve strength properties after exposure to high temperatures.

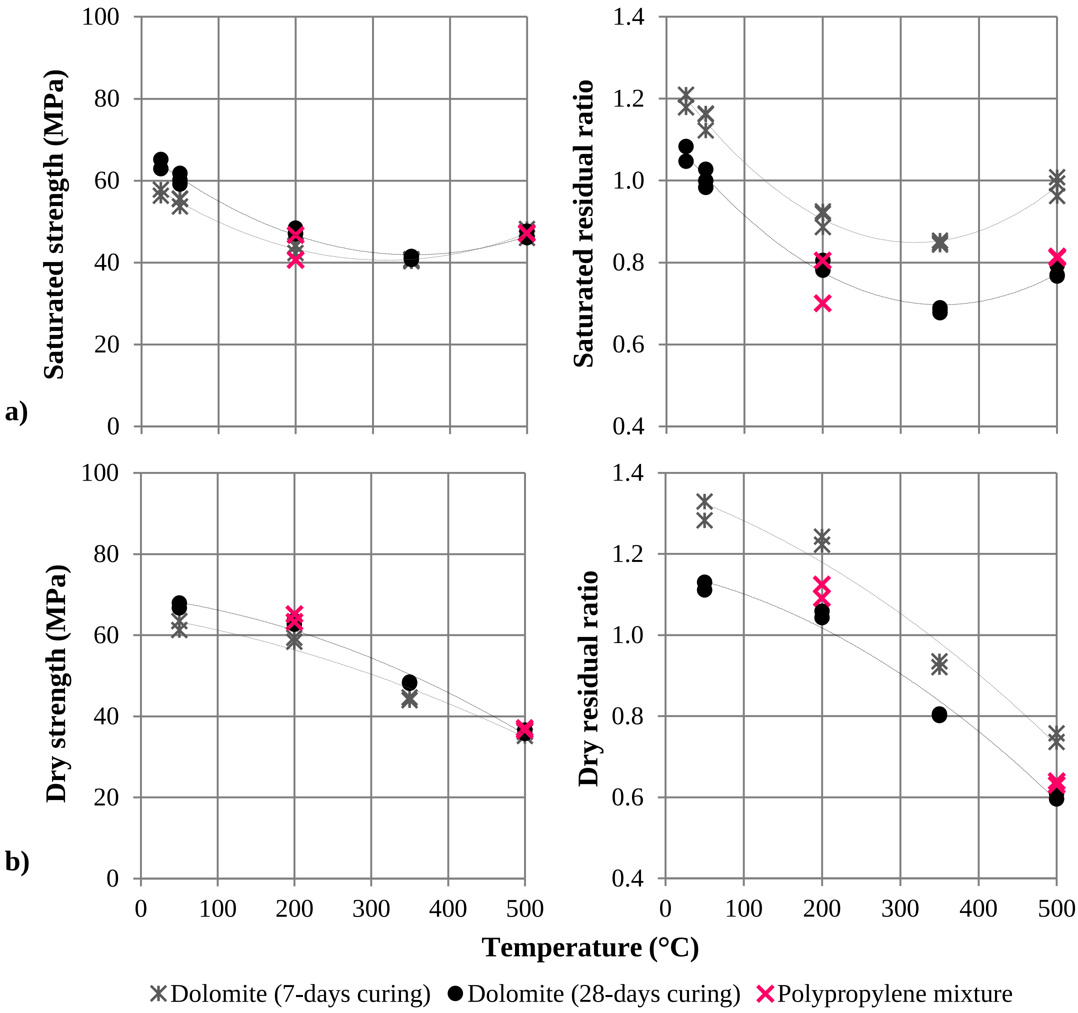

Figure 6-1: Effect of temperature on concrete compressive strength

As mentioned in the literature review, the influence of water/cement or water/binder ratio on the properties of concrete exposed to elevated temperatures are more consistent throughout research. Corresponding to literature, the use of lower water/cement ratios lead to reduced performance of concrete at elevated temperatures as shown in **Figure 6-2 a)**. The concrete with a water/cement ratio of 0.3 showed rapid strength loss and the deterioration of strength was to such an extent that no autogenous recovery was observed. Concrete with a water/cement ratio of 0.6 and 0.7 showed substantial autogenous recovery at 500°C, with strengths higher than the twenty-eight-day water cured strengths. The initial concrete strength of all the mixtures were vastly different, however the variation in the results when considering the actual strength after exposure to 500°C was relatively small and all the strengths were between 40 MPa and 50 MPa. The effect of elevated temperature on the dry compressive strength is illustrated in **Figure 6-2 b)**. The compressive strength gradually decreased up to 500°C and no autogenous recovery was observed with specimen tested in a dry state.

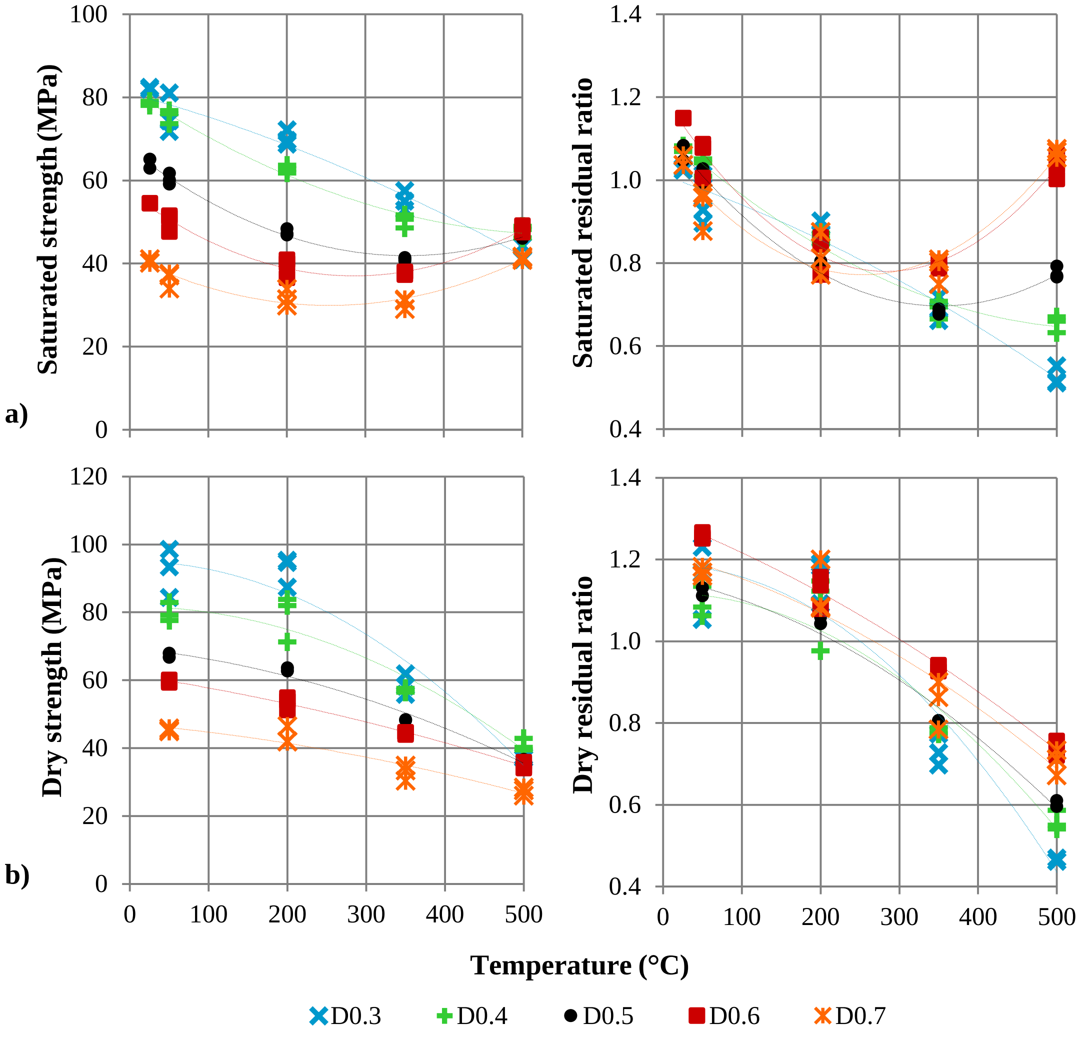

Figure 6-2: Effect of w/c ratio on concrete strength as a function of temperature

The effect of elevated temperature on the saturated compressive strength of concrete with varying coarse to total aggregate ratios is illustrated in **Figure 6-3 a)**. The results indicate a gradual loss in concrete compressive strength up to 350°C and autogenous strength recovery at temperatures of 500°C. Concrete with no coarse aggregate (D0) showed substantial autogenous recovery at 500°C, with strengths similar to the initial twenty-eight-day water cured strengths. The initial concrete strengths were fairly similar, however when considering the actual strength after exposure to 500°*C*, substantial variation in strength can be observed. Mortars have more uniform or homogeneous microstructures which might lead to less internal damage when subjected to elevated temperature. It can be concluded that coarse aggregate content has a substantial effect on the residual compressive strength of concrete exposed to elevated temperatures. The effect of elevated temperature on the dry compressive strength is illustrated in **Figure 6-3 b)**. The same trend can be observed as with saturated testing conditions, except that the compressive strength gradually decreased up to 500°*C*. Some of the specimen were tested after exposure to 460°*C*, which showed that there was a slight difference in strength when compared to the specimen tested after exposure to 500°*C*. The results indicate a very small amount of autogenous recovery at 500°*C* for both the mixture with 0% and 75% coarse to total aggregate ratio. This was not visible in the previous results as none of the tests were performed at 460°*C*. According to literature the loss of strength due to elevated temperatures tend to recover over time, not only with wetting, but also with storage at normal relative humidity, which correlates with the results obtained.

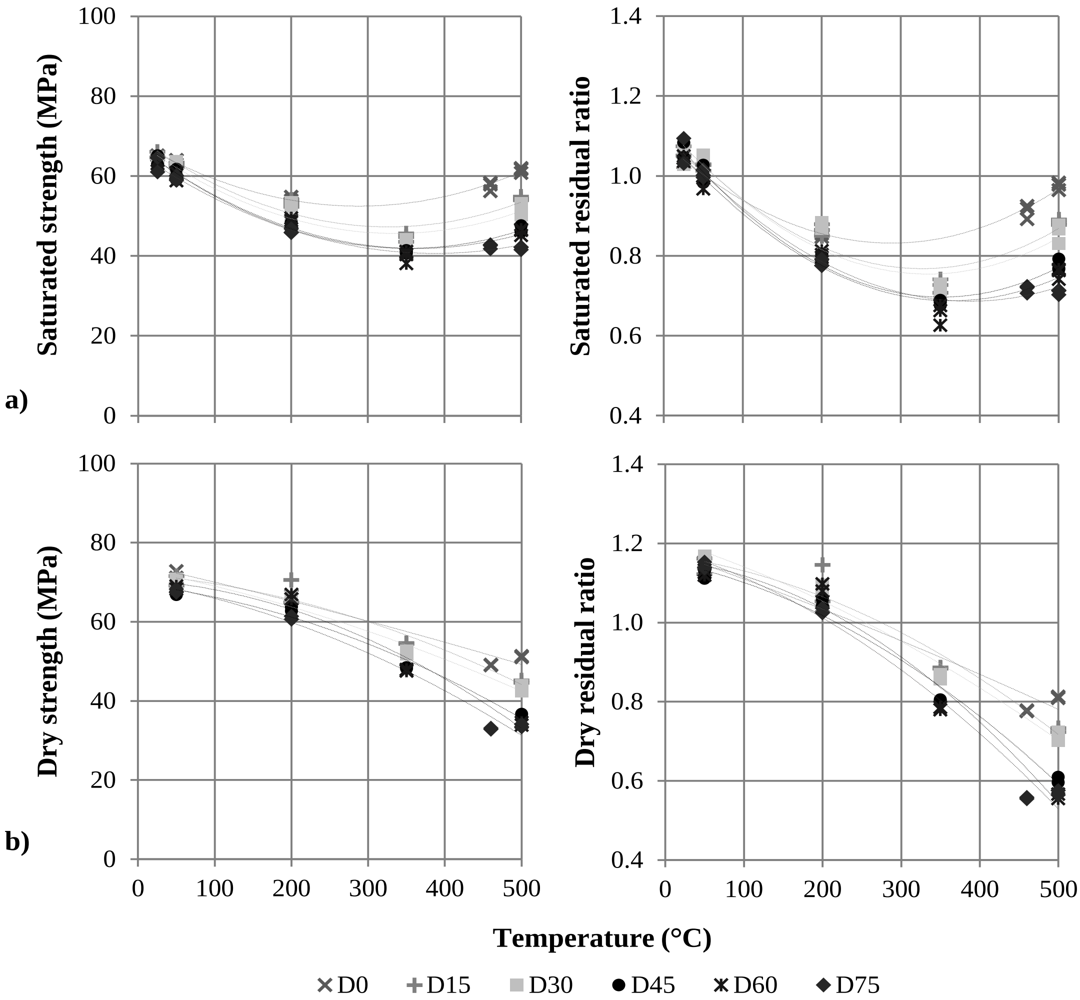

Figure 6-3: Effect of sand content on concrete strength as a function of temperature

The effect of elevated temperature on the saturated and dry compressive strength of concrete made using different aggregate types is illustrated in **Figure 6-4**. As mentioned in literature, the traditional order of preference of aggregate type for concrete exposed to elevated temperatures is calcareous (limestone, dolomite), felspathic (andesite, basalt, dolerite, gabbro), granites and then siliceous (quartz, quartzite). However, other research suggested that felspathic and granite aggregates should be given preference. The results obtained in this study tend to correspond with the latter, especially when considering the dry compressive strength results. andesite, dolerite and tillite performed better compared to felsite, granite and dolomite. When considering the saturated compressive strength, dolerite and tillite still outperformed the other aggregate types, while dolomite aggregate produced the lowest compressive strengths at all temperature conditions.

