Structural lightweight aerated concrete

By

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DECLARATION

I, the undersigned, hereby declare the work contained in this thesis is my own original work except where specifically referenced in text, and that I have not previously in its entirety or in part submitted it at any university for a degree.

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SYNOPSIS

Cellular concrete is a type of lightweight concrete that consists only of cement, water and sand with 20 per cent air by volume or more air entrained into the concrete. The two methods used for air entrainment in cellular concrete are (1) the use of an air entraining agent (AEA), and (2) the use of pre-formed foam. If pre-formed foam is used to entrain air into the concrete the concrete is named foamed concrete and if an AEA is used the concrete is termed aerated concrete. Depending on the type of application, structural or non-structural, cellular concrete can be designed to have a density in the range of range of 400 to 1800 kg/m^3. Non-structural applications of cellular concrete include void and trench filling, thermal and acoustic insulation. Structural applications of cellular concrete include pre-cast units such as concrete bricks, partitions, roof slabs etc. Due to the high levels of air in cellular concrete it is challenging to produce compressive strengths that are sufficient to classify the concrete as structurally useful when non-autoclaving curing conditions are used. The autoclaving process combines high temperature and pressure in the forming process, which causes higher strength and reduced shrinkage. This process is also limited to prefabricated units. Non-autoclave curing conditions include moist curing, dry curing, wrapping the concrete in plastic, etc. However, now that the world is moving in an energy efficient direction, ways to exclude energy-intensive autoclaving are sought. It has for instance been found that the utilisation of high volumes of fly-ash in cellular concrete leads to higher strengths which make it possible to classify the concrete as structurally useful. Now, that there is renewed interest in the structural applications of the concrete a design methodology using an arbitrary air entraining agent needs to be found. The research reported in this thesis therefore attempts to find such a methodology and to produce aerated concrete with a given density and strength that can be classified as structurally useful.

For the mix design methodology, the following factors are investigated: water demand of the mix, water demand of the mix constituents, and the amount of AEA needed to produce aerated concrete with a certain density. The water demand of the mix depends on the mix constituents and therefore a method is proposed to calculate the water demand of the mix constituents based on the ASTM flow turn table. Due to the complex nature of air entrainment in concrete, the amount of air entrained into the concrete mix is not known beforehand, and a trial and error method therefore had to be developed. The trial mixes were conducted in a small bakery mixer. From the trial mixes estimated dosages of AEA were found and concrete mixes were designed based on these mixes.

The factors that influence the mix design and strength of aerated concrete include filler/cement ratio (f/c), fly-ash/cement ratio (a/c) and design target density. Additional factors that influence the strength of aerated concrete are specimen size and shape, curing, and concrete age. It was found that the sand type and f/c ratio influence the water demand of the concrete mix. Sand type and f/c ratio also influence compressive strength, with higher strength for a finer sand type and lower f/c ratios. However, the concrete density is the factor that influences the strength the most.

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DEFINITION OF TERMS AND ACRONYMS

Cellular concrete	The term used to collectively describe aerated concrete and foamed concrete
Foamed concrete	Cellular concrete produced from pre-formed foam
Aerated concrete	Cellular concrete produced from an air-entraining agent
Foaming agent	A concrete admixture that is used to entrain air into the concrete either by adding the agent to the concrete mix or by adding the agent to a solution that can produce foam to be added to the concrete base mix. Typical foaming agents are manufactured from hydrolised proteins, vinsol resin, etc.
Air-entraining agent	A concrete admixture that is classified in the group of air-entraining agents in concrete engineering
Lightweight aggregate concrete	Concrete having a closed structure and a density if not more than 2200 kg/m^3 consisting of or containing a proportion of artificial or natural lightweight aggregates having a particle density less than 2000 kg/m^3 (BS EN 1992-1-1:2004, 185)
Structural lightweight aerated concrete	Aerated concrete having sufficient compressive strength, say 25 MPa or more, to be classified as structurally useful

LIST OF SYMBOLS

ρ_m	the target density in kg/m^3
ρ_{dry}	dry density in kg/m^3
$\rho_{measured}$	measured density in kg/m^3
x_c	cement content in kg/m^3
(f/c)	filler/cement ratio
(w/s*)	water/solids ratio
(w/s)	water/sand ratio
(w/c)	water/cement ratio
(w/a)	water/ash ratio
(s/c)	sand/cement ratio
(a/c)	fly-ash/cement ratio
RD_c	relative density of cement
RD_s	relative density of sand
RD_a	relative density of fly-ash
RD_f	relative density of foam
RD_{AEA}	relative density of air entraining agent
m_w	mass of the water in kg
m_s	mass of the sand in kg
m_a	mass of the fly-ash in kg
m_{AEA}	mass of the air-entraining agent in kg
m_{tot}	sum of the masses of the mix constituents in kg
V_w	volume of the water in litres
V_s	volume of the sand in litres
V_a	volume of the fly-ash in litres
V_{AEA}	volume of the air-entraining agent in litres
V_{tot}	sum of the volumes of the mix constituents in litres
$V_{a(req)}$	required volume of air to be entrained in the concrete base mix in litres
V_f	volume of foam in litres
σ_{comp}	compressive strength of the concrete in MPa
σ_{split}	tensile splitting strength of the concrete in MPa
E_c	modulus of elasticity of the concrete in GPa
σ_1	the compressive stress corresponding to 40 % ultimate load in MPa
σ_0	the compressive stress corresponding to 0.005 % strain in MPa
ε_1	strain corresponding to the compressive stress at 40 % ultimate load (mm/mm)
ε_0	0.005 % (mm/mm)

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CHAPTER 1 - PRELUDE

1.1 INTRODUCTION

In a world that is ever changing, nothing is too good to be improved upon. Design methods, codes, and design standards are constantly being updated to account for technological advancements in certain areas and engineers are heading the charge by constantly striving for efficiency in all aspects of engineering. When one strives towards efficiency, one uses minimum resources to make the impossible possible. For instance, building materials, in particular construction materials, are the foundation of civil engineering. Therefore, in order to be efficient one should use efficient construction material. As concrete constitutes a very large portion of a typical building. It is therefore reasonable to start looking at efficient construction material. Lightweight concrete is a type of concrete that can be considered as an energy efficient construction material. Some of the properties of lightweight concrete that make it an attractive alternative to normal density concrete are reduced weight, lower coefficient of thermal conductivity, the use of less materials, etc. These “improved” properties open a wide range of possible applications for the concrete. For instance, due to the lower coefficient of thermal conductivity the concrete may be used as an insulation material. If better insulation is achieved in buildings the use of the heating and cooling systems that utilise energy can be reduced significantly.