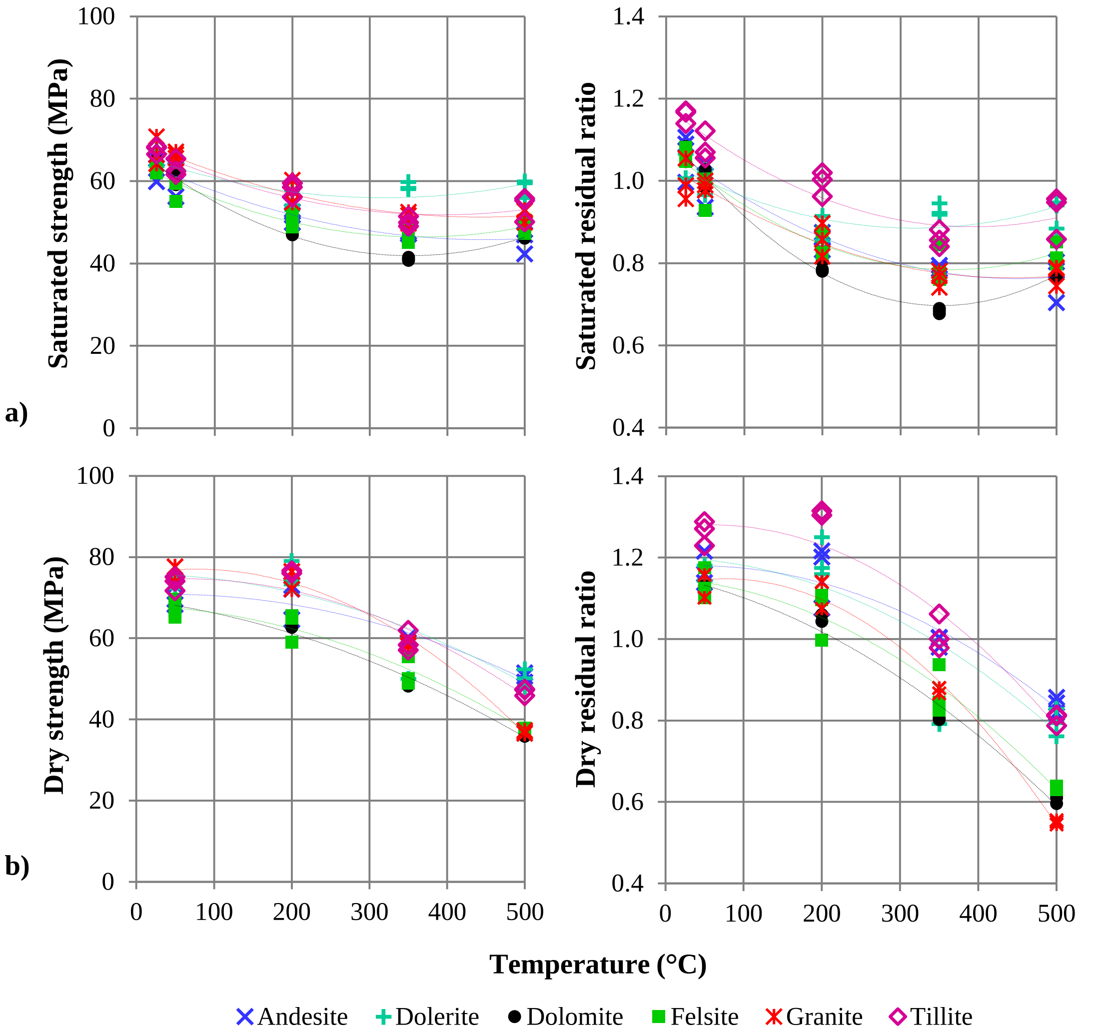

Figure 6-4: Effect of aggregate type on concrete strength as a function of temperature

According to literature the addition of fly ash in concrete increase its resistance to elevated temperature. However, contradicting results were obtained in this study, as illustrated in **Figure 6-5 a)**, when specimen were tested saturated. The concrete containing fly ash showed higher strength deterioration compared to the reference concrete (FA0). The disruptive effects of the cement paste shrinkage opposed by aggregate expansion will be greater for concrete with a compact microstructure. The addition of SCMs in concrete mixtures usually enhance particle packing which produce concrete with a compact microstructure that lead to rapid deterioration of strength at elevated temperatures. Further experimental work will however be required to establish the exact reason for the unexpected trend. The fly ash concrete as well as the pure Portland cement concrete showed autogenous recovery at 500*°C*.

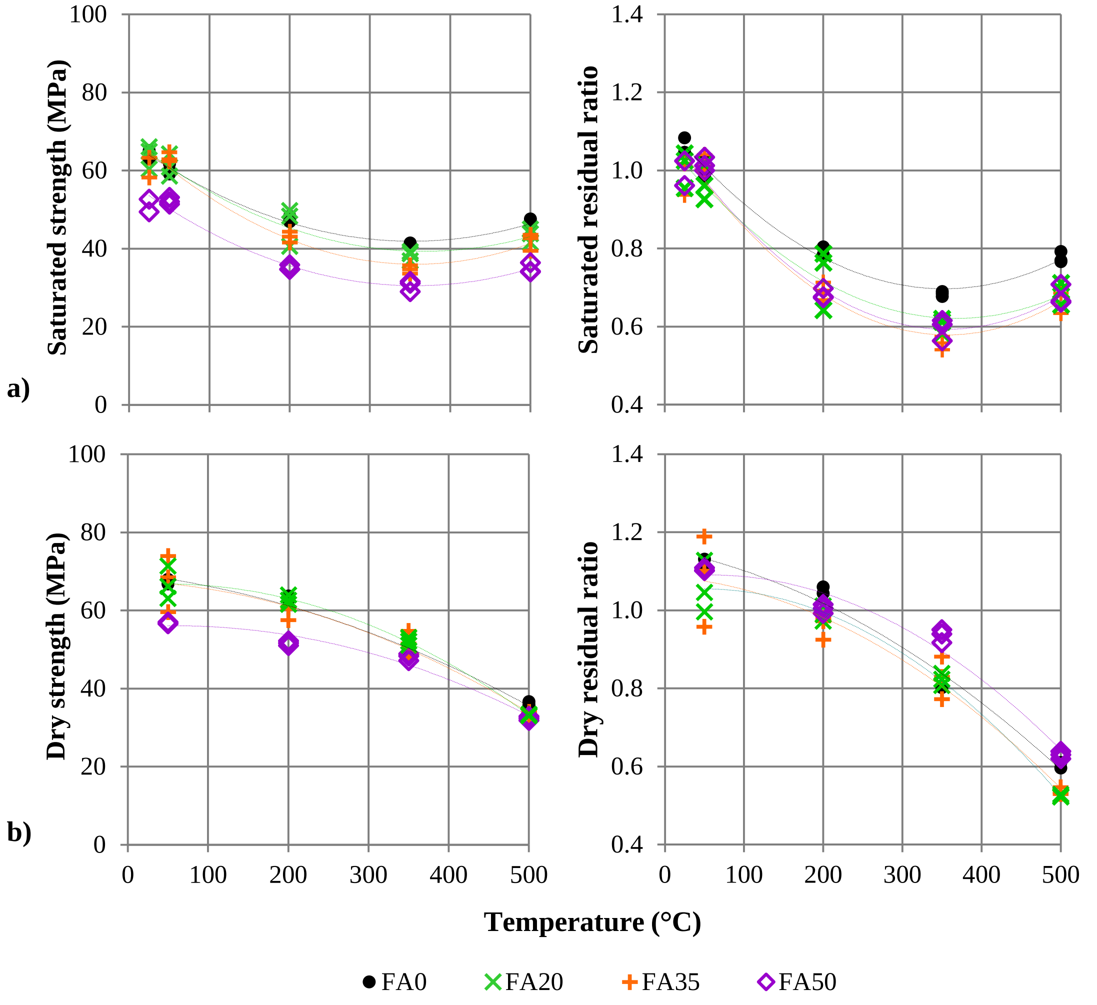

Figure 6-5: Effect of fly ash on concrete strength as a function of temperature

The effect of elevated temperature on the dry compressive strength is illustrated in **Figure 6-5 b)**. The compressive strength gradually decreases up to 500*°C* and no autogenous recovery was observed with specimen tested in a dry state. When considering the actual strength after temperature exposure little difference can be observed between the different concrete mixtures, except for the concrete containing 50% fly ash, which had slightly lower strength at 50°C and 200°C. The relative compressive strength indicates that concrete containing 20% and 35% fly ash (FA20 and FA35) produce lower strengths relative to the ninety-eight days water cured strength. However, the concrete with 50% fly ash (FA50) correlate with previous literature, as the relative dry compressive strength was higher than that of the reference concrete (FA0).

The reduction in compressive strength of environmentally friendly concrete exposed to elevated temperatures is illustrated in **Figure 6-6**. The effect of elevated temperature exposure on the saturated compressive strength is illustrated in **Figure 6-6 a)**, while the effect of the dry compressive strength is illustrated in in **Figure 6-6 b)**. The results indicate a gradual loss in saturated concrete compressive strength up to 350°C. Autogenous recovery was observed at 500°C for all the concrete mixtures tested saturated. The same trend for the dry compressive strength results can be observed as with saturated testing conditions, except that the compressive strength gradually decreased up to 500°C and no autogenous recovery was observed with specimen tested in a dry state. The only exception was with the concrete containing superplasticiser that showed a rapid increase in dry compressive strength up to 200°C and then only decreased. The concrete containing pure Portland cement, superplasticiser and dolomite aggregate (SP) outperformed the other concrete mixtures, retaining 80% of its initial twenty-eight-day water cured strength after being subjected to 500°C, while the other concrete mixtures only retained 60%. The results obtained in this study present contradicting trends to previously published literature that found that the addition of superplasticiser had little effect in the performance of concrete exposed to elevated temperatures. The improved performance might be attributed to reduced cement and water content. Moreover, the addition of superplasticiser produces a more uniform or homogeneous microstructure (Boshoff et al., 2021) which might lead to improved behaviour at elevated temperatures. Further research is required to determine the influence of superplasticiser on the performance of concrete exposed to elevated temperatures.

According to literature, studies show opposing results when considering RAC after exposure to elevated temperatures. Furthermore, the variability of the source and nature of the RCA makes it extremely challenging to compare results with previously published literature. Some experimental studies found that the residual mechanical properties of the RAC subjected to elevated temperatures are approximately the same as that of conventional concrete, while other studies either concluded that RAC subjected to elevated temperatures performed better relative to natural aggregate concrete or that the performance of RAC diminished after exposure to elevated temperature. The results obtained in this study tend to correspond with the former. The compressive strength at all the temperature conditions were slightly lower for the concrete containing RCA compared to the dolomite concrete (FA0) after exposure to elevated temperature. However, when considering the residual compressive strength results, an insignificant difference was observed between the RAC and dolomite concrete after exposure to elevated temperature. Literature further state that the addition of fly ash may significantly contribute to improve the behaviour of RAC at elevated temperatures. The combined concrete (RCAF) showed equivalent or improved performance compared to the concrete containing RCA for all temperature and testing conditions. Considering the previous results from this study, the addition of superplasticiser may be the cause for the improved performance rather than the addition of fly ash. Further testing is however required to determine whether the improved performance can be attributed to the addition of fly ash or the addition of superplasticiser.

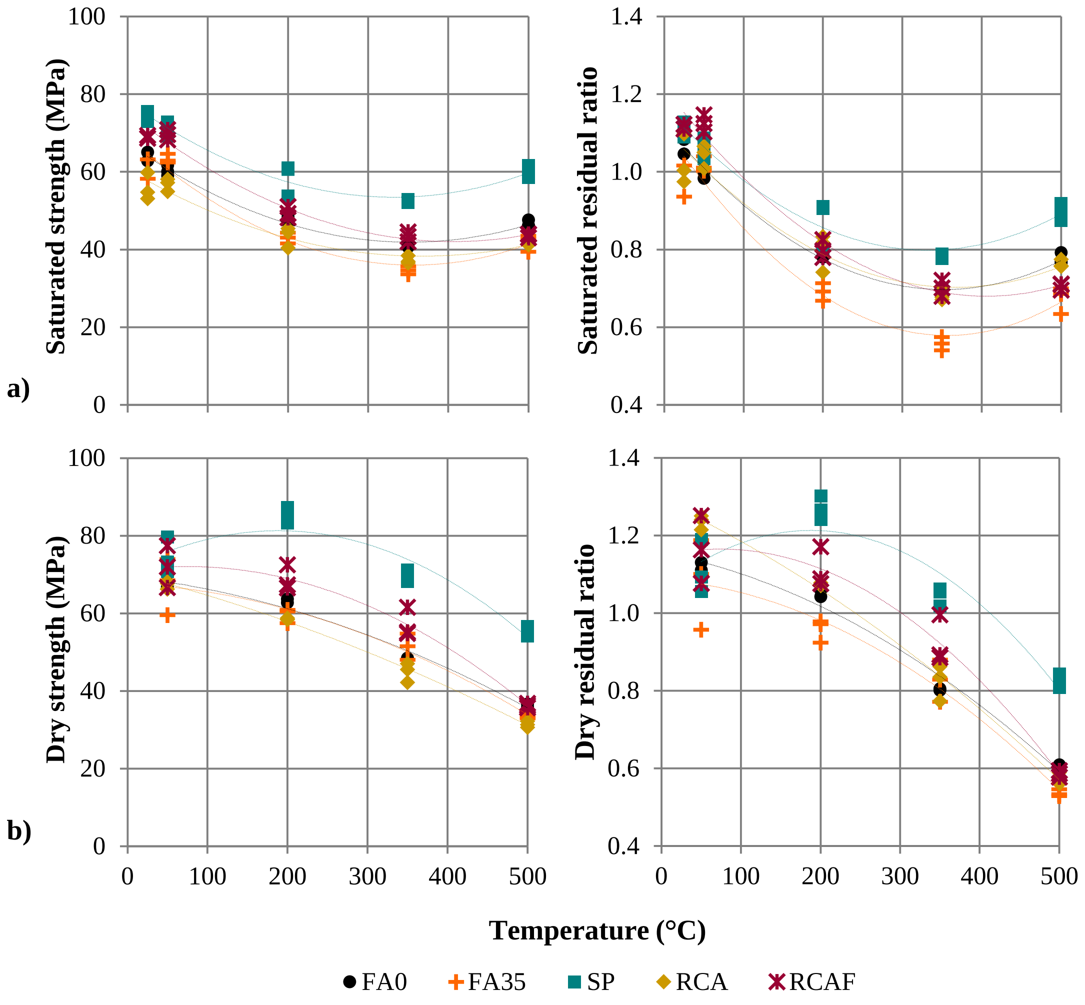

Figure 6-6: Effect of temperature on the strength of environmentally friendly concrete

6.3 SPLITTING TENSILE STRENGTH

The reduction in splitting tensile strength of concrete exposed to elevated temperatures is discussed in this section. All the specimen were tested in a saturated state, according to standard test methods.

The effect of early age temperature exposure (7-days after casting) on the splitting tensile strength is illustrated in **Figure 6-7**. A gradual loss in strength was observed up to 500°*C*. Concrete exposed to elevated temperature at an early age produced lower splitting tensile strength values at temperatures of 50°*C* and 200°*C* and slightly higher values at temperatures of 350°*C* and 500°*C*, compared to the concrete only exposed to elevated temperatures after twenty-eight days of curing. Corresponding to literature, no autogenous recovery was observed in any of the splitting tensile strength results.

As with the compressive strength results, the inclusion of fibres did not improve the residual splitting tensile strength and only a slight increase in strength at 500°*C* compared to the reference mixture was measured. The results reinforce the statement previously made, that the addition of polypropylene fibres might prevent explosive spalling but will not improve strength properties after exposure to high temperatures.

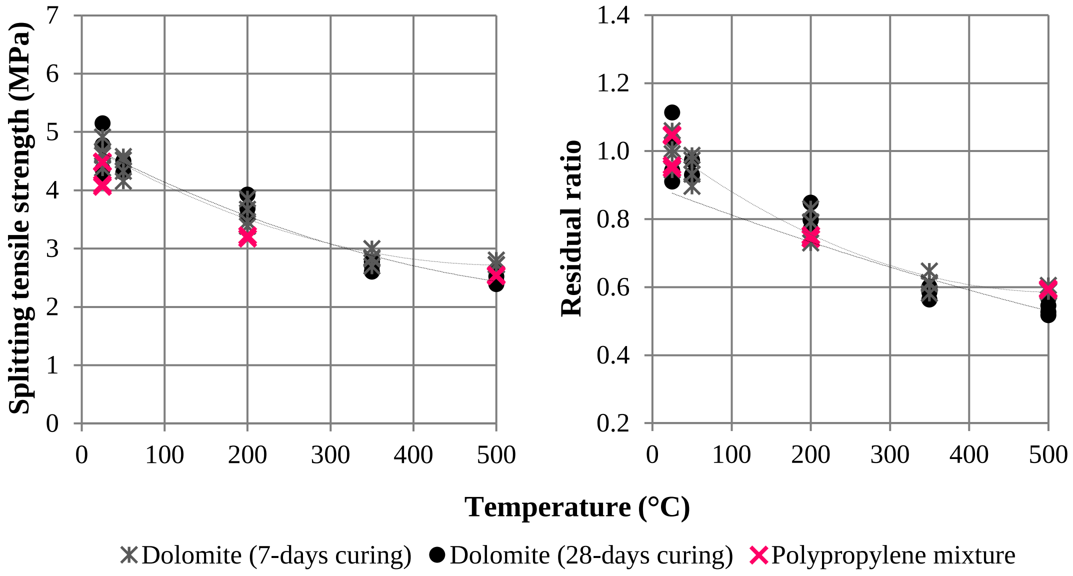

Figure 6-7: Effect of temperature on concrete splitting tensile strength

As previously stated, the use of lower water/cement ratios lead to reduced performance of concrete at elevated temperatures as shown in **Table 6-1** and **Figure 6-8**. The deterioration of strength was to such an extent that no autogenous recovery was observed for the concrete with a water/cement ratio of 0.3. On the other hand, concrete with higher water/cement ratios, 0.6

and 0.7, showed substantial autogenous recovery at 500°*C*, similar to the compressive strength results.