The study therefore aims to suggest that lightweight concrete is an energy efficient construction material. Various forms of lightweight concrete include lightweight aggregate concrete (LWAC), no “fines” concrete, and cellular concrete. LWAC is made from lightweight aggregate, no “fines” concrete is made by omitting the sand in the concrete mix resulting in a highly porous concrete and cellular concrete is made by omitting the coarse aggregate (stone) and entraining 20 per cent or more air by volume into the concrete mix. The focus of the investigation is on cellular concrete in particular aerated concrete which is a form of cellular concrete.

1.2 PROBLEM STATEMENT

Benefits of structural lightweight concrete for the concrete construction industry include low energy and reduced self-weight buildings. A simple way to produce cellular concrete is to add an air entraining agent (AEA) to the concrete mix. Due to the wide range of AEA’s available commercially, a design approach using an arbitrary AEA for producing aerated concrete with a certain density has to be developed. The purpose of the research can therefore be summarized as follows:

- Develop a suitable mix design procedure for producing aerated concrete
- produce an energy efficient lightweight concrete of sufficient strength for structural use
- produce compressive strengths in the range of 25 and 30 MPa
- produce concrete density in the range of 1600 kg/m^3 – 1920 kg/m^3

1.3 OUTLINE

In chapter 2, literature review, the topic of air-entrainment in concrete is introduced. The methods of air-entrainment, mechanisms of air-entrainment, and factors affecting air-entrainment in concrete are also discussed.

In chapter 3, mix design, the mix design procedure for aerated concrete is discussed. An overview of the factors influencing the mix design procedure is discussed as well as the mix design considerations.

In chapter 4, experimental design, the details of the experimental design are given.

In chapter 5, results and discussion, the results of the experiments performed on the concrete are presented and discussed. The conclusions and recommendations are given in chapter 6.

CHAPTER 2 - LITERATURE REVIEW

2.1 INTRODUCTION

Air entrainment is the process whereby many small air bubbles are incorporated into concrete and become part of the matrix that binds the aggregate together in the hardened concrete. In America, this process was accidentally discovered in the 1930s when it was observed that certain concrete pavements were more durable and frost damage resistant than other concretes (Dodson (1990)). A further study revealed that the cement used for these concrete pavements was manufactured with grinding aids, which acted as an air entrainer. After the discovery of air entrainment in concrete, research was dedicated into the effects of air entrainment in concrete. This gave rise to the development of air entraining agents (AEA). Research into air entrainment in concrete revealed that some of the benefits include improved workability of the concrete mix in the fresh state, reduced bleeding, increase in the yield, and improved durability of concrete to frost damage.

The most important reason for air entrainment in concrete is for durability purposes. The inclusion of air voids in the concrete, especially when the concrete can be subjected to freezing conditions, provides resistance to freeze-thaw cycles. This topic was investigated by many researchers but the work of Powers was considered significant in this field. Powers (1954) proposed a procedure for mix designing of air entrained concrete based on the spacing factor. The spacing factor can be considered as the average distance between air voids in the cement paste. The bases of the design method rested on Powers' belief in the existence of a "thin" shell around an air void in the cement paste in which expansion cannot take place and if these shells were to overlap the paste would be protected against expansion. This inability of the paste to expand ensures freeze-thaw resistance.

Another reason for air entrainment in concrete is the fact that certain properties can be altered to suit certain purposes. An example of this is the production of cellular concrete. Cellular concrete is produced when the coarse aggregate is omitted from normal concrete and entraining 20 per cent or more air by volume into the concrete. The advantages of cellular concrete include a low thermal coefficient of thermal conductivity and a low self-weight. Cellular concrete is classified as lightweight concrete and is normally not used for its structural resistance but rather for its low self-weight and low thermal conductivity compared to normal concrete. Cellular concrete has a low compressive strength when wet or dry curing is used. Hence, the use of cellular concrete has been limited to non-structural applications.

However, if cellular concrete is subjected to autoclaving, the concrete yields sufficient compressive strength to be used for structural applications. Autoclaving is a process whereby the concrete is cured at high temperature and pressure in a chamber called the autoclave. Unfortunately autoclaving can only be done in laboratories and factories, thus, the structural applications of cellular concrete are confined to precast units.

Short and Kinniburgh (1963) and Rudnai (1963) discussed the uses of cellular concrete in detail. The common theme between the different authors is that the uses of cellular concrete can be classified into two categories: (1) in-situ and (2) precast. For in-situ application cellular concrete is normally used as an insulating material for roofs, pipelines and cold stores. For the second case, precast units are made from cellular concrete. Typical precast units are floor slabs, roof slabs, partitions, and cladding. These units can be reinforced with steel but the steel should be protected against deterioration.

Normally when non-autoclaving curing conditions are used the concrete yields compressive strengths that make it impossible for the concrete to be classified as structural but now that the world is moving in an energy efficient direction, and is looking for alternative ways to dispose/use waste products, large utilization of powder fly-ash, a waste product generated by form coal-fired power station, has become popular as a filler replacement in the concrete mix. Utilization of fly-ash in cellular concrete makes it possible for the concrete to yield sufficient strength to be classified as structural. The fact that fly-ash increases the strength of cellular concrete is rather counterintuitive because the addition of fly-ash in normal weight concrete decreases the early age strength of the concrete. Two reasons exist for this increase in concrete strength: (1) the particle size of the fly-ash is small and it has been reported by numerous authors that the use of fine filler yields greater concrete strength to density ratios, (2) the fly-ash reacts with the products of cement hydration and forms products with cementing properties. Also, another observation made by various authors (i.e. Kearsley and Wainwright (2001) and Jones et al (2003)) is that the addition of fly-ash increases the rate of strength development over longer periods.