Table 6-1: Effect of w/c ratio on concrete tensile strength as a function of temperature

		D0.3	D0.4	D0.5	D0.6	D0.7
50°*C*	Splitting tensile strength (*MPa*)	5.16	4.76	4.40	4.27	3.97
	Standard deviation (*MPa*)	0.02	0.03	0.14	0.21	0.12
	Coefficient of variation	0.00	0.01	0.03	0.05	0.03
	Residual ratio	0.91	0.95	0.95	1.00	1.06
200°*C*	Splitting tensile strength (*MPa*)	3.09	3.42	3.72	3.41	3.11
	Standard deviation (*MPa*)	0.31	0.38	0.15	0.16	0.01
	Coefficient of variation	0.10	0.11	0.04	0.05	0.00
	Residual ratio	0.55	0.68	0.80	0.80	0.84
350°*C*	Splitting tensile strength (*MPa*)	2.44	2.60	2.72	3.23	3.09
	Standard deviation (*MPa*)	0.24	0.16	0.08	0.11	0.03
	Coefficient of variation	0.10	0.06	0.03	0.03	0.01
	Residual ratio	0.43	0.52	0.59	0.76	0.83
500°*C*	Splitting tensile strength (*MPa*)	1.32	2.56	2.49	3.48	3.24
	Standard deviation (*MPa*)	0.07	0.18	0.10	0.11	0.05
	Coefficient of variation	0.05	0.07	0.04	0.03	0.02
	Residual ratio	0.23	0.51	0.54	0.82	0.87

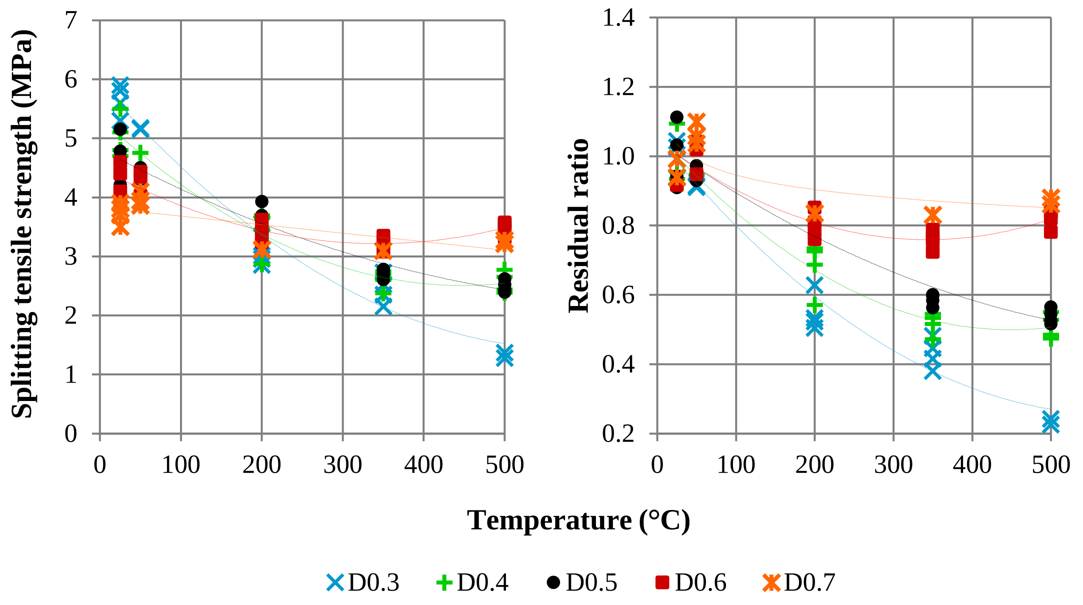

Figure 6-8: Effect of w/c ratio on concrete tensile strength as a function of temperature

Autogenous splitting tensile strength recovery was observed, even though literature state that there is little evidence available to indicate autogenous recovery in the tensile strength of concrete. The values obtained for the relative splitting tensile strength were considerably lower than the corresponding relative compressive strength values. Corresponding to literature, this indicates that tensile strength is much more affected by microcrack development in the concrete due to elevated temperatures.

The effect of elevated temperature on the splitting tensile strength of concrete with varying coarse to total aggregate ratios is tabulated in **Table 6-2** and illustrated in **Figure 6-9**. The results indicate similar trends to the compressive strength results. The splitting tensile strength gradually reduced up to 350°*C*, whereafter concrete containing higher sand contents showed autogenous strength recovery after exposure to 500°*C* (D0, D15, D30). The initial splitting tensile strength of concrete with varying coarse to total aggregate ratios were fairly similar, however the variation in results increased as the exposure temperature increased. This indicates that coarse aggregate content has a notable effect on the splitting tensile strength of concrete exposed to elevated temperatures.

Table 6-2: Effect of sand content on concrete tensile strength as a function of temperature

		D0	D15	D30	D45	D60	D75
50°*C*	Splitting tensile strength (*MPa*)	4.94	5.10	5.42	4.40	4.49	4.32
	Standard deviation (*MPa*)	0.34	0.30	0.38	0.14	0.21	0.38
	Coefficient of variation	0.07	0.06	0.07	0.03	0.05	0.09
	Residual ratio	1.05	1.10	1.26	0.95	1.13	1.02
200°*C*	Splitting tensile strength (*MPa*)	4.07	4.02	3.93	3.72	3.45	3.11
	Standard deviation (*MPa*)	0.17	0.18	0.10	0.15	0.28	0.10
	Coefficient of variation	0.04	0.04	0.02	0.04	0.08	0.03
	Residual ratio	0.87	0.87	0.92	0.80	0.87	0.73
350°*C*	Splitting tensile strength (*MPa*)	4.03	2.98	2.97	2.72	2.51	2.18
	Standard deviation (*MPa*)	0.08	0.19	0.21	0.08	0.09	0.43
	Coefficient of variation	0.02	0.06	0.07	0.03	0.04	0.20
	Residual ratio	0.86	0.64	0.69	0.59	0.63	0.51
500°*C*	Splitting tensile strength (*MPa*)	3.94	3.28	3.08	2.49	1.88	1.27
	Standard deviation (*MPa*)	0.17	0.13	0.25	0.10	0.09	0.18
	Coefficient of variation	0.04	0.04	0.08	0.04	0.05	0.14
	Residual ratio	0.84	0.71	0.72	0.54	0.47	0.30

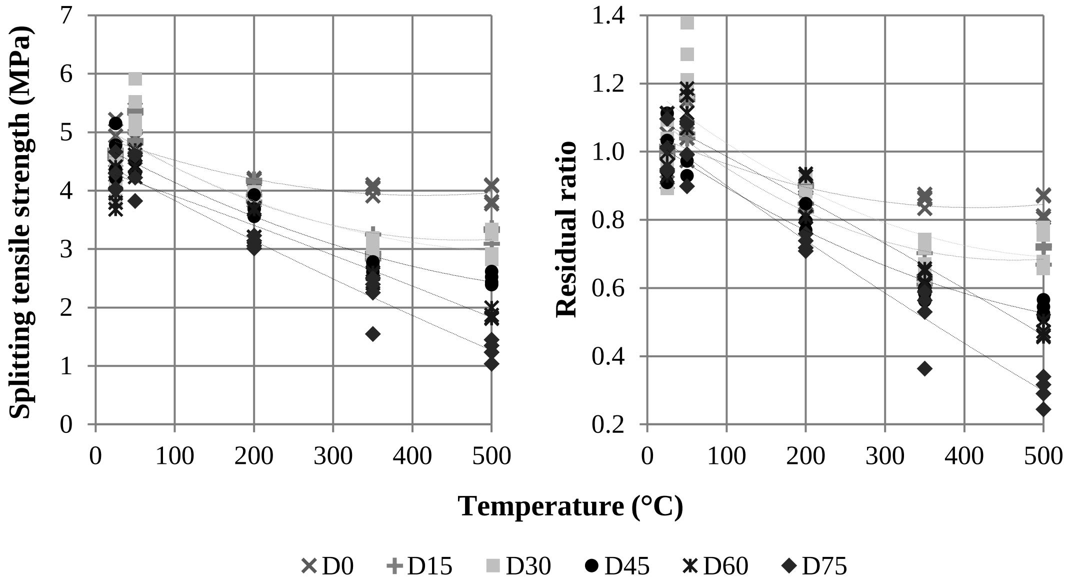

Figure 6-9: Effect of sand content on concrete tensile strength as a function of temperature

The effect elevated temperature on the splitting tensile strength of concrete made using different aggregate types is shown in **Table 6-3** and **Figure 6-10**.

Table 6-3: Effect of aggregate type on concrete tensile strength as a function of temperature

		A	**R**	**D**	**F**	**G**	**T**
50°C	Splitting tensile strength (*MPa*)	4.21	4.43	4.40	4.39	4.56	4.40
	Standard deviation (*MPa*)	0.21	0.17	0.14	0.14	0.32	0.22
	Coefficient of variation	0.05	0.04	0.03	0.03	0.07	0.05
	Residual ratio	0.96	0.96	0.95	0.98	0.96	0.99
200°C	Splitting tensile strength (*MPa*)	4.06	4.00	3.72	3.96	3.49	3.39
	Standard deviation (*MPa*)	0.21	0.57	0.15	0.22	0.24	0.45
	Coefficient of variation	0.05	0.14	0.04	0.05	0.07	0.13
	Residual ratio	0.92	0.87	0.80	0.89	0.73	0.76
350°C	Splitting tensile strength (*MPa*)	3.25	3.23	2.72	3.94	3.09	3.16
	Standard deviation (*MPa*)	0.09	0.17	0.08	0.14	0.42	0.12
	Coefficient of variation	0.03	0.05	0.03	0.03	0.14	0.04
	Residual ratio	0.74	0.70	0.59	0.88	0.65	0.71
500°C	Splitting tensile strength (*MPa*)	3.37	3.35	2.49	3.71	2.96	3.29
	Standard deviation (*MPa*)	0.10	0.24	0.10	0.10	0.07	0.21
	Coefficient of variation	0.03	0.07	0.04	0.03	0.02	0.06
	Residual ratio	0.77	0.73	0.54	0.83	0.62	0.74

The results obtained for the splitting tensile strength correspond well with the compressive strength results. Concrete containing dolomite and granite aggregate produced the lowest splitting tensile strengths at all temperature conditions. However, concrete containing felsite aggregate outperformed all aggregate types when considering the splitting tensile strength. Concrete containing andesite, dolerite and tillite showed slight autogenous recovery after exposure to 500*°C*. The initial splitting tensile strength of concrete with varying aggregate types were fairly similar, however the variation in results increased as the exposure temperature increased. This again illustrates the notable influence of aggregate type on the properties of concrete subjected to elevated temperatures.

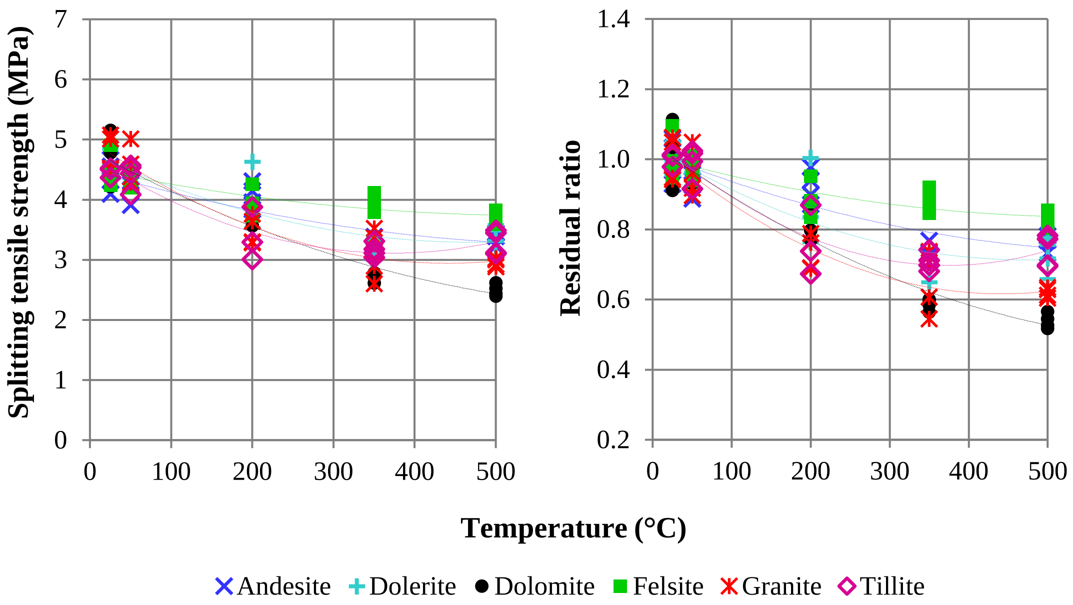

Figure 6-10: Effect of aggregate type on concrete tensile strength as a function of temperature

The effect of elevated temperatures on the splitting tensile strength of blended cement concrete is shown in **Table 6-4** and **Figure 6-11**. The splitting tensile strength gradually decreased up to 500*°C*, except for mixture with 35*%* and 50*%* fly ash replacement which showed recovery of strength after exposure to 500*°C*. The general trend indicates that splitting tensile strength decreased as the replacement percentage of cement with fly ash increased. This can again be attributed to the enhanced particle packing of the mixtures containing fly ash which produced concrete with a compact microstructure. However, the difference in splitting tensile strength was fairly small after exposure to 500*°C*. As with the compressive strength, the addition of fly ash did not substantially improve the behaviour of concrete when subjected to elevated temperatures.

Table 6-4: Effect of fly ash on concrete tensile strength as a function of temperature

		FA0	FA20	FA35	FA50
50°C	Splitting tensile strength (*MPa*)	4.40	5.26	5.52	5.17
	Standard deviation (*MPa*)	0.14	0.46	0.48	0.55
	Coefficient of variation	0.03	0.09	0.09	0.11
	Residual ratio	0.95	1.11	1.03	1.05
200°C	Splitting tensile strength (*MPa*)	3.72	3.23	3.19	2.59
	Standard deviation (*MPa*)	0.15	0.74	0.26	0.14
	Coefficient of variation	0.04	0.23	0.08	0.05
	Residual ratio	0.80	0.68	0.60	0.53
350°C	Splitting tensile strength (*MPa*)	2.72	2.79	2.15	2.38
	Standard deviation (*MPa*)	0.08	0.08	0.14	0.15
	Coefficient of variation	0.03	0.03	0.06	0.06
	Residual ratio	0.59	0.59	0.40	0.48
500°C	Splitting tensile strength (*MPa*)	2.49	2.43	2.76	2.33
	Standard deviation (*MPa*)	0.10	0.35	0.20	0.06
	Coefficient of variation	0.04	0.14	0.07	0.03
	Residual ratio	0.54	0.51	0.52	0.47

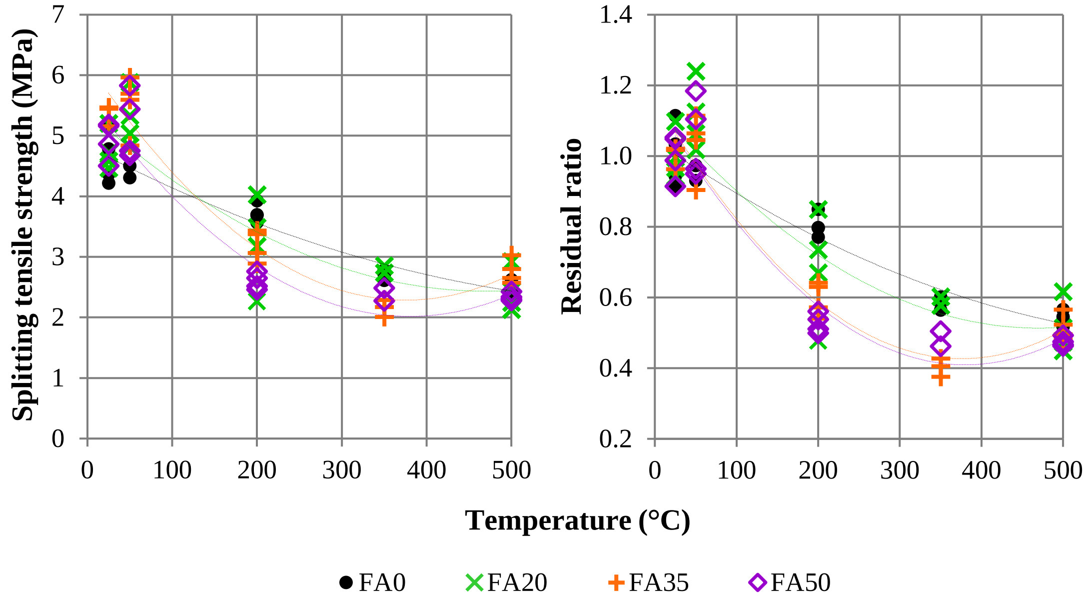

Figure 6-11: Effect of fly ash on concrete tensile strength as a function of temperature

The effect of elevated temperature exposure on the splitting tensile strength of environmentally friendly concrete is tabulated in **Table 6-5** and illustrated in **Figure 6-12**. The results indicate a gradual loss in concrete splitting tensile strength up to 500°*C*. Autogenous strength recovery was observed after exposure to 500°*C* for the concrete with 35% fly ash (FA35) as well as the concrete containing pure Portland cement, superplasticiser and dolomite aggregate (SP). As with the compressive strength results, the concrete containing superplasticiser produced the highest splitting tensile strength at elevated temperature.