Now that there is renewed interest in the structural ability of cellular concrete a mix design methodology needs to be developed for aerated concrete made with an arbitrary air entraining agent. Therefore, the literature review is structured so as to give the reader background knowledge on how air entrainment is achieved in cellular concrete, mechanisms that govern air entrainment and factors that affect the amount of air entrained in concrete. The literature review then concludes with a brief overview of the uses of cellular concrete in the industry.

2.2 AIR ENTRAINMENT

Air entrainment is the process whereby many small air bubbles are incorporated into concrete and become part of the matrix that binds the aggregate together in the hardened concrete (Ramachandran, 1984, p.269). The air bubbles are dispersed throughout the hardened cement paste but are not, part of the paste. Air entrainment may be accomplished by the use of an air-entraining agent (AEA) that forms part of the group called concrete admixtures. Air entrainment can also be achieved through the use of pre-formed foam, which is added to the mix slurry. However, the use of pre-foamed foam is specialised and is commonly used only in the production of foamed concrete. Foamed concrete forms part of the group of cellular concretes and is the term used to describe cellular concrete that is made from pre-formed foam. AEAs can be used to entrain air into normal weight concrete and it can be used to produce aerated concrete. Aerated concrete also forms part of the group of concrete called cellular concrete. The term aerated concrete is used to describe cellular concrete that is made using an AEA. A detailed discussion of the use of pre-formed foam and an AEA to produce foamed concrete and aerated concrete is given in the following sections.

2.2.1 AIR ENTRAINING AGENTS

In order to understand the process of air entrainment it is vital that one should be familiar with the composition, properties, and behaviour of air entraining agents. Also, in order to manipulate the air entrainment process, the mechanisms, concepts and theories encompassing air entrainment must be understood. Therefore, the section starts by defining air entraining agents on the microscopic level, and then explains the composition, behaviour and properties of air entraining agents, after which the mechanisms of air entrainment are discussed and air bubble stabilization is explained.

2.2.1.1 PROPERTIES OF SURFACTANTS

Nearly all modern day air entraining agents are part of the chemical group called surfactants. A surfactant is a material whose molecules are adsorbed strongly at the air-water or solid-water interfaces. These molecules are therefore partially attracted from the solution phase and strongly concentrated at the interfaces. Du and Folliard (2005) describe the chemical unit as having a hydrophilic head, that has a strong attraction to the solvent, and a hydrophobic tail, which has little attraction to water as depicted in Figure 1. The head can be one of three things; (1) if it is positively charged the air entraining agent is cationic, (2) if the head is neutral the air entraining is nonionic and (3) if it is negatively charged the air entraining agent is anionic. Some AEA have been reported to perform better than others. A reason for this is because of the charge on the head of the surfactant. For example anionic type air entraining agents precipitate insoluble salts that help in the air bubble stabilization process.

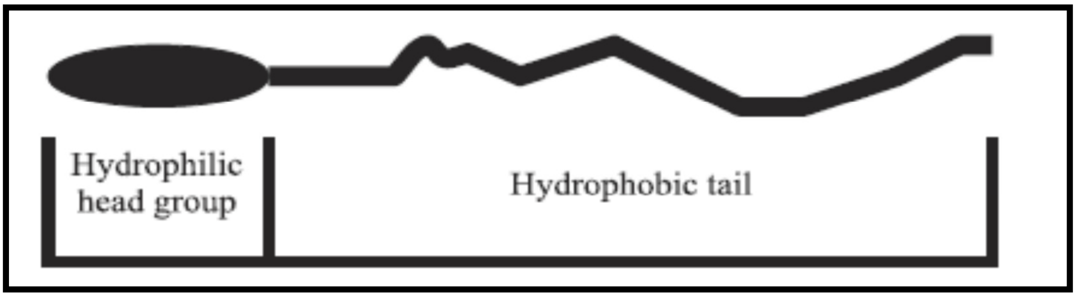

Figure 1: The basic chemical nature of surfactants (Myers, 1992)

2.2.1.2 TYPES OF AIR ENTRAINING AGENTS

Zhang (1996) summarised the eight generic types of air entraining agents used in the UK according to BS 5075: Part 2: 1982. These categories may be presumed to be similar in the rest of the world.

1. Blend of vinsol resin and synthetic surfactants (A1)
2. Protenation surfactant (A2)
3. Vinsol resin (A3)
4. Sulphonated hydrocarbon (A4)
5. Vinsol resin (A5)
6. Epoxy sulphate (A6)
7. Modified fatty acid (A7)
8. Salt and fatty acid (A8)

2.2.1.3 MECHANISMS OF AIR ENTRAINMENT

According to Ramachandran (1984) the air bubbles in concrete are generated by the mixing action and that all the air entraining agent does is stabilize the bubbles that form and not generate them. Since air entraining agents only stabilize the air bubbles that are already present in the concrete the questions arise as to the origins of air in concrete. Mielenz et al (1958) investigated this and proposed four different sources: (1) air already present in intergranular spaces in the cement and aggregate; (2) air originally present within the particles of cement and aggregate but expelled from the particles before hardening of the concrete by inward movement of water under hydraulic and capillary potential; (3) air originally dissolved in the mixing water; (4) air which is in-folded and mechanically enveloped within the concrete during mixing and placing.

Powers (1968) discussed how air is included in the plastic concrete during placing. He noted that there are two different processes at work. One is the infolding of air by essentially a vortex action, like the stirring of any liquid. The air is drawn into the vortex and then dispersed and broken up into smaller bubbles by the shearing action. The second process is called the “three dimensional screen” which involves the aggregate. The aggregate traps and holds air bubbles within its network of particles as masses fall and cascade on each other during the mixing.

Another mechanism of air-entrainment that was proposed by Mielenz (1969) is the dissolution of air from bubbles and a diffusional transfer from one bubble to another bubble. In order to demonstrate the mechanism Mielenz et al (1958) tracked two air bubbles over time by photographing the bubbles at different times. The photographs show the smaller bubble reducing in size over time and the bigger bubble increasing in radius.

2.2.1.4 STABILIZATION OF AIR BUBBLES

The function of an air entraining agent is to stabilize the air bubbles that form in the concrete. In the absence of an air entraining agent, consolidation of the concrete will cause most of the air bubbles to make their way to the surface of the concrete and burst. Only a small amount cannot escape. This small amount of air is termed “entrapped air”. However, if an air entraining agent is used, the air in the concrete is stabilised by one or more actions of the air entraining agent. Figure 2 shows the distribution of surfactants at the air water interface.

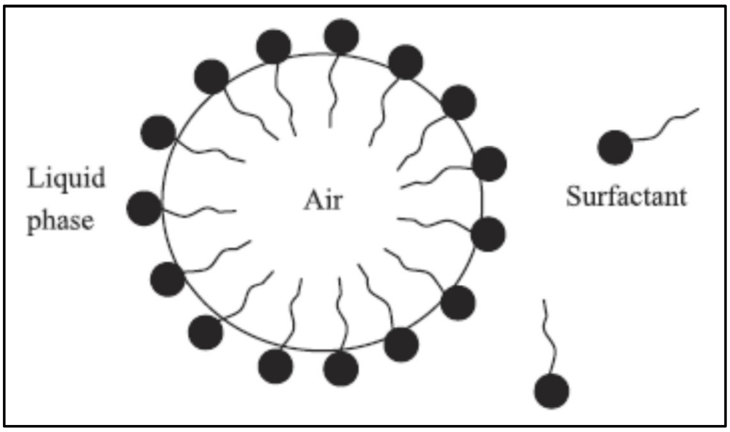

Figure 2: Distribution of surfactant molecules at the air-water interface (Du and Folliard, 2005)

When a bubble is formed in the mix in the presence of an air entraining agent the surfactants are concentrated at the air-water interphase. The surfactants orientate themselves so that they can satisfy both parts of the molecule, the hydrophilic head is found on the surface of the interface in the liquid phase and the hydrophobic tail is found in the air phase. The hydrophobic tail consists mainly of a hydrocarbon chain and when the surfactant is concentrated at the air-water interface the concentration of the surfactant increases, whereby the surface tension is reduced. Eventually a limit is reached whereby no further increase in surfactant concentration and decrease in surface tension can take place. When this limit is reached a new structure forms call a micelle. Du and Folliard (2005) attributed this property of air entraining agents to the fact that there is a limit to the percentage of air that can be entrained in concrete. Stated differently, there is a “critical” dosage whereby more air entraining agent will have negligible effect.

The mechanisms that may lead to collapse of air bubbles in foam were discussed by Myers (1999) and can be summarized as follows:

1. Diffusion of air from a small air bubble to a larger air bubble (large air bubbles have a lower internal pressure than smaller bubbles)
2. Bubble coalescence due to capillary flow
3. Rapid hydrodynamic drainage of liquid between bubbles leading to rapid collapse

These mechanisms can apply to air being stabilized in concrete containing an air entraining agent. Mechanism 2 is similar to the mechanism proposed by Mielenz (1969). If the air entraining agent is cationic or anionic in nature the hydrophilic head is charged positively or negatively. The surfaces of all the air bubbles have the same charge and repel each other during mixing. Therefore, the bubbles do not come into contact with each other and coalescence is prevented. It is also worthwhile to note that this property of air entraining agents causes an increase in workability of the concrete mix. The surfactants in air entraining agents are also adsorbed onto the cement particles and on the aggregate, and this adhesion prevents the air bubble from floating to the surface. Evidence of air bubbles adhering to cement particles was found by Mielenz et al (1958) when they observed the phenomenon. The mechanism is shown in Figure 3.

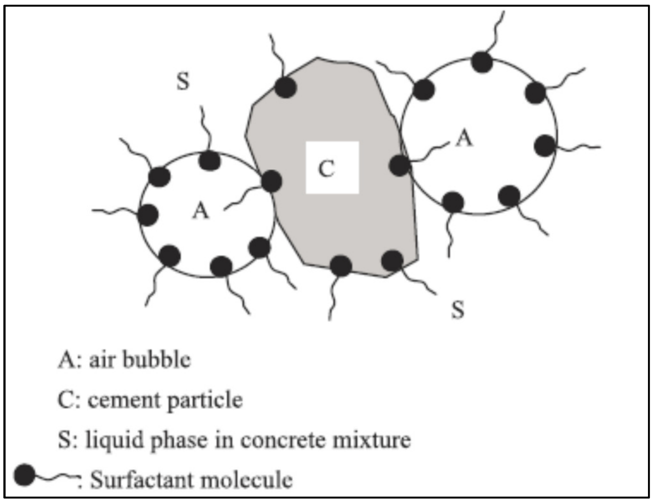

Figure 3: Interaction between air bubbles and cement particles (Du and Folliard, 2005)

2.2.2 FOAMING AGENTS

Another way to incorporate air into a concrete mix is via foam. Revision of previous literature indicates that this method is strictly confined to the production of foamed concrete. However, it is worthwhile noting that a combination of air entraining agents can be used in conjunction with foaming agents or pre-formed foam to produce cellular concrete. With foaming agents air entrainment is achieved in concrete via pre-formed foam. According to Rudnai (1963) the foaming agent is dissolved in water and whipped in a mechanical machine or foamed by compressed air and then added to the mortar mix. The following types of foaming agents are in common use today in the production of foamed concrete: (1) glue-resin foaming agent, (2) saponated resin foaming agent, (3) alumosulphonate foaming agent, (4) hydrolysed blood foaming agent and (5) hydrolysed keratin foaming agent. These are not further discussed here, as the focus of this study is on air entraining agents.