Table 6-5: Effect of temperature on the tensile strength of environmentally friendly concrete

		FA0	FA35	SP	RCA	RCAF
50°*C*	Splitting tensile strength (*MPa*)	4.40	5.52	5.50	4.45	4.94
	Standard deviation (*MPa*)	0.14	0.48	0.15	0.21	0.24
	Coefficient of variation	0.03	0.09	0.03	0.05	0.05
	Residual ratio	0.95	1.03	1.13	1.02	0.96
200°*C*	Splitting tensile strength (*MPa*)	3.72	3.19	4.26	2.75	3.28
	Standard deviation (*MPa*)	0.15	0.26	0.20	0.19	0.21
	Coefficient of variation	0.04	0.08	0.05	0.07	0.07
	Residual ratio	0.80	0.60	0.88	0.63	0.64
350°*C*	Splitting tensile strength (*MPa*)	2.72	2.15	3.24	2.15	2.61
	Standard deviation (*MPa*)	0.08	0.14	0.21	0.20	0.15
	Coefficient of variation	0.03	0.06	0.07	0.09	0.06
	Residual ratio	0.59	0.40	0.67	0.49	0.51
500°*C*	Splitting tensile strength (*MPa*)	2.49	2.76	3.32	1.96	2.43
	Standard deviation (*MPa*)	0.10	0.20	0.19	0.10	0.16
	Coefficient of variation	0.04	0.07	0.06	0.05	0.06
	Residual ratio	0.54	0.52	0.68	0.45	0.47

As mentioned previously, some studied concluded that RAC subjected to elevated temperatures may perform better, equal to or worse relative to natural aggregate concrete. RAC subjected to elevated temperatures performed equal to natural aggregate concrete when considering the compressive strength results obtained in this study. However, the splitting tensile strength at all the temperature conditions were slightly lower for the RAC compared to the dolomite concrete (FA0). The reduced performance might be attributed to either the weak ITZ between the old cement paste (forming part of the RCA) and the new hardened cement paste or induced microcracking during sourcing and reprocessing. This again highlights that tensile strength is much more affected by microcrack development in the concrete due to elevated temperatures than compressive strength. As with compressive strength, the actual splitting tensile strength of the combined concrete mixture (RCAF) was fairly similar to that of the dolomite concrete (FA0). However, the residual splitting tensile strength for the combined concrete (RCAF) were substantially lower compared to the dolomite concrete (FA0) as well as concrete containing pure Portland cement, superplasticiser and dolomite aggregate (SP). The combined concrete (RCAF) performed fairly similar to the RAC when considering the residual splitting tensile strength.

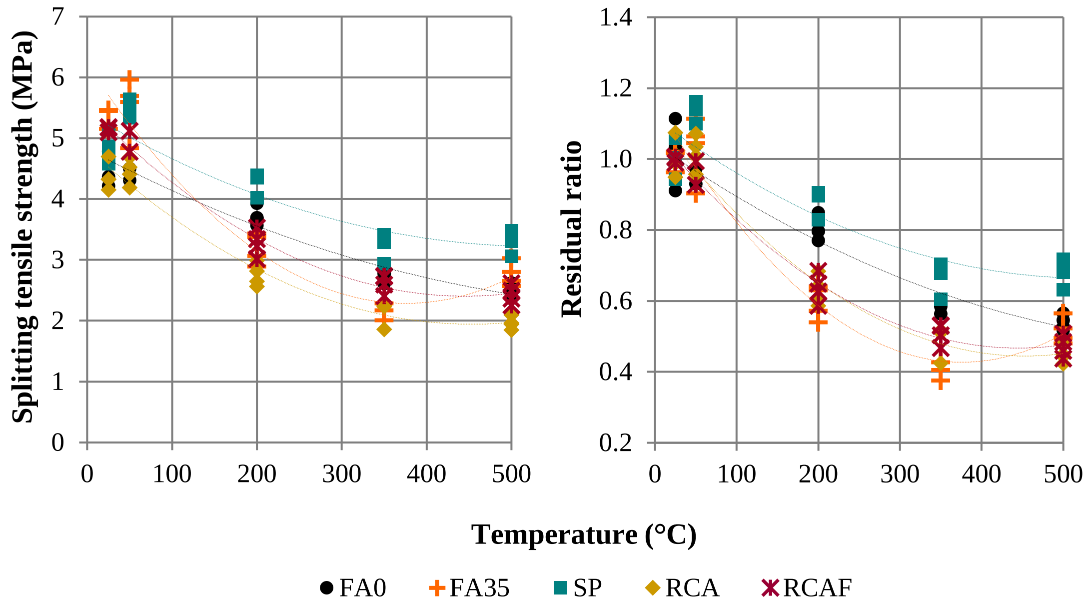

Figure 6-12: Effect of temperature on the tensile strength of environmentally friendly concrete

6.4 MODULUS OF ELASTICITY

The decrease in the stiffness of concrete exposed to elevated temperatures is discussed in this section. All the specimen were tested saturated according to standard test methods. As mentioned in literature, factors influencing elastic modulus of concrete at elevated temperatures are the type of aggregate used, water/cement ratio, cement properties, the compaction of the mix, and the presence of microcracks.

The effect of early age temperature exposure (7-days after casting) on the modulus of elasticity is illustrated in **Figure 6-13**. A gradual loss in stiffness was observed up to 500°*C*. The difference in modulus of elasticity of concrete subjected to elevated temperatures at an early age compared to exposure after twenty-eight days of curing was deemed insignificant. No autogenous recovery was observed in any of these results. As with the residual compressive strength and splitting tensile strength, the inclusion of polypropylene fibres had a limited effect on the modulus of elasticity behaviour after exposure to elevated temperatures.

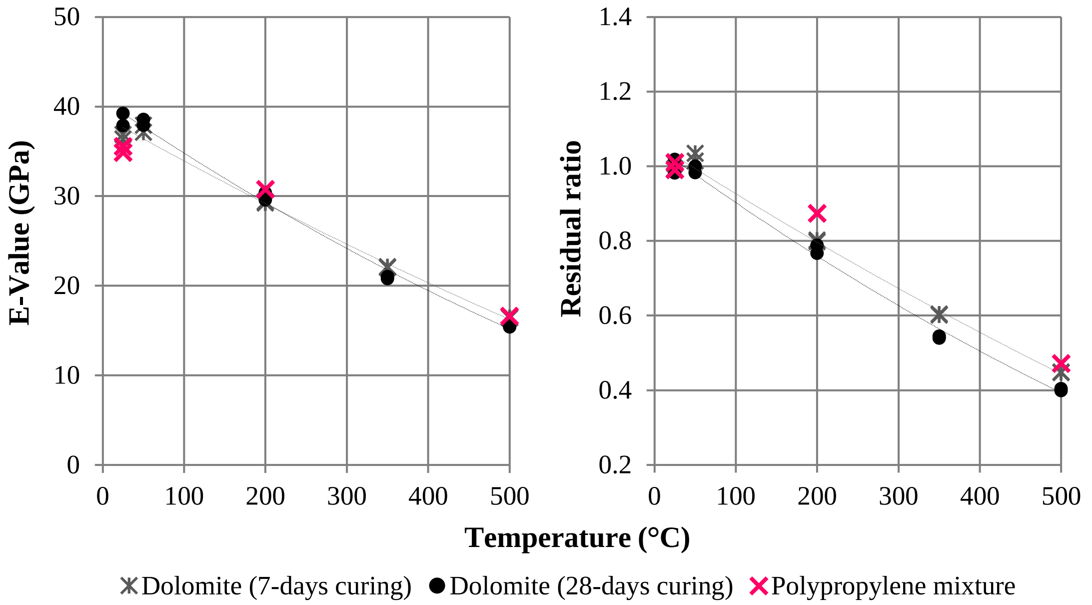

Figure 6-13: Effect of temperature on concrete stiffness

The effect of elevated temperatures on the modulus of elasticity of concrete with varying water/cement ratios is illustrated in **Table 6-6** and **Figure 6-14**. The concrete stiffness gradually reduced as the exposure temperature increased. Concrete with higher water/cement ratios, 0.6 and 0.7, initially had the lowest water cured modulus of elasticity values. However due to autogenous recovery at 500°*C*, the concrete with higher water/cement ratios produced substantially higher modulus of elasticity values compared to the concrete with low water/cement ratios. Concrete with a water/cement ratio of 0.3 retained less than 30% of its initial water cured modulus of elasticity after exposure to 500°*C*, while concrete with a water/cement ratio of 0.7 maintained approximately 70% its initial stiffness. Similar trends were observed with both the compressive strength and splitting tensile strength results. The results highlight that water/cement ratio have a notable influence of the behaviour of concrete subjected to elevated temperatures.

The effect of elevated temperature on the modulus of elasticity of concrete with varying coarse to total aggregate ratios is shown in

Table 6-7 and **Figure 6-15**. The results indicate similar trends to both the corresponding compressive strength and splitting tensile strength results. The initial water cured modulus of elasticity values of concrete with varying coarse to total aggregate ratios were fairly similar, however the variation in results increased as the exposure temperature increased.

Table 6-6: Effect of w/c ratio on concrete stiffness as a function of temperature

		D0.3	D0.4	D0.5	D0.6	D0.7
50°C	Modulus of elasticity (*GPa*)	39.4	38.8	38.2	34.0	31.5
	Standard deviation (*GPa*)	0.19	1.47	0.46	0.25	2.08
	Coefficient of variation	0.00	0.04	0.01	0.01	0.07
	Residual ratio	0.95	0.96	0.99	0.90	1.07
200°C	Modulus of elasticity (*GPa*)	32.1	32.9	30.0	28.8	25.2
	Standard deviation (*GPa*)	2.30	0.00	0.57	0.69	0.28
	Coefficient of variation	0.07	0.00	0.02	0.02	0.01
	Residual ratio	0.77	0.81	0.78	0.76	0.85
350°C	Modulus of elasticity (*GPa*)	18.1	23.0	20.9	21.5	19.1
	Standard deviation (*GPa*)	0.43	0.50	0.14	1.56	1.41
	Coefficient of variation	0.02	0.02	0.01	0.07	0.07
	Residual ratio	0.44	0.57	0.54	0.57	0.65
500°C	Modulus of elasticity (*GPa*)	10.3	17.3	15.5	21.1	19.8
	Standard deviation (*GPa*)	0.14	0.24	0.13	0.45	0.63
	Coefficient of variation	0.01	0.01	0.01	0.02	0.03
	Residual ratio	0.25	0.43	0.40	0.56	0.67

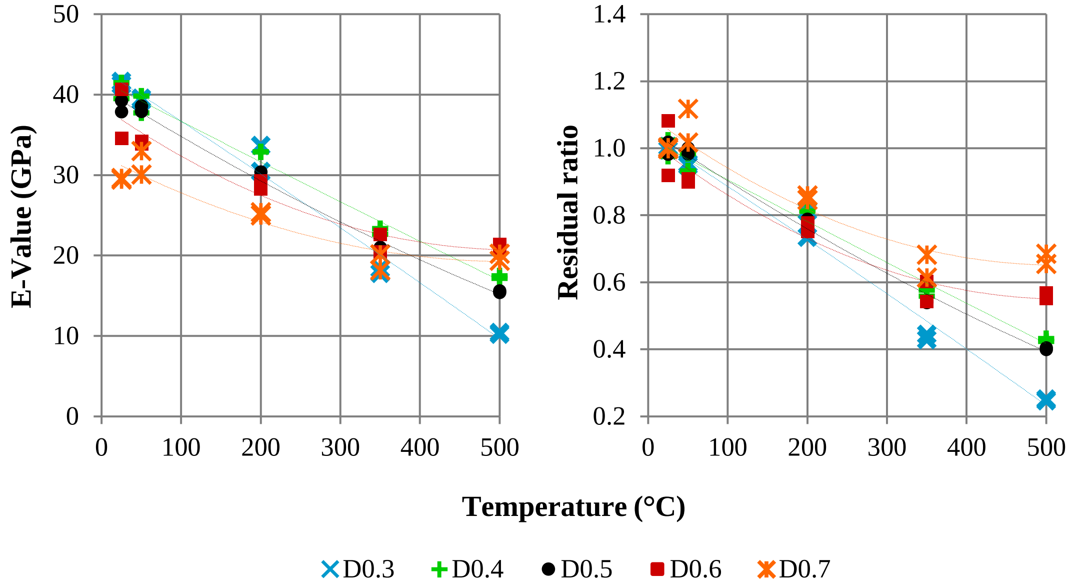

Figure 6-14: Effect of w/c ratio on concrete stiffness as a function of temperature

Table 6-7: Effect of sand content on concrete stiffness as a function of temperature

		D0	D15	D30	D45	D60	D75
50°C	Modulus of elasticity (*GPa*)	35.4	36.2	37.3	38.2	36.3	38.4
	Standard deviation (*GPa*)	0.13	0.14	0.76	0.46	0.47	1.69
	Coefficient of variation	0.00	0.00	0.02	0.01	0.01	0.04
	Residual ratio	1.00	0.98	0.98	0.99	0.95	0.96
200°C	Modulus of elasticity (*GPa*)	29.6	29.4	29.8	30.0	29.8	28.8
	Standard deviation (*GPa*)	0.68	0.13	0.30	0.57	1.30	0.97
	Coefficient of variation	0.02	0.00	0.01	0.02	0.04	0.03
	Residual ratio	0.83	0.80	0.78	0.78	0.78	0.72
350°C	Modulus of elasticity (*GPa*)	23.6	22.0	21.6	20.9	19.5	16.3
	Standard deviation (*GPa*)	0.39	0.22	0.28	0.14	0.01	1.32
	Coefficient of variation	0.02	0.01	0.01	0.01	0.00	0.08
	Residual ratio	0.66	0.60	0.56	0.54	0.51	0.41
500°C	Modulus of elasticity (*GPa*)	22.0	18.1	16.9	15.5	13.3	12.1
	Standard deviation (*GPa*)	1.28	0.12	0.27	0.13	0.40	0.50
	Coefficient of variation	0.06	0.01	0.02	0.01	0.03	0.04
	Residual ratio	0.62	0.49	0.44	0.40	0.35	0.30