2.2.3 FACTORS INFLUENCING THE AMOUNT OF AIR ENTRAINED

Air entrainment in concrete is an extremely complex topic which is influenced by a number of factors that are not all mutually exclusive, thus making it difficult to quantify each parameter.

2.2.3.1 TYPE AND DOSAGE OF AIR ENTRAINING AGENT

In any concrete mix the dosage and type of air entraining agent used affects the amount of air entrained into the concrete. According to Chatterji (2003) air entraining agents can be classified into two groups. In the first group the air entraining agents reacts with the calcium hydroxide of the cement paste to precipitate insoluble salts. These salts are hydrophobic and collect at the water-air-cement grain contact regions, which is the main cause of air

entrainment and bubble stability. These air entraining agents include vinsol resin (A3), sodium adipate (A8), sodium oleate (A8), etc.
The second group are the synthetic tendites or surfactants. This group consist of chains of aliphatic and/or aromatic hydrocarbons (A7) with a water soluble group like SO_4 or SO_3 or OH etc, attached at one end. The calcium salts precipitated from this group are soluble in water. The surfactants collect at the air-water interface and lower the surface tension, this is the main cause of air entrainment and stability.

It can be noted that with an increase in dosage of air entraining agent there is an increase in the concentration of the agent in the mix. For group one air entraining agents there will be an increase in the precipitants, which will increase the amount of air entrained in the concrete. For group two air entraining agents the same argument can be raised. However, Du and Folliard (2005) noted that there is a minimum dosage of air entraining agent required to entrain air in the concrete. There exist low dosages of air entraining agents that do not lower the surface tension of water and hence no air is entrained into the concrete mix. There also exists a dosage at which no further increase in concentration of the surfactant at the air-water interface is possible, thus no further reduction in surface tension can take place. For this reason there is a maximum amount of air that can be entrained into concrete.

2.2.3.2 BATCH VOLUMES AND MIXING

The mixing process and the batch volumes also affect the amount of air entrained into the concrete. The mixing process is influenced by the type of mixer used. This includes the speed at which the mixer operates, volume capacity of the mixer and the mixing action employed by the mixer. It is known that the volume of the mix constituents affect the efficiency of the mixer. For example, mixing low volumes of concrete in a mixer that has a high capacity can result in improper mixing of the mix constituents in pan mixers. Rixon (1978) investigated the influence of batch volumes by varying the mix volume as a percentage of the capacity of the mixer. From the data the author concluded that the batch volume only slightly affected the air content in the mixes. However, it should be noted that these tests were performed on concrete with low air content in the range of 4 to 6 % and that this conclusion cannot be stretched for cellular concrete where the norm is to entrain 20 % or more air by volume in the concrete mix.

It can be observed that for an increase in mixing time there is an increase in air entrainment in concrete. However, after the maximum amount of air is entrained into the concrete there is a drop in air content with prolonged mixing time. Ramachandran (1984) suggested that a reason for this decrease in air content could be related to the decrease in slump of the concrete that also occurs with prolonged mixing. The author also reported that worn mixer blades or a build-up of hardened concrete on the blades and overloading the mixer results in a decrease in the air content.

2.2.3.3 CEMENT

The physical and chemical properties of the cement and cementitious material also play a major role in the air entrainment process. For the cement, the fineness, cement content and the alkali content are the factors to be considered. The investigation by Rixon (1978) into the fineness of the cement found that decreasing the fineness of the cement could lead to doubling the air entraining agents' dosage to achieve the same amount of air content. The converse is also true. The air content of the mix was found to decrease with increasing cement content. Du and Folliard (2005) offered explanations for the two observations made. In the case of an increase in the fineness of the cement there is an increase in the surface area which leads to the fact that more of the air entraining agent is adsorbed onto the solid surfaces and less is available for bubble formations and stabilization. Also, an increase in the fineness of the cement increases the hydration rate which also leads to a decrease in the amount of air entrained into the concrete because of temperature increases.

2.2.3.4 AGGREGATES

Aggregate in concrete is divided into two categories according to size. Coarse aggregate, stone, is defined to be any aggregate that cannot pass through a sieve with square 4.75 mm openings. Fine aggregate, sand, is defined to be any aggregate of which all particles pass through a sieve with square 4.75 mm openings. Although aggregate is inert in the concrete mix it does affect the fresh state properties and the hardened state properties of the concrete. The properties such as surface texture, shape, size, grading, and density of the aggregate plays a role in air entrainment in concrete. The volume of the aggregate also influences the air entraining process.

Coarse aggregate affects the size and the distribution of bubbles in the concrete by influencing the shearing and impact actions in the mixing phase. If the shape of the aggregate is flaky, the aggregate would be more prone to 'entrap' air underneath it during the mix. However, most of this is likely to be expelled from the concrete during consolidation. The typical size of entrapped air bubbles expelled from the concrete is in the order of 1 mm and greater. Another way in which coarse aggregate can influence air entrainment is when crusher dust is used. This is only possible if the crusher dust is used in large quantities so that it can influence the grading of the fine aggregate. This was highlighted by Rixon (1978).

Fine aggregate provides the three dimensional screen affect and traps the air bubbles in it. It provides space for the cement paste, air bubbles, and the air entraining agent to stabilize the air bubble. Various sources such as Mielenz et al (1958), Powers (1968) and Folliard (2005) report that the grading of the fine aggregate affects the amount of air entrained into the concrete. The sources indicate that the optimum range of sand size for air entrainment lies between 150 and 600 micron. It therefore stands to reason that sand with a low fineness modulus would perform better than sand with a high fineness modulus. A typical size range of entrained air is 0.1 to 1 mm with a large percentage under 0.3 mm Owens (2009).

2.2.3.5 TEMPERATURE

The temperature of concrete has been found to influence the effectiveness of an air entraining agent. The air content of concrete varies inversely with temperature Proportioning Concrete mixes (1974). It was reported from a series of laboratory and field tests that the air content at 10 ˚C was approximately 30 % greater than the air content at 21 °C. Dodson (1990) reports a similar observation that the air content at 21 °C may be 25 % more than the air content at 38 °C and that 40 % more air can be entrained at 4 °C than at 21 °C. Du and Folliard (2005) attributed this to the fact that at low temperatures the yield stress and viscosity of concrete is increased because the viscosity of the water is higher at low temperatures reducing the hydration products at a very early age. Therefore more air entraining agent is available for foaming and that the effectiveness of the agent is improved in the absence of high calcium concentrations.

2.2.3.6 CEMENT EXTENDERS

Cement extenders and replacers which include pozzolan materials such as ground granulated blastfurnace slag (GGBS), fly-ash (FA), and condensed silica fume (CSF) have also been reported to affect air entrainment in concrete. These materials have cementing properties when used with cement, increase durability of the concrete, reduce the cost of the concrete and may improve workability. Earlier work had shown that these materials affect the amount of dosage of the air entraining agent required. Zhang (1996) launched an investigation into the effect of powdered fly ash (PFA) on air entrainment in fresh concrete with the eight different types of air entraining agents. The three areas that Zhang (1996) focussed on was (1) the effect of PFA on the required dosage of AEA for a given air content; (2) the effect of PFA on air content stability; and (3) the air loss in air entrained PFA concrete.

For the case of dosage Zhang (1996) found that for a given PFA level the amount of air entraining agent used two to six times that required for the neat OPC concrete mix and that air entraining agents based on surfactants and vinsol resin were at the lower end of this range.

2.2.3.7 OTHER CHEMICAL ADMIXTURES

Other chemical admixtures in concrete include plasticizer, superplasticizer, accelerators and retarders. The interaction between the air entraining agent and any other admixture is difficult to predict without observing the behaviour of the two admixtures together in a concrete mix. For example, some (super) plasticizers act as air entraining agents as well, when this is the case the dosage of the air entraining agent should be lowered to correct for the additional air entrained in the mix or a defoaming agent may be used.

2.3 CELLULAR CONCRETE

2.3.1 INTRODUCTION TO CELLULAR CONCRETES

According to Neville (1963) one means of obtaining lightweight concrete is by the introduction of gas bubbles in the plastic mortar mix in order to produce a material with a cellular structure, similar to a sponge rubber. The resulting concrete is known as cellular concrete. Two methods of producing aeration have already been introduced in this thesis, and depending on the method used, the product is most likely to be named after it. The two methods are (1) use of an air entraining agent, (2) use of pre-formed foam. If an air entraining agent is used the concrete will most likely be named aerated concrete and in the case of pre-formed foam the name will be foamed concrete. A further differentiation is made to cellular concrete based on the curing method employed. Note that, as in the case of an air entraining agent, a foaming agent may also be added to the mix, as opposed to pre-formed foam. These agents may be in powder or liquid form. They differ from the AEA's in that they add air, and do not merely capture / stabilise already trapped air in the mix.

Generally, there are various methods of producing aeration in concrete. However all of these methods can be categorized into two categories; (1) controlled air content and (2) uncontrolled air content. In the controlled air content category a method that produces an air content of known quantity is used. Foamed concrete is an example of a controlled air method because a known quantity of pre-formed foam is used to make foamed concrete. The foam is produced using a solution of a foaming agent with water and whipping the foam until a certain density is obtained. Valore (1954) reported this method of producing voids in the concrete to be the most economical and controllable because there are no chemical reactions involved. The uncontrolled air content method, where an unknown quantity of air is produced, produces aerated concrete. In this method chemicals are used to produce gas bubbles that are “trapped” in the mix, thus producing a porous matrix. A disadvantage of this method is the fact that the amount of air entrained into the concrete mix is unknown and it is only through observation and tests that one can estimate the volume of air entrained into the concrete mix.

2.3.2 MIX DESIGN OF CELLULAR CONCRETES

Cellular concrete is different to normal weight concrete in the sense that it contains no coarse aggregate. Therefore the design procedures for proportioning a mix that applies to normal weight concrete cannot be applied to any form of cellular concrete. Throughout the literature, different authors such as McCormick (1967), Valore (1954) and Rudnai (1963) have used mix design methodologies for cellular concrete which vary vastly from that of normal concrete. These methodologies all have the following in common; the mass of the mix constituents is expressed as a factor of the cement content by weight.

As stated by Kearsley and Mostert (2005), in the mix design phase of normal weight concrete the water/cement (w/c) ratio dictates the design. This w/c ratio gives an indication of the strength of the concrete at 28 days. However, in the design of foamed concrete the target density is determined and the w/c ratio and sand/cement (s/c) ratios are chosen. In the mix design of foamed concrete, two variables need to be solved, namely the (1) cement content and (2) the foam content. For this purpose two equations can be presented, one based on the sum of the mass of the constituents and the other based the sum of the volume of the mix constituents. The two equations presented below by the authors summarize the mix design methodology for foamed concrete.

$$\rho_m = x_c + x_c(w/c) + x_c(a/c) + x_c(s/c) + x_c(a/c)(w/a) + x_c(s/c)(w/s) + RD_f V_f \qquad (2.1)$$

$$1000 = x_c/RD_c + x_c(w/c) + x_c(a/c)/RD_a + x_c(s/c)/RD_s + x_c(a/c)(w/a) + x_c(s/c)(w/s) + V_f \qquad (2.2)$$

where

ρ_m = the target density in kg/m^3

x_c = cement content in kg/m^3

(f/c) = filler/cement ratio

(w/s) = water/solids ratio

(s/c) = sand/cement ratio

(a/c) = fly-ash/cement ratio

RD_c = relative density of cement

RD_s = relative density of sand

RD_a = relative density of fly-ash

RD_f = relative density of the foam

V_f = volume of the foam (l)

2.3.3 CURING

Cellular concrete can also be classified according to the method of curing used. There are several curing methods that are commonly used for cellular concrete. These methods can be classified into two groups; (1) autoclaved; (2) non-autoclaved. Combining these classification methods (based on pore formation, based on curing) the type of cellular concrete one can obtain is non-autoclaved aerated (foamed) concrete and autoclaved aerated (foamed) concrete. A brief overview of the curing methods is now presented.