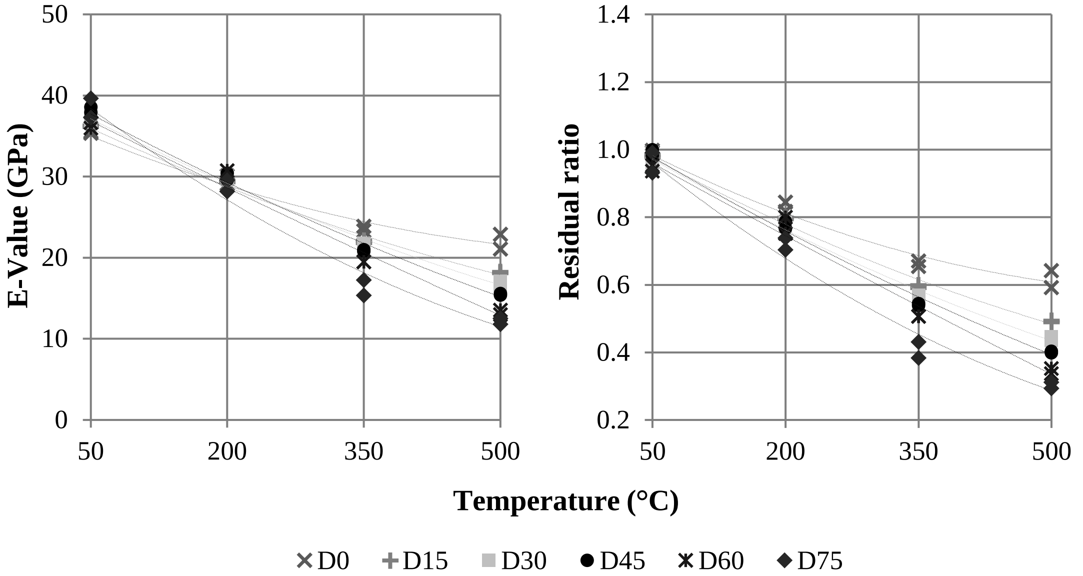

Figure 6-15: Effect of sand content on concrete stiffness as a function of temperature

Corresponding with previously published research, aggregate type has a notable effect on the elastic modulus of concrete at elevated temperatures as indicated in **Table 6-8** and **Figure**

6-16. The results obtained follow the same trend as observed for the splitting tensile strength results. A decrease in the elastic modulus with increasing temperatures was observed. This may be attributed to the degradation of the interface zone between the microstructure of the hardened cement paste and the aggregate. As mentioned in literature, materials with high coefficient of thermal expansion such as granites and dolomites should not be used in circumstances with large temperature variations. This coincides with the results obtained in this study. The elastic modulus of concrete containing dolomite aggregates substantially reduced when exposed to elevated temperatures and only 40*%* of the initial stiffness was retained after exposure to 500*°C*. Dolomite showed superior performance with water cured twenty-eight-day modulus of elasticity compared to the other aggregate types. However, the stiffness reduced to such an extent that dolomite aggregate produced the lowest elastic modulus after exposure to 500*°C*. The concrete containing felsite aggregates outperformed all the other aggregates, as 80*%* of the initial stiffness was retained after exposure to 500*°C*. Autogenous recovery after exposure to 500*°C* was also observed for concrete containing andesite, dolerite and tillite. The results highlight the notable influence of aggregate type on the elastic properties of concrete subjected to elevated temperatures.

Table 6-8: Effect of aggregate type on concrete stiffness as a function of temperature

		A	**R**	**D**	**F**	**G**	**T**
50°*C*	Modulus of elasticity (*GPa*)	33.8	30.9	38.2	34.4	29.1	28.4
	Standard deviation (*GPa*)	1.19	0.51	0.46	1.74	0.35	0.25
	Coefficient of variation	0.04	0.02	0.01	0.05	0.01	0.01
	Residual ratio	1.06	0.90	0.99	0.99	0.93	0.99
200°*C*	Modulus of elasticity (*GPa*)	27.8	24.0	30.0	28.9	24.8	23.8
	Standard deviation (*GPa*)	0.77	0.86	0.57	0.94	0.21	0.80
	Coefficient of variation	0.03	0.04	0.02	0.03	0.01	0.03
	Residual ratio	0.87	0.70	0.78	0.83	0.80	0.83
350°*C*	Modulus of elasticity (*GPa*)	21.4	19.8	20.9	27.5	18.4	18.4
	Standard deviation (*GPa*)	0.78	0.04	0.14	0.97	0.15	0.17
	Coefficient of variation	0.04	0.00	0.01	0.04	0.01	0.01
	Residual ratio	0.68	0.58	0.54	0.79	0.59	0.64
500°*C*	Modulus of elasticity (*GPa*)	22.4	21.7	15.5	27.6	17.7	18.4
	Standard deviation (*GPa*)	1.02	1.22	0.13	0.18	0.53	0.28
	Coefficient of variation	0.05	0.06	0.01	0.01	0.03	0.02
	Residual ratio	0.70	0.63	0.40	0.80	0.57	0.64

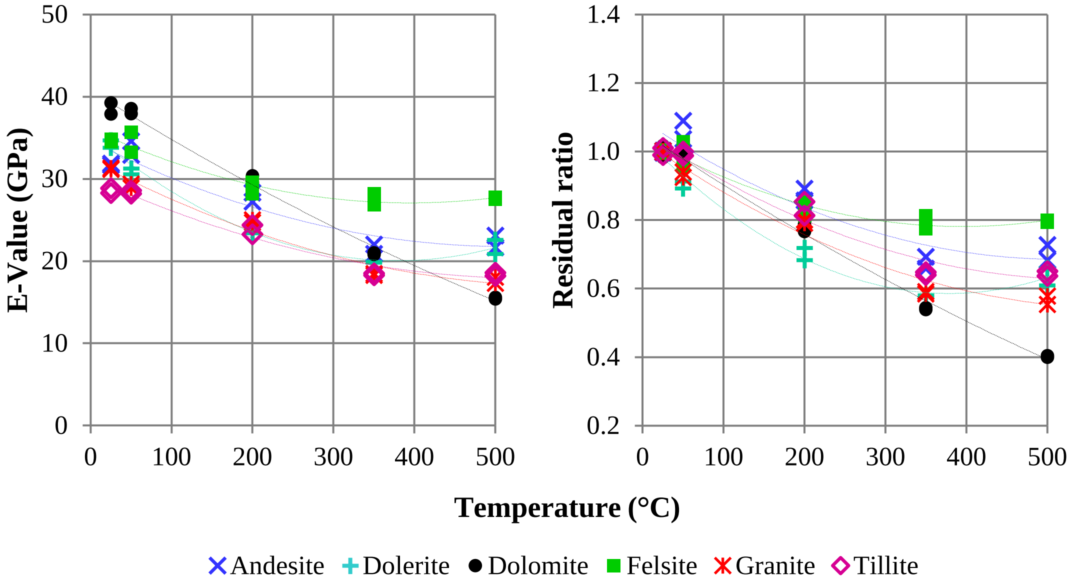

Figure 6-16: Effect of aggregate type on concrete stiffness as a function of temperature

The effect of elevated temperatures on the modulus of elasticity of blended cement concrete is indicated in **Figure 6-17** and **Table 6-9**. The modulus of elasticity decreased as the replacement percentage of cement with fly ash increased, for each temperature condition up to 350°*C*, whereafter, the stiffness recovered at 500°*C* for the concrete containing 35*%* and 50*%* fly ash. The difference in modulus of elasticity after autogenous recovery between the different concrete mixtures was deemed insignificant.

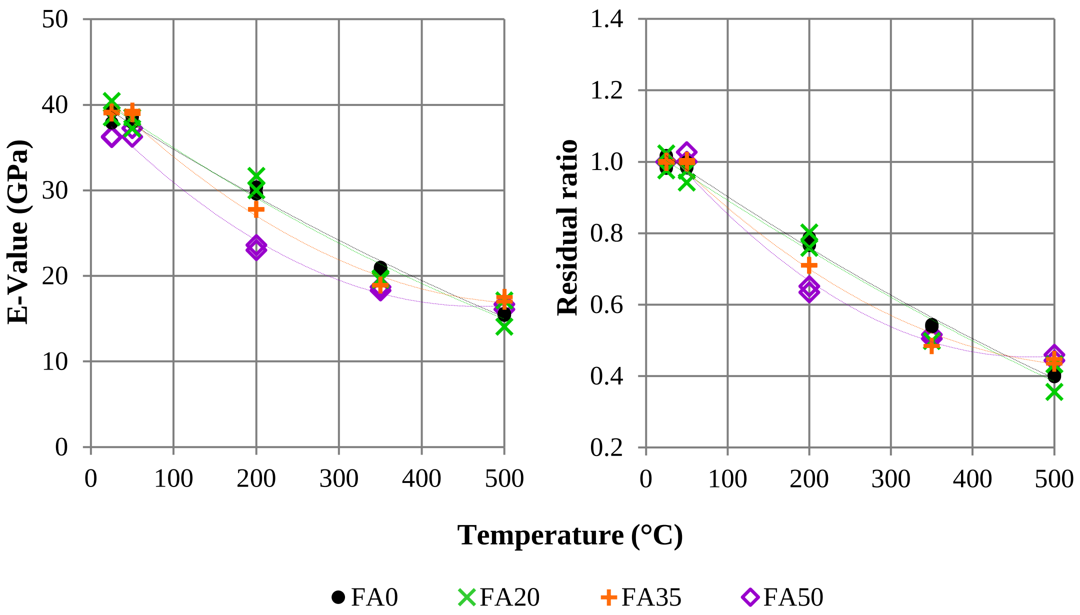

Figure 6-17: Effect of fly ash on concrete stiffness as a function of temperature

Table 6-9: Effect of fly ash on concrete stiffness as a function of temperature

		FA0	**FA20**	**FA35**	**FA50**
50°C	Modulus of elasticity (*GPa*)	38.2	37.9	39.1	36.7
	Standard deviation (*GPa*)	0.46	0.93	0.26	0.71
	Coefficient of variation	0.01	0.02	0.01	0.02
	Residual ratio	0.99	0.96	1.00	1.01
200°C	Modulus of elasticity (*GPa*)	30.0	30.8	27.8	23.3
	Standard deviation (*GPa*)	0.57	1.21	0.06	0.43
	Coefficient of variation	0.02	0.04	0.00	0.02
	Residual ratio	0.78	0.78	0.71	0.64
350°C	Modulus of elasticity (*GPa*)	20.9	19.6	18.9	18.5
	Standard deviation (*GPa*)	0.14	0.14	0.07	0.28
	Coefficient of variation	0.01	0.01	0.00	0.02
	Residual ratio	0.54	0.50	0.48	0.51
500°C	Modulus of elasticity (*GPa*)	15.5	15.6	17.3	16.4
	Standard deviation (*GPa*)	0.13	2.17	0.36	0.42
	Coefficient of variation	0.01	0.14	0.02	0.03
	Residual ratio	0.40	0.39	0.44	0.45

The effect of elevated temperature exposure on the modulus of elasticity of environmentally friendly concrete is shown in **Table 6-10** and **Figure 6-18**. The stiffness as well as variation in stiffness gradually reduced with an increase in exposure temperature. Substantial differences were observed in the modulus of elasticity of the different concretes. Concrete containing superplasticiser produced the highest elastic modulus at all temperature conditions, while the RAC produced the lowest elastic modulus. However, all the concrete mixtures retained similar percentages of their initial water cured modulus of elasticity at all temperature conditions. The deterioration in modulus of elasticity of all the concretes was therefore comparable and differences were deemed insignificant.

Table 6-10: Effect of temperature on the modulus of elasticity of environmentally friendly concrete

		FA0	FA35	SP	RCA	RCAF
50°C	Modulus of elasticity (*GPa*)	38.2	39.1	46.1	29.5	34.6
	Standard deviation (*GPa*)	0.46	0.26	1.11	0.27	0.61
	Coefficient of variation	0.01	0.01	0.02	0.01	0.02
	Residual ratio	0.99	1.00	1.0	1.0	1.0
200°C	Modulus of elasticity (*GPa*)	30.0	27.8	39.4	21.1	26.1
	Standard deviation (*GPa*)	0.57	0.06	1.06	0.45	0.18
	Coefficient of variation	0.02	0.00	0.03	0.02	0.01
	Residual ratio	0.78	0.71	0.85	0.71	0.76
350°C	Modulus of elasticity (*GPa*)	20.9	18.9	24.9	14.9	17.8
	Standard deviation (*GPa*)	0.14	0.07	0.97	0.49	0.25
	Coefficient of variation	0.01	0.00	0.04	0.03	0.01
	Residual ratio	0.54	0.48	0.54	0.50	0.51
500°C	Modulus of elasticity (*GPa*)	15.5	17.3	20.7	11.1	14.5
	Standard deviation (*GPa*)	0.13	0.36	0.04	0.23	0.02
	Coefficient of variation	0.01	0.02	0.00	0.02	0.00
	Residual ratio	0.40	0.44	0.45	0.38	0.42

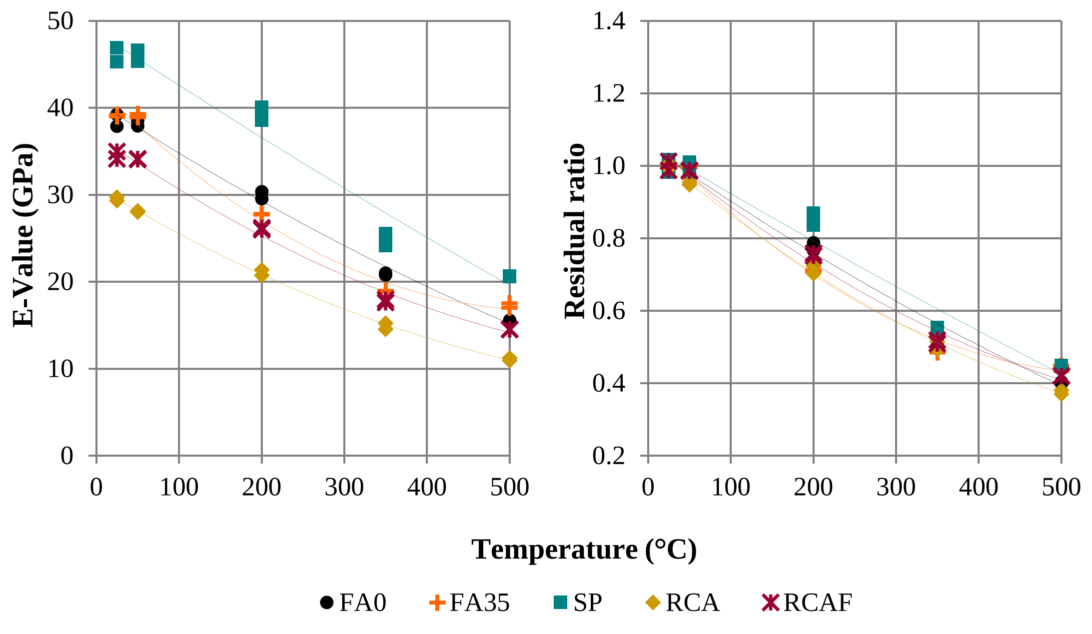

Figure 6-18: Effect of temperature on the modulus of elasticity of environmentally friendly concrete

6.5 MASS LOSS

According to literature, mass loss take place when concrete is exposed to elevated temperatures. These changes are almost entirely caused by the loss of evaporable water and can be minimised by using concrete with a low thermal expansion. The effect of early age temperature exposure (7-days after casting) on the percentage mass loss is shown in **Figure 6-19 a)**. The influence of specimen size as well as inclusion of polypropylene fibres was also considered. Early age temperature exposure produced slightly lower mass loss percentages at higher temperatures (350°*C* and 500°*C*) compared to exposure after twenty-eight days of curing. These specimen were however dried before exposure to heat and less chemically bound water would be present at early ages, resulting in more evaporation before heat exposure. The inclusion of polypropylene fibres had a limited effect on the percentage mass loss.