2.3.3.1 AUTOCLAVED

Aerated concrete can be non-autoclaved (NAAC) or autoclaved (AAC) based on the method of curing. Autoclaving is a process whereby the concrete is cured in a chamber with high temperature and high pressure for a certain amount of time. According to Narayanan and Ramamurthy (2000) a wide range in the pressure (4 - 16 MPa) and duration (8 - 16 hours) of the autoclaving process may be used. Furthermore, they reported that autoclaving reduces the drying shrinkage in aerated concrete significantly, and it is essential if aerated concrete products are required within acceptable levels of strength and shrinkage.

2.3.3.2 NON AUTOCLAVED

Non autoclaved curing processes include moist curing, steam curing, and dry curing. Although these curing methods can be applied to cellular concrete they are mostly not favoured because the strength of the resulting concrete is low compared to that of autoclaving. However, autoclaving requires factory production and if cellular concrete is to move forward as a structural material in the industry alternative forms of high strength development in cellular concrete would have to be developed.

2.3.4 USES OF CELLULAR CONCRETE

Although Americans claim to have discovered air entrainment in concrete in the late 1930's (Ramachandran, 1984) it should be noted that aerated concrete was first produced in about 1929 (Short and Kinniburgh, 1963). At that stage cellular concrete was being manufactured and used in the building industry as blocks. The concrete, though not known for its strength, has been used for other attractive properties. These properties include the low self-weight of the concrete and a low coefficient of thermal conductivity. Cellular concrete can be made so that it can have a density in the range of 400 - 1800 kg/m^3. In the design of buildings, concrete with a low density would be favourable since the low density would provide a low self-weight component. The thermal properties of concrete are also of importance these days since the world is moving towards an energy efficient direction. A low coefficient of thermal conductivity would prove useful as it would be better equipped than normal concrete to protect the reinforcement in the case of a fire. The cellular concrete would also provide better insulation than normal weight concrete.

In the concrete industry, advantages of low self-weight and good thermal properties of cellular concrete are overshadowed by the low compressive strength the concrete yields and drying shrinkage (depending on the type of curing regime employed). It has been proven that a decrease in concrete density gives rise to a considerably large drop in compressive strength. This has made cellular concrete less considerable for structural use. However, if autoclaving is used, the resulting concrete has adequate strength to be considered for use as structural elements. Autoclaving also eliminates the problem of drying shrinkage. It has been reported by Short and Kinniburgh (1963) that the drying shrinkage of cellular concrete that has been autoclaved is between one quarter and one fifth of the drying shrinkage of air-dry cured cellular concrete. Also, if drying shrinkage is a problem, shrinkage reducing agents can be employed to decrease the adverse effect of drying shrinkage in the concrete.

Since autoclaving can only be done in the factories, cellular concrete in the concrete building industry has been restricted to precast units. These precast units are normally rectangular in shape and are cut with high precision to meet size specifications. In the case of floors, wall slabs and roof slabs, reinforcement can also be provided. Grooves are normally present in the slabs for ease of alignment during construction.

CHAPTER 3 - MIX DESIGN

3.1 INTRODUCTION

Concrete mix design can be described as the art of selecting different materials such as cement, water, aggregates and additives in order to achieve the required physical properties of the final product. The two main physical properties for normal weight concrete are the slump and the characteristic compressive strength of the concrete. The slump of the concrete is an indicator of the workability of the concrete mix and the characteristic compressive strength is taken as the compressive strength of the concrete at the age of 28 days. These two properties are normally specified for concrete mix designers of normal weight concrete and a mix is proportioned in order to meet the requirements. However, there exist different types of concrete and the physical properties of the concrete specified for the mix designers to achieve are not necessarily just the slump and the characteristic strength of the concrete. An example of this would be the design of structural lightweight aggregate concrete. For this type of concrete, the concrete density, slump and characteristic strength is specified and this requirement has to be met by the designers.

In the mix design of normal weight concrete, the strength of the concrete and the workability of the mix are what govern the design. In the early 1900s, it was discovered that the ratio of the total water content of the mix to the cement content of fully compacted concrete was the factor that controlled the strength of the concrete. Therefore, today design charts give the estimated strength of the concrete mix at 28 days based on the water/cement ratio and cement type used. Other ingredients such as the stone (coarse aggregate) and sand (fine aggregate) are considered to be inert and only provide bulk to the concrete. The workability of the mix is affected by the amount of water in the concrete mix. If there is little water in the mix the concrete is stiff and hard to work. Too much water in the mix would result in segregation of the mix. The workability of a normal concrete mix is usually measured by performing a slump test. Besides workability, this test gives an indication of the consistency of the mix as well. Based on this test a mix may be accepted or rejected. In the latter case, the water content of the mix is adjusted accordingly until the workability requirement is satisfied.

For lightweight concrete, Short and Kinniburgh (1978) list: concrete compressive strength, workability and relative density as the three performance requirements that must be met in the mix design procedure. In the mix design process for lightweight concretes it is important to note that the properties of the aggregates play a major role in mix proportioning. For example, it is well known that because of the aggregates and mix constituents, certain lightweight concretes can only have a certain density range and therefore can only achieve concrete strengths in a certain range because the strength of lightweight concretes depends on the density of the concrete.

In summary, for the different types of concrete there exist different techniques used for mix proportioning and before tackling the mix design it is important to know how the different mix constituents affect these techniques. This section discusses the factors that influence the mix design procedure, such as the water demand of the mix which is greatly influenced by the filler content, filler/cement (f/c) ratio, density and choice of binder for aerated concrete and proposes a methodology of design.