According to literature, higher water cement ratios caused higher percentage mass loss when concrete is subjected to elevated temperatures. This corresponds with results obtained in this study for only the lower temperature exposure conditions (50°*C* and 200°*C*) as seen in **Figure 6-19 b)**. However, when the concrete was subjected to 500°*C*, higher water/cement ratios caused a slightly lower percentage mass loss compared to concrete with lower water/cement ratios. The effect of elevated temperature on the percentage mass loss of concrete with different coarse to total aggregate ratios is shown in **Figure 6-19 c)**. The concrete with higher sand contents had lower percentage mass loss at 50°*C* due to the denser microstructure of the concrete. However, the percentage mass loss increased as the temperature conditions increased, where lower mass loss percentages were measured at 500°*C* for concrete with higher coarse aggregate contents.

The effect of elevated temperature on the percentage mass loss of concrete with different aggregate types is illustrated in **Figure 6-19 d)**. The general trend indicates that tillite and granite aggregate measured the highest mass loss percentages for all temperature conditions, where andesite had the lowest percentages. Further testing would be required to confirm whether dolomite aggregate concrete could experience higher percentage mass loss due to dissociation of dolomite at temperatures above 600°*C*, as mentioned in literature.

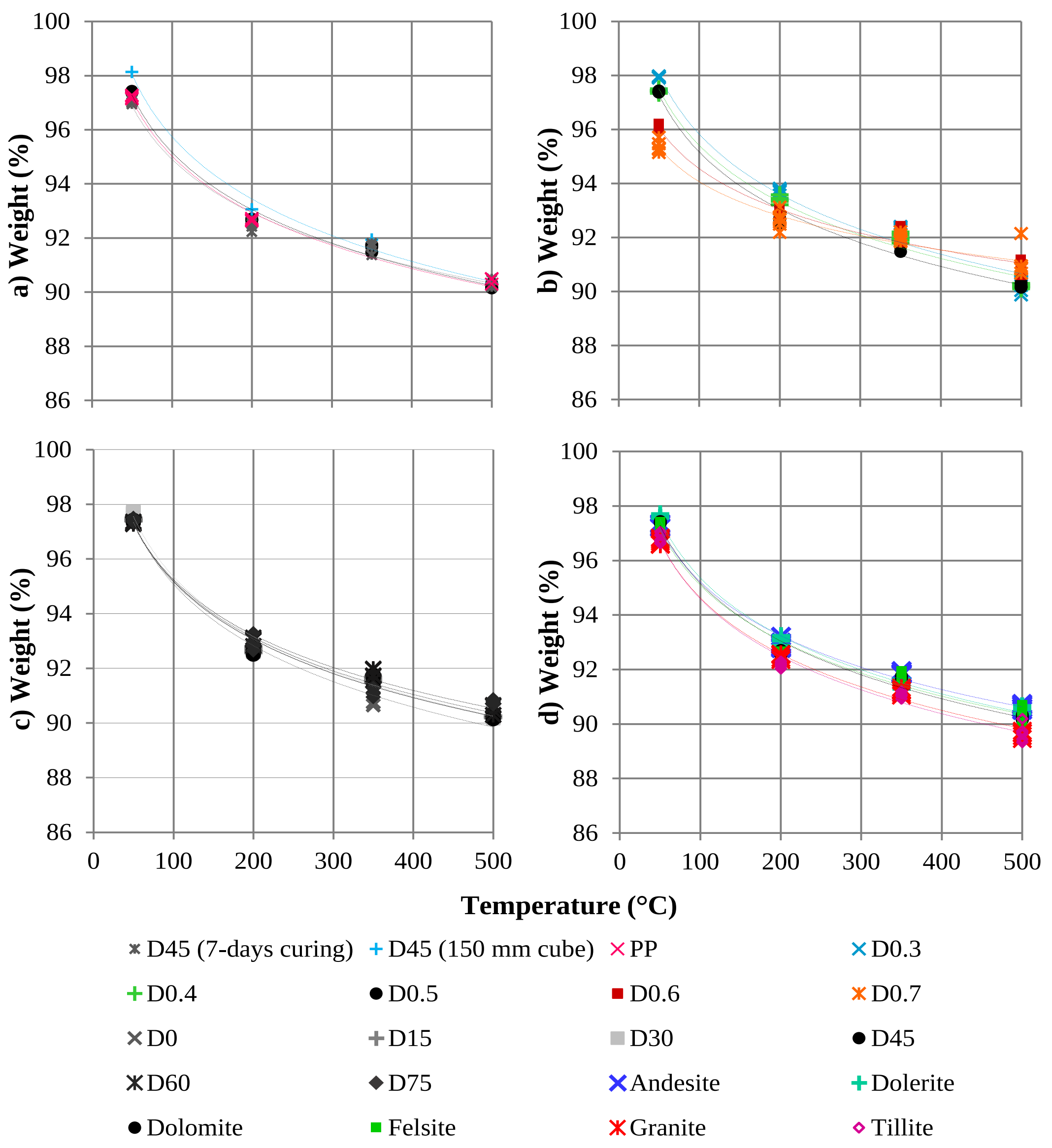

Figure 6-19: Effect of temperature on the mass loss of concrete

The effect of elevated temperature on the percentage mass loss of environmentally friendly concrete is shown in **Figure 6-20**. Corresponding to literature, the mass loss for the RAC was greater than that of the natural aggregate concrete (FA0). This was attributed to the higher water absorption of the aggregates and hence increased water content in the RCA mixture due to pre-saturation of the aggregates. The reduced water content in the concrete containing pure Portland cement, superplasticiser and dolomite aggregate (SP) resulted in reduced mass loss percentages. The mass loss for the dolomite concrete (FA0), concrete containing fly ash and the combined concrete (RCAF) was fairly similar.

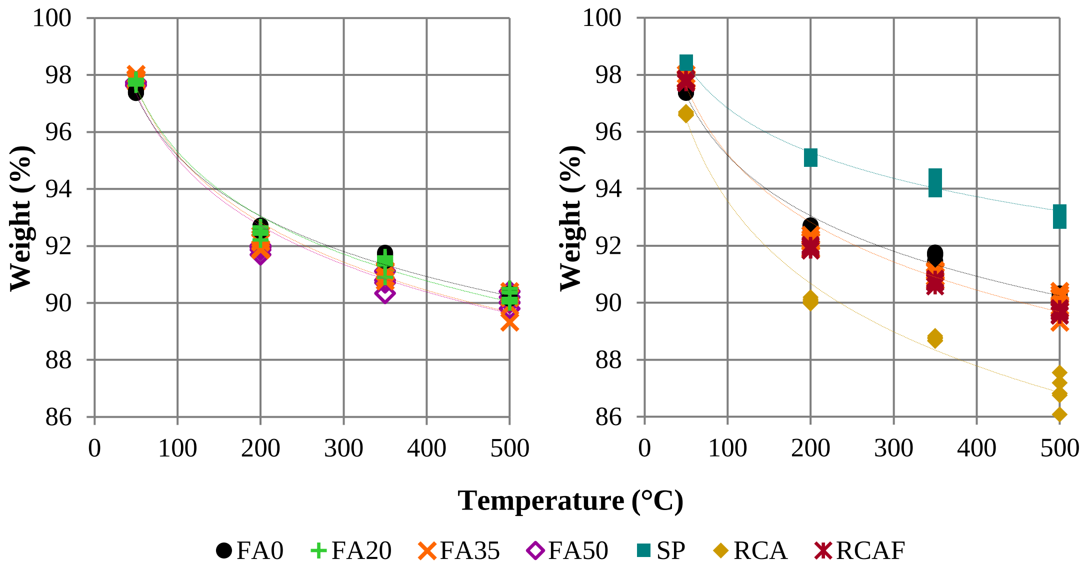

Figure 6-20: Effect of temperature on the mass loss of environmentally friendly concrete

6.6 POTENTIAL CONCRETE DURABILITY

As mentioned in the literature review, there is limited research on the effect of elevated temperatures on the potential durability performance of concrete. The effect of elevated temperature on the potential durability of concrete with varying coarse to total aggregate ratios and different aggregate types is illustrated in **Figure 6-21** and **Figure 6-22** respectively. Potential durability of the concrete gradually reduced as the exposure temperature increased. The measured OPI values at all temperature conditions decreased as the coarse aggregate content increased. Substantial variations in the measured OPI values were obtained between the different aggregate types and no clear trend could be observed. According to the suggested durability classification for OPI values by Alexander et al. (1999), excellent and good durability concrete for all fractions and aggregate types were obtained up to 200°*C*, whereafter the potential concrete durability rapidly deteriorates with very poor durability concrete after exposure to 500°*C*. When considering the suggested durability classification for porosity values by Moore et al. (2021), poor to very poor concrete for all fractions and aggregate types were obtained up to 200°*C*. The measured porosity values, in both figures, substantially increased as the exposure temperature increased with very poor durability concrete after exposure to 500°*C*. The loss of potential concrete durability may be attributed to the weakened ITZ between the hardened cement paste and the aggregates. The results highlight the substantial effect of elevated temperatures on the potential durability of concrete. It can be concluded that for temperatures smaller than 200°*C* there will be limited loss in potential concrete durability for all fractions and aggregate types. For higher temperature condition the concrete can be considered as no longer durable.

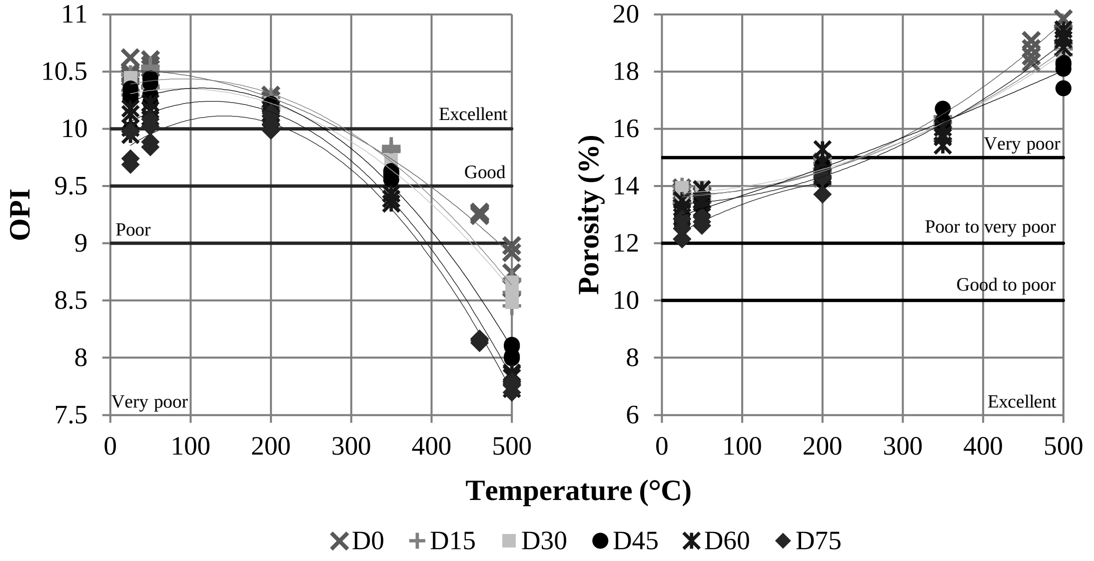

Figure 6-21: Effect of sand content on potential concrete durability as a function of temperature

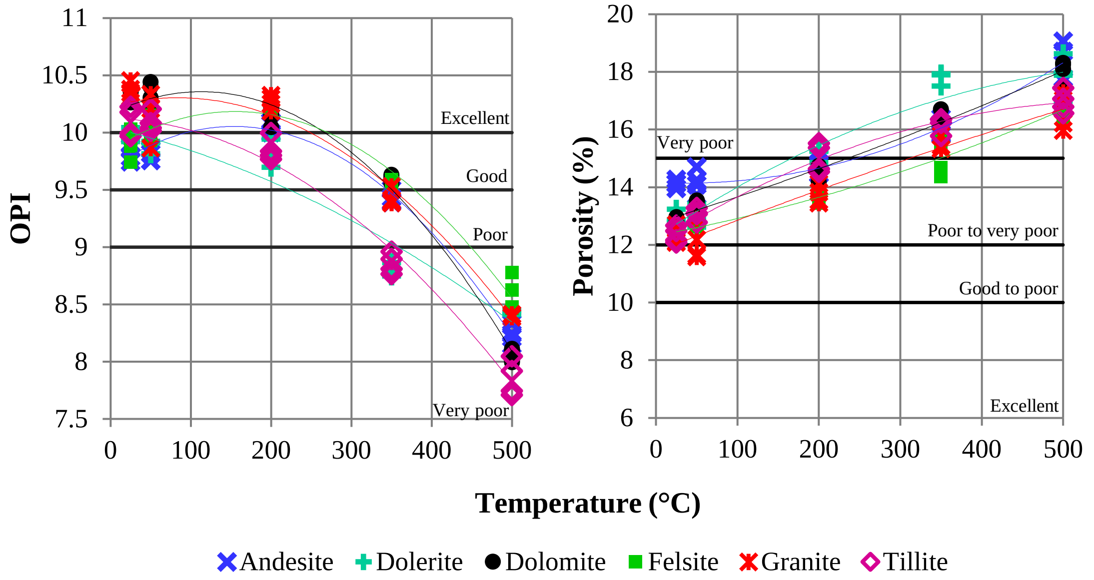

Figure 6-22: Effect of aggregate type on potential concrete durability as a function of temperature

6.7 INTERPRETATION OF RESULTS

This chapter provides a discussion of the results and the necessary trends observed throughout the study while focussing on the effect of elevated temperature on the properties of concrete made with different aggregate types, including RCA. All the residual compressive strength, splitting tensile strength and modulus of elasticity ratios were compared to literature as indicated in **Figure 6-23**. The EN 1992-1-2 together with the ACI 261.1-2007 standards for strength reduction of normal weight concrete with calcareous aggregates at elevated temperatures is also illustrated in the figure. Other aggregate types such as felspathic and granitic are not considered in the standards. The experimental results all meet the lower range obtained from previous literature as indicated in the figure. Some results fall outside the upper range, indicating that higher residual strength values were obtained in this study compared to previous studies. Furthermore, it is clear from the graph that tensile strength and modulus of elasticity is much more affected by microcrack development in the concrete due to elevated temperatures than compressive strength.

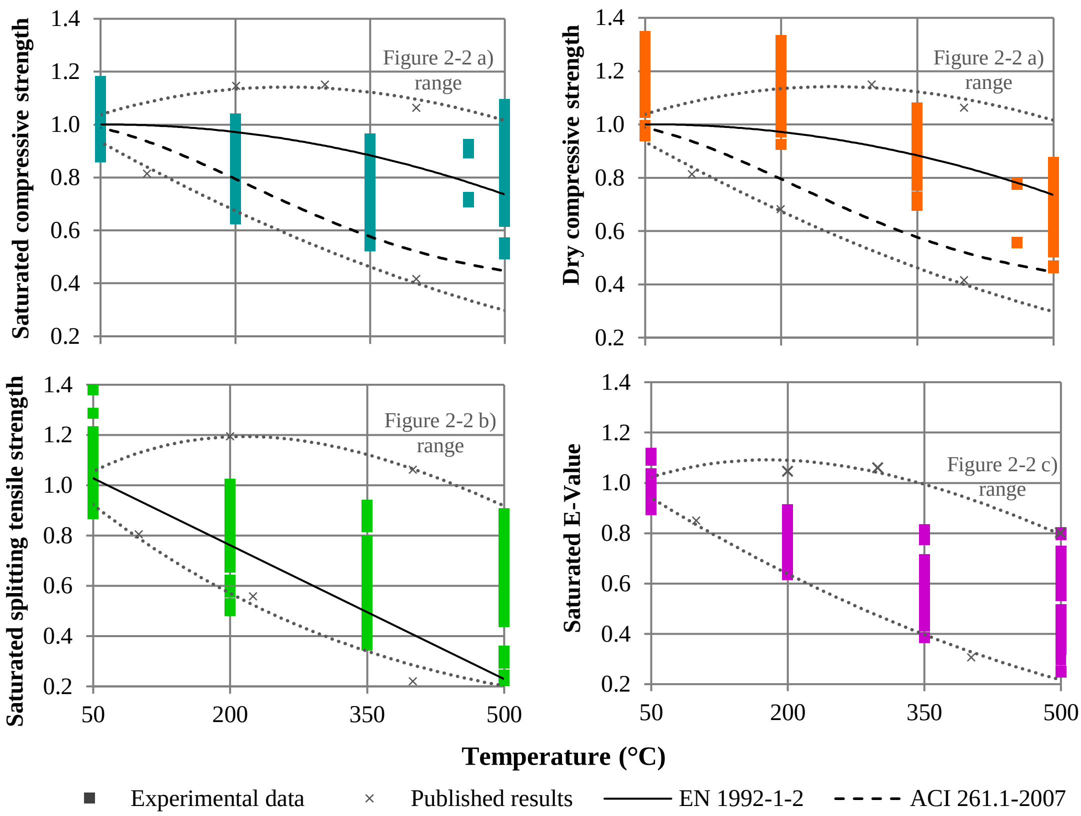

Figure 6-23: Experimental results compared to published literature

The effect of temperature on concrete strength as a function of paste volume is illustrated in **Figure 6-24**. Typical concrete mixtures have paste volumes of approximately 300 l/m^3, however with higher strength concrete the cement content increases resulting in an increase in paste volume. The paste volumes for mixtures used in this study with a water/cement ratio of 0.3 and 0.4 are extremely high and impractical (> 400 l/m^3). From the graph it can be seen that these high paste volumes show much more deterioration in strength after exposure to elevated temperatures. It is also evident that mixtures with lower paste volumes show more autogenous recovery when tested saturated than the mixture with higher paste volumes. It would therefore be recommended to avoid the use of concrete mixtures with excessive paste volumes or cement contents.

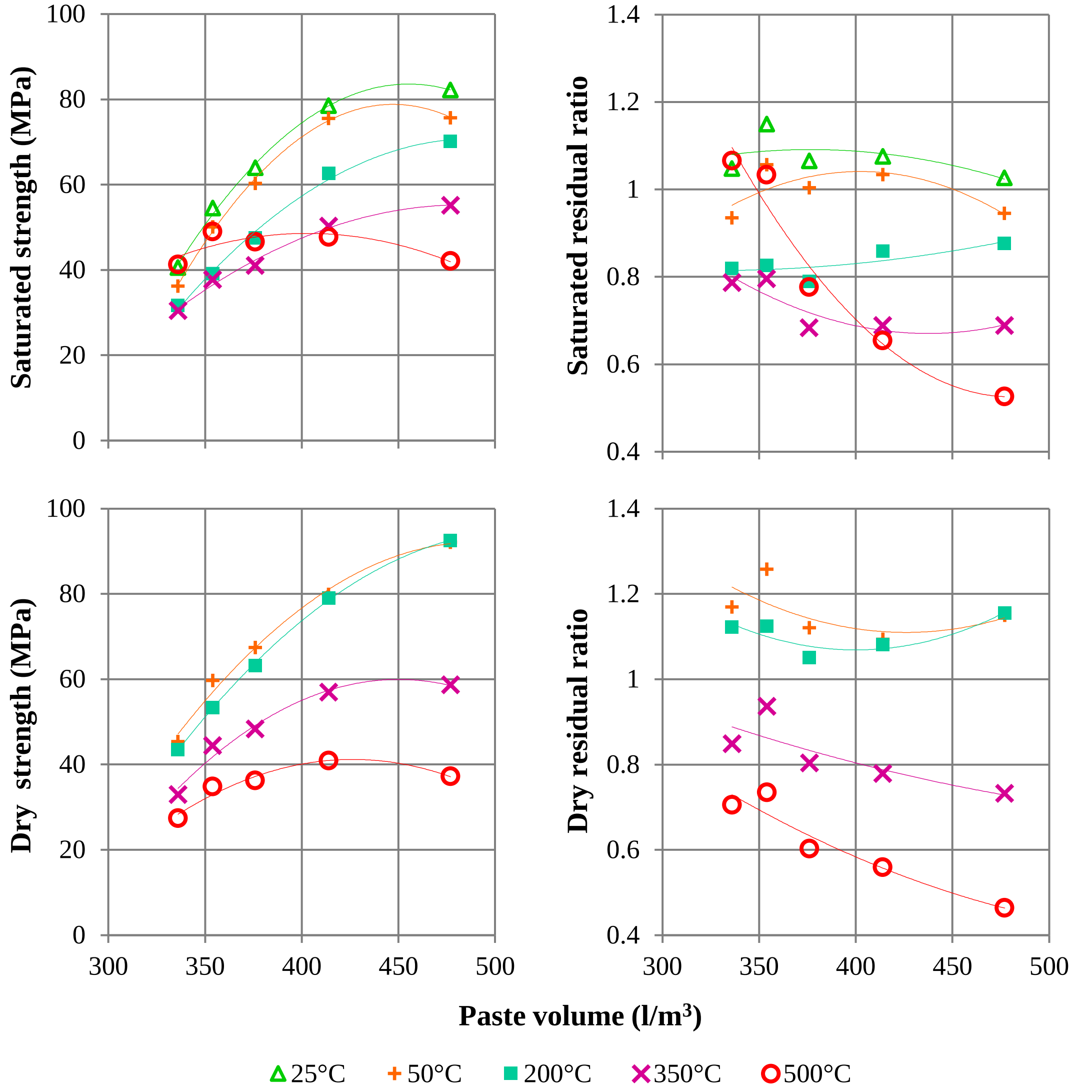

Figure 6-24: Effect of temperature on concrete strength as a function of paste volume

The effect of the CTE of concrete as well as the mass loss for the different aggregate types obtained from the TGA (thermogravimetric analysis) on the dry compressive strength results is illustrated in **Figure 6-25**. It is well known that concrete exposed to elevated temperatures in service should preferably contain aggregate with a low coefficient of thermal expansion (Ballim & Otieno, 2021). This relates to the results obtained in this study, as concrete with lower CTE yielded higher compressive strength results after exposure to elevated temperatures (**Figure 6-25 a**). However, for each of the three exposure temperatures there seems to be a maximum compressive strength after exposure for aggregates with TGA mass loss in the region of 1% – 2% (**Figure 6-25 b**). Aggregates with lower and higher mass loss yielded lower dry compressive strengths. These turning points (blue cross) for each exposure temperature is illustrated in **Figure 6-26**.

The graph indicates that if a concrete structure, such as containment structures in nuclear power stations, is exposed to 350°*C*, aggregates with a mass loss percentage at 350°*C* of approximately 1.2*%* should preferable be used in the concrete design to yield maximum strength results when exposed to 350°*C* temperatures. The fact that a clear maximum residual compressive strength was observed for each exposure temperature, indicates that there could be benefit in using aggregates that show limited degradation at the exposure temperature. As the aggregates expands due to heat exposure at a rate determined by the CTE, the stress placed on the cement paste can be limited if the aggregate at the same time reduces in volume due the volatile mineral decomposition as indicated by the TGA results. Excessive aggregate mass loss would not only result in gaps forming in the concrete, but also in loss of integrity of the aggregate. Further testing is required to evaluate the results obtained for mass loss of aggregate from TGA against the behaviour after exposure to elevated temperatures with other aggregate types not used in this study.

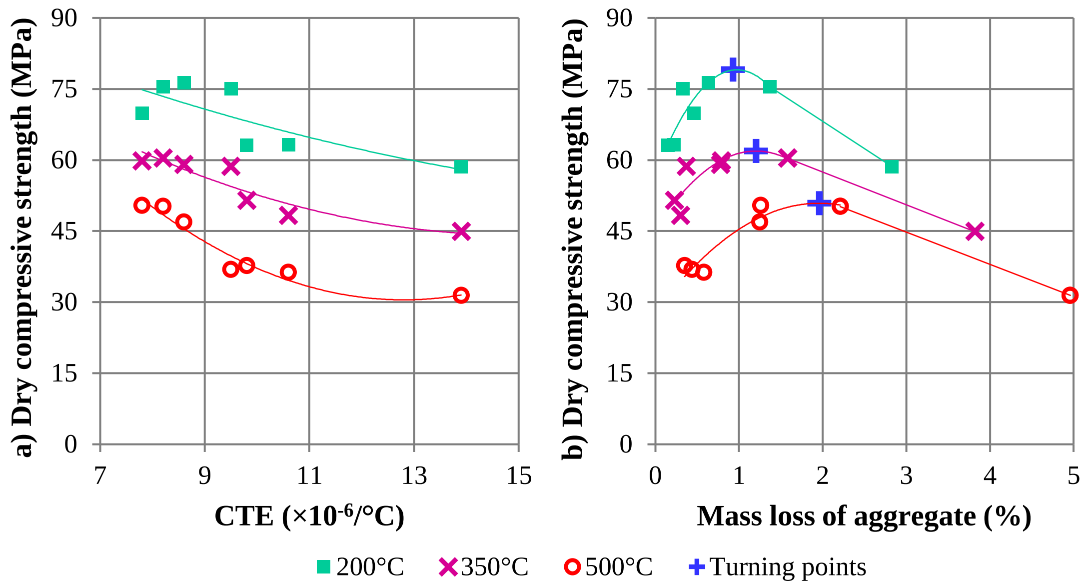

Figure 6-25: The effect of a) CTE of concrete and b) aggregate mass loss on the dry compressive strength

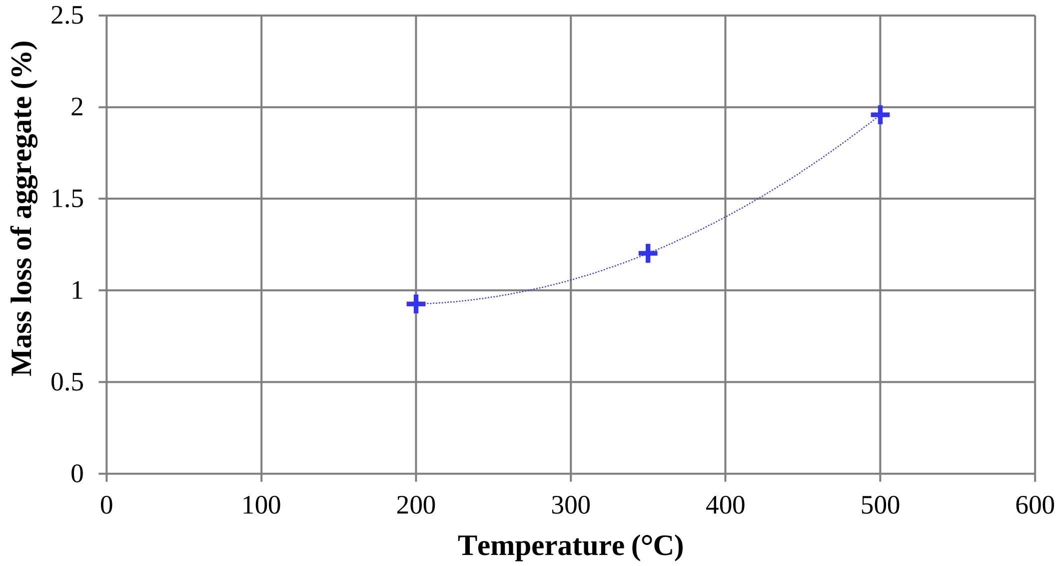

Figure 6-26: Turning points for each exposure temperature

The effect of the CTE of concrete as well as the mass loss for the different aggregate types obtained from the TGA on the saturated compressive strength results is illustrated in **Figure 6-27**. Unlike the dry compressive strength results, no clear trend could be observed. This may be attributed to autogenous recovery observed in all of the specimen at 500°*C*. However, when considering all the results it can be seen that most of the saturated compressive strengths are higher than 40 *MPa* even after temperature exposure of up to 500°*C*.

The probability density function for the saturated compressive strength data (all three exposure temperatures) is illustrated in **Figure 6-28** together with the normal distribution function. Twenty-one results were used to obtain the probability density function. As mentioned in Chapter 3.6, concrete exhibits an inherent variability of strength. Concrete strength is therefore specified not at a minimum strength but as a characteristic concrete strength (f_{ck}). The difference between the characteristic strength and average compressive strength (target mean strength) is called the "margin" which is normally taken as 1.645 (5% defectives level) times the standard deviation (*SD*). By using Equation 3-1, a left-tail region which contains a probability (α = significance level) of 0.05 (5%) is illustrated in the graph. It can therefore be concluded with 95% confidence that the concrete exposed to elevated temperatures of up to 500°*C* retained a saturated compressive strength of 40 *MPa* (target mean strength) regardless of aggregate type used. Consequently, yielding a characteristic strength of at least 30 *MPa* (*SD* = 6.1 *MPa*). Furthermore, with 95% confidence it can be inferred that concrete exposed to 350°*C* retained more than 64% of its original strength, while concrete exposed to 500°*C* retained more than 70% of its original strength after autogenous recovery.

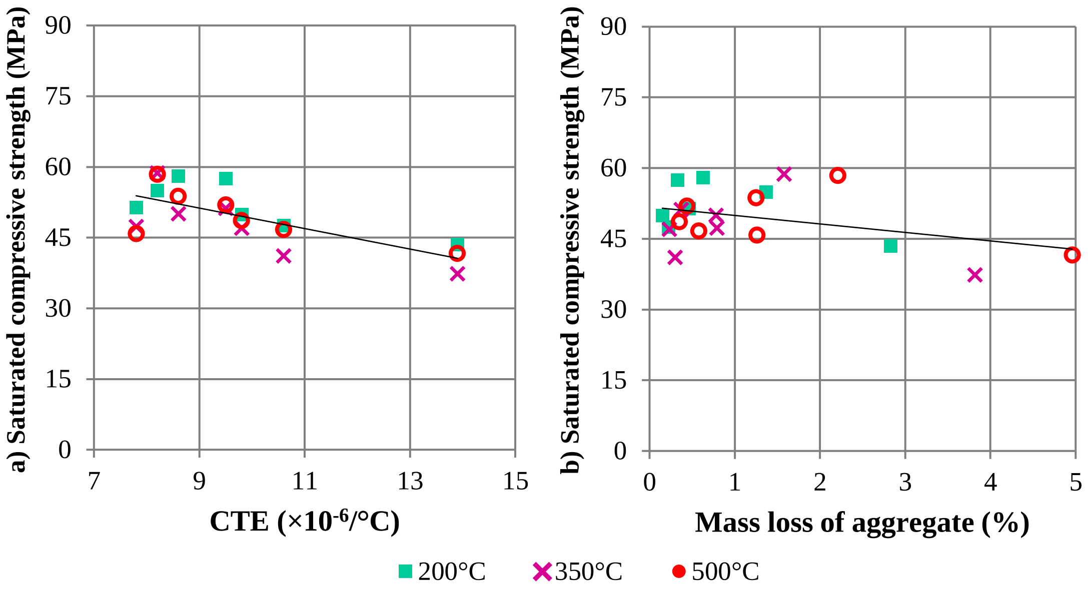

Figure 6-27: The effect of a) CTE of concrete and b) aggregate mass loss on the saturated compressive strength

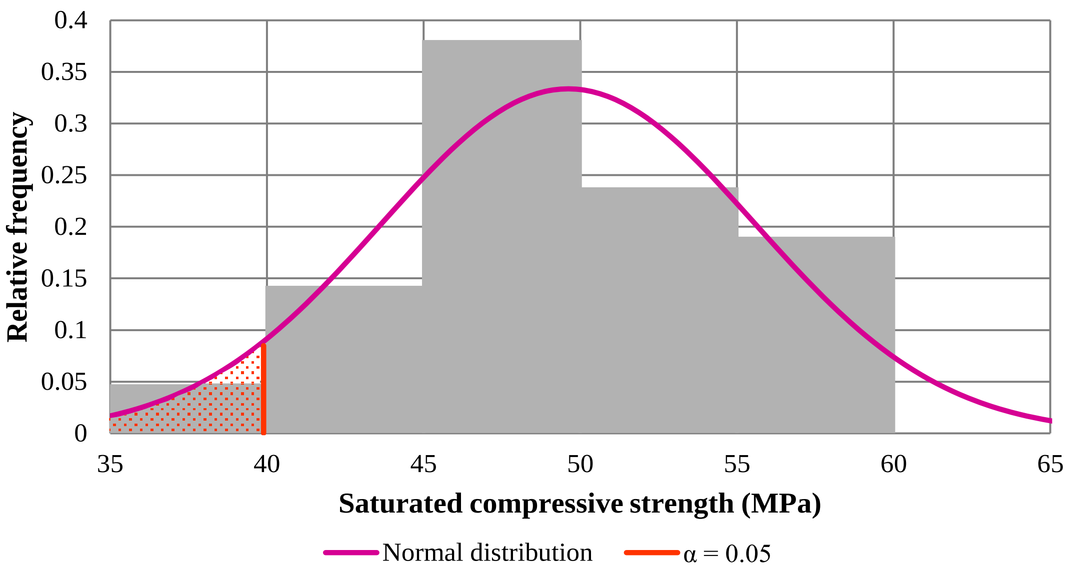

Figure 6-28: Probability density

The effect of the mass loss for the different aggregate types obtained from the TGA on the splitting tensile strength as well as the modulus of elasticity results is illustrated in **Figure 6-29**. No clear trend could be obtained. However, similar reduction in strength and stiffness can be observed for the splitting tensile strength and modulus of elasticity. As mentioned previously, the tensile strength and modulus of elasticity is much more affected by microcrack development in the concrete due to elevated temperatures. The graphs clearly confirm that the modulus of elasticity decreased as the splitting tensile strength decreased.

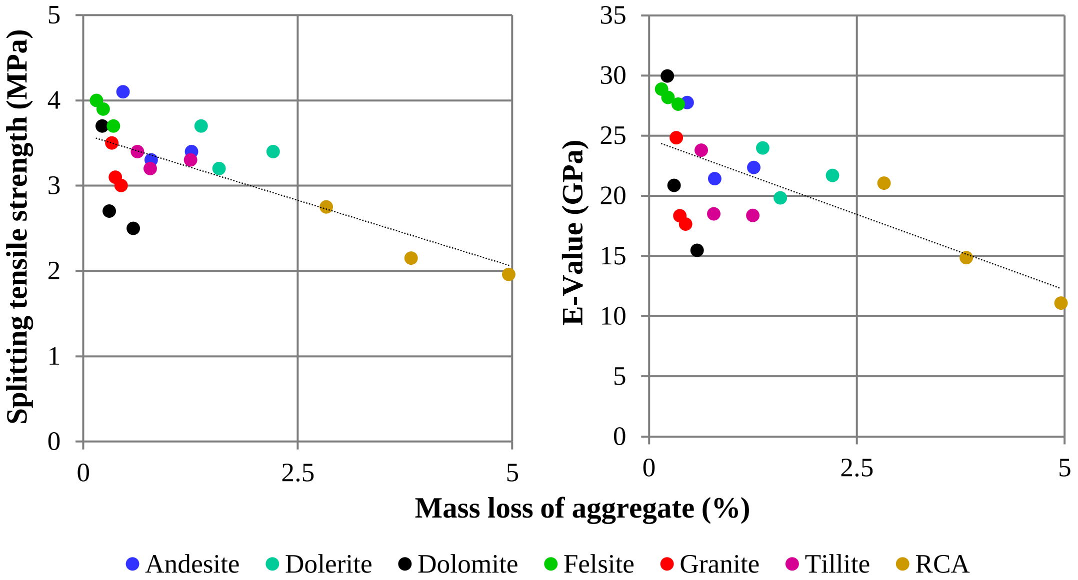

Figure 6-29: The effect of mass loss on a) tensile strength and b) stiffness of concrete

The effect of the CTE of concrete as well as the mass loss for the different aggregate types obtained from the TGA on the mass loss of the concrete specimen at different temperature ranges is illustrated in **Figure 6-30**. As mentioned, when concrete with a low thermal expansion is exposed to elevated temperatures, the mass loss is less significant (Kaplan, 1989). This coincides with the results obtained in this study. Concrete with lower CTE yielded lower mass loss results after exposure to elevated temperatures. Furthermore, the mass loss of the concrete increased as the mass loss of the aggregate obtained from the TGA (thermogravimetric analysis) increased.

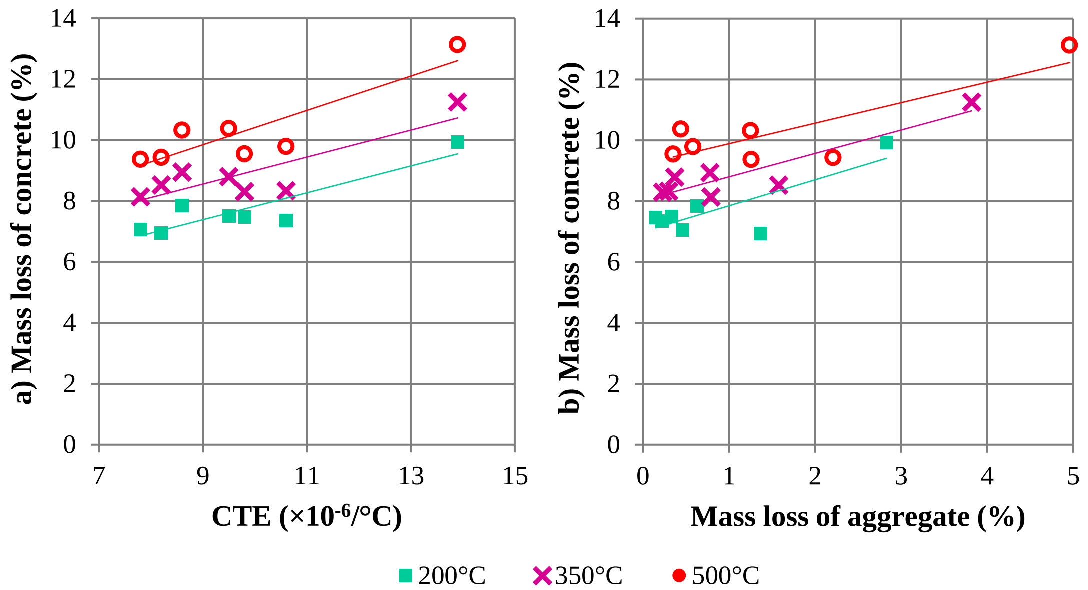

Figure 6-30: The effect of a) CTE of concrete and b) aggregate mass loss on the mass loss of concrete

6.8 SUMMARY

The experimental study highlights the importance of mixture constituents and proportions on the behaviour of concrete at elevated temperature. The maturity of concrete at the time of exposure, size and shape of the element and the maximum temperature reached also played a notable role. The relative compressive strength results indicate superior behaviour when concrete was exposed to elevated temperature at an early age (7-days after casting) compared to the performance after twenty-eight days of water curing. However, when considering the actual compressive strength at the various exposure temperatures, an insignificant difference was obtained. Furthermore, maturity of concrete at the time of exposure had a limited effect on the splitting tensile strength as well as modulus of elasticity. Based on the experimental study, it can be concluded that the addition of polypropylene fibres will not improve the behaviour of concrete after exposure to high temperatures.

Furthermore, both water/cement ratio and coarse aggregate content have a substantial influence on the behaviour of concrete when subjected to elevated temperatures. Reduced performance of concrete at elevated temperatures can be expected with the use of both lower water/cement ratios and higher coarse to total aggregate ratios. According to literature the loss of strength due to elevated temperatures tend to recover over time, not only with wetting, but also with storage at normal relative humidity, which correlates with the results obtained in the study. Autogenous recovery was observed with the use of higher water/cement ratios as well as lower coarse aggregate contents.

The influence of aggregate type on the properties of concrete is again evident in the results obtained, especially when considering concrete exposed to elevated temperatures. As mentioned previously, the degradation of concrete due to temperature exposure is caused by the shrinkage of the hardened cement paste and thermal expansion of the aggregates initiate stresses and cracking in the ITZ leading to rapid loss in strength and stiffness. When considering the residual compressive strength, splitting tensile strength, modulus of elasticity and mass loss the following order of preference of natural aggregate type for concrete exposed to elevated temperatures (up to $500°C$) is:

1) Dolerite
2) Felsite
3) Andesite
4) Tillite
5) Dolomite
6) Granite

The concrete containing fly ash showed higher strength deterioration compared to pure Portland cement concrete. The disruptive effects of the cement paste shrinkage opposed by aggregate expansion will be greater for concrete with a compact microstructure. The addition of SCMs, such as fly ash, in concrete mixtures usually enhance particle packing which produce concrete with a compact microstructure that, as mentioned, lead to rapid deterioration of strength at elevated temperatures. The results obtained in this study present contradicting trends to previously published literature that found that the addition of superplasticiser had little effect in the performance of concrete exposed to elevated temperatures. Concrete containing superplasticiser produced the highest strength and modulus of elasticity at elevated temperatures. Further testing is however required to establish the effect of superplasticiser on the performance of concrete exposed to elevated temperatures. Furthermore, RAC subjected to elevated temperatures performed equal to natural aggregate concrete, indicating that RAC can compete with concrete made with aggregates from conventional quarries. In conclusion, RAC can be utilised as a resource-saving alternative to preserve natural resources, decrease landfill waste, and encourage sustainability.

The study demonstrated that the loss of strength due to elevated temperatures recovered over time with wetting and it can be concluded with 95$\%$ confidence that concrete exposed to 350$°C$ retained more than 64$\%$ of its original strength, while concrete exposed to 500$°C$ retained more than 70$\%$ of its original strength after recovery. The tensile strength and modulus of elasticity was much more affected by microcrack development in the concrete due to elevated temperatures and it can be concluded that the modulus of elasticity is a function of the splitting tensile strength. Due to the large range of trends observed for the different variables considered, the results provided in Chapter 6 were combined to determine whether the effect of elevated temperatures on the concrete properties was predictable based on the measured aggregate properties. In this study it was established that the mass loss of aggregate obtained from the thermogravimetric analysis (TGA) might give an indication on the performance of concrete exposed to elevated temperatures, especially considering the dry compressive strength as well as the mass loss of the concrete. For each of the three exposure temperatures there seems to be a maximum compressive strength after exposure for aggregates with TGA mass loss in the region of 1$\%$ – 2$\%$. Aggregates with lower and higher mass loss yielded lower residual dry compressive strengths.

7 CONCLUSIONS AND RECOMMENDATIONS

7.1 CONCLUSIONS

The study highlights the notable influence of aggregate type on the performance of concrete subjected to elevated temperatures. According to literature, the degradation of concrete due to temperature exposure is caused by the shrinkage of the hardened cement paste and thermal expansion of the aggregates initiate stresses and cracking in the ITZ leading to rapid loss in strength and stiffness. The study proofed that concrete exposed to elevated temperatures in service should preferably contain aggregate with a low coefficient of thermal expansion. Concrete exposed to 350°*C* retained more than 64*%* of its original strength, while concrete exposed to 500°*C* retained more than 70*%* of its original strength after recovery as a result of rehydration when exposed to water. The tensile strength and modulus of elasticity was notably more affected by microcrack development in the concrete due to elevated temperatures. Similar reduction in strength and stiffness after exposure to elevated temperatures was observed for the splitting tensile strength and modulus of elasticity. When considering the compressive strength, splitting tensile strength, modulus of elasticity and mass loss the order of preference for use of natural aggregate type in concrete exposed to elevated temperatures (up to 500°*C*) is:

1) Felspathic (andesite, dolerite)
2) Granitic (granite, felsite)
3) Calcareous (dolomite)

Based on the experimental study, it can be concluded that high paste volumes (> 400 l/m^3) show notably more deterioration in strength after exposure to elevated temperatures and that mixtures with lower paste volumes show more autogenous recovery when tested saturated. It is therefore recommended that the use of concrete mixtures with excessive paste volumes or cement contents should be avoided. The use of SCMs, such as fly ash, showed higher strength deterioration compared to pure Portland cement concrete. This was attributed to the disruptive effects of the cement paste shrinkage opposed by aggregate expansion for concrete with a compact microstructure. RAC can compete with concrete made with aggregates from conventional quarries, not only under normal temperature conditions but also after exposure to high temperatures. Structural concrete can easily be manufactured where 100% coarse aggregate and 30% fine aggregate is replaced with RCA. RAC have the following properties in comparison to natural aggregate concrete:

- similar slump,
- reduced heat of hydration,
- lower density,
- reduced compressive strength,
- similar splitting tensile strength,
- lower modulus of elasticity,
- increased CTE,
- increased drying shrinkage, and
- equal potential durability.

Furthermore, the addition of superplasticiser resulted in improved behaviour after exposure to elevated temperature. The addition of superplasticiser produces a more uniform or homogeneous microstructure which might lead to improved behaviour at elevated temperatures. Further research is required to determine the influence of superplasticiser on the performance of concrete exposed to elevated temperatures.

It was hypothesised that the damaged caused by concrete exposed to high temperatures can be limited by using aggregate with a low thermal expansion coefficient and that aggregate that contain elements and minerals that decompose at relatively low temperatures, would place less stress on the surrounding cement paste, thus reducing the damage caused to the ITZ by the thermal expansion of the aggregate. It was established that the mass loss of aggregate obtained from the TGA might give an indication on the performance of concrete exposed to elevated temperatures, especially considering the dry compressive strength as well as the mass loss of the concrete. For each of the three exposure temperatures there seems to be a maximum compressive strength after exposure for aggregates with TGA mass loss in the region of 1% – 2%. Aggregates with lower and higher mass loss yielded lower residual dry compressive strengths. These results indicate that it would be possible to limit the extent of thermal damage to concrete by selecting aggregates with limited (at least 1%) but not excessive (less than 2%) mass loss at the exposure temperature. This study demonstrated that degradation of concrete due to temperature exposure is not only caused by the thermal expansion of the aggregates but also by the mass loss of aggregates. To limit the damage caused to concrete by exposure to elevated temperatures, there seems to be a balance required between the thermal expansion of the aggregate and the reduction in stress caused by the aggregate degradation as indicated by mass loss of the aggregate at the specific exposure temperature. Further research is however required to evaluate the results obtained for mass loss of aggregate from TGA against the behaviour after exposure to elevated temperatures with other aggregate types not used in this study.

7.2 RECOMMENDATIONS FOR FUTURE RESEARCH

The 49 different mixtures cast in this study give a good general trend regarding the high-temperature response that does add value to the body of knowledge. It is however recommended that concrete mixtures used in structural and non-structural elements subjected to thermal normal working conditions should be tested before use. The EN 1992-1-2 as well as the ACI 261.1-2007 standard can be used to predict the strength reduction of normal weight concrete with calcareous aggregates at elevated temperatures.

The South African National Standard (SANS 5865:1994) specify that cores should be stored in water for forty-eight hours before testing. The study demonstrated that the loss of strength and stiffness due to elevated temperatures ($500°C$) recovered over time with wetting and this is known as autogenous recovery. Portlandite decomposes at approximately $500°C$ and with the reintroduction of water after temperature exposure, the decomposed portlandite may rehydrate, causing a recovery in compressive strength. It is therefore recommended that specimen exposed to elevated temperatures should rather be tested in a dry state and not saturated as per standard procedures.

The following research aspects should be investigated to provide further insight into the behaviour of concrete exposed to elevated temperatures:

- Testing of RCA crushed from concrete containing other aggregates.
- Studying the effect of superplasticiser on the performance of concrete exposed to elevated temperatures.
- Evaluating the results obtained for mass loss of aggregate from TGA against the behaviour of concrete after exposure to elevated temperatures with other aggregate types not used in this study.
- Performing statistical modelling on the results obtained.
- Determining the effect of heating rates according to standards or real fire scenarios to assess potential performance in fire events.

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