3.2 WATER DEMAND OF THE MIX CONSTITUENTS

In cellular concrete, the target density is obtained by entraining a certain volume of air into the concrete mix. In foamed concrete, where pre-formed foam is used, the target density is usually achieved with great accuracy, usually within a range of ±25 - 50 kg/m^3. However, for this accuracy, certain requirements have to be met. The main factor that is responsible for stability of foamed concrete in the fresh state is the water content in the concrete base mix. A stable mix will therefore be able to produce a foamed concrete mix where the ratio of the design target density and wet/plastic density is close to unity. However, there are only a few literatures published on the fresh state properties of foamed concrete. As a result, there is no uniformity in methodologies to evaluate the fresh properties in foamed concrete. To evaluate the fresh state properties in foamed concrete Kearsley and Mostert (2005) used the ASTM flow table test for hydraulic cements for measuring spreadability, and Jones et al (2003) used the Brewer spread test and the flow test as per BS 4551-1. Furthermore, Kearsley and Mostert (2005) proposed a mix design procedure that yielded foamed concrete densities within 5 % of its design density. The reason for the degree of accuracy is that the water demand of the mix constituents was satisfied. It is evident now that the water content of the mix is the most important factor to be considered in the mix design process of cellular concrete. Therefore an approach for aerated concrete design similar to that of Kearsley and Mostert (2005) shall be followed. But first, a brief overview of the properties of the mix constituents follows.

3.2.1 MATERIALS

The choice of binder used for the investigation according to SANS 50197-1:2000 was Ordinary Portland Cement CEM I 52.5 N with a relative density of 3.14. Locally available sand was used as filler. The two fillers used were Malmesbury sand (M) and Phillippi sand (P) both with a relative density of 2.65 as determined from SANS 5844:2006 and fineness modulus (FM) of 2.21 and 1.45 respectively. Sand with a low FM indicates that the sand mainly consists of fine particles. On the other hand, a high FM indicates a high degree of coarse material in the sand. It has been observed that sands with low FM have a higher water requirement than those with high FM. Figure 4 illustrates the particle distribution of the two filler types used throughout the investigation.

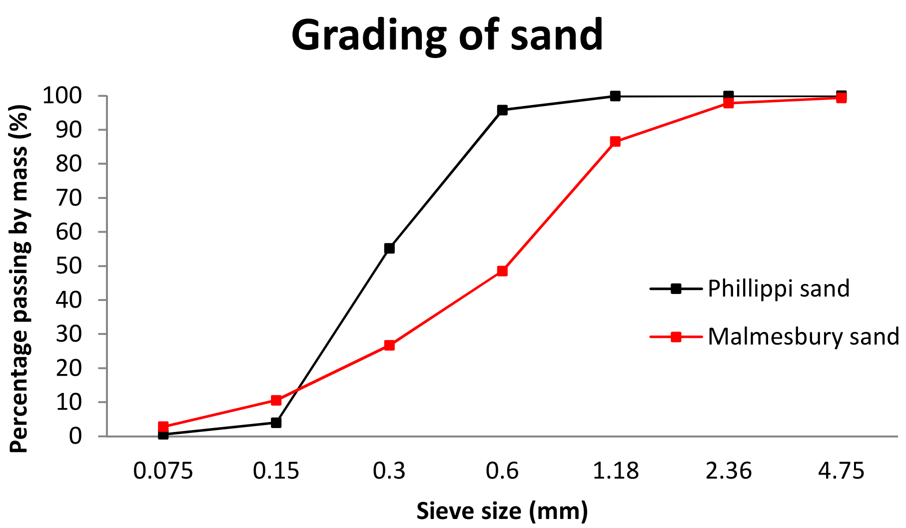

Figure 4: Particle distribution of the two fillers used

A single AEA available commercially (Mapeplast PT2) in South Africa was selected for use throughout this work, after preliminary testing with two products. The AEA has a light yellow colour and a density of 1.02 ± 0.02 kg/l at 20 ℃ according to ISO 758. In addition to entraining air into the concrete the AEA reduces bleeding, facilitates pumping and acts as a plasticiser.

3.2.2 WATER DEMAND OF FILLER

According to Kearsley (1996) in the past, for foamed concrete, the water demand of the concrete mix was determined by systematically increasing the water/cement ratio until no visual breakdown of the foam could be seen. The author explains that this method is user sensitive and proposed an alternative method which makes use of the ASTM flow table test for hydraulic cements. A detailed explanation of the flow table test for hydraulic cements can be found in *ASTM C230/ C230M - $98^{\varepsilon 2}$: Standard specification for Flow Table Use in Tests of Hydraulic Cement*. However, a brief summary of the test setup and methodology is required for further explanations. The flow table test consists of a mini cone with a height of 50 mm, top diameter of 70 mm and bottom diameter of 100 mm and a base plate with a diameter of 255 mm. The mini cone is filled with cement paste base (concrete base mix) and dropped from a height of 12.7 mm 15 times. The diameter of the paste is then measured which gives an indication of the spreadability and workability of the base mix. Figure 5 shows such a flow table in the laboratory of Stellenbosch University. The author used this test in conjunction with the visual breakdown method and determined the water demand of the cement and fly ash used in the concrete mix. Kearsley and Mostert (2005) also stated that if the water content was too low in the concrete mix the cement would draw water from the foam and rapid degeneration of foam would take place resulting in a higher density. Based on these observations and principles a similar approach to determine the water demand for the mix constituents of aerated concrete can be found.

Figure 5: Flow table test for hydraulic cements

Although they serve the same purpose, air-entraining agents work differently to pre-formed foam in cellular concrete. They both produce air voids in the concrete mix which has to be stabilized in the concrete's fresh state. For example, if pre-formed foam is used, the stability of the foam in the mix phase is of concern. The concern to be addressed is the question of whether enough water is present in the concrete base mix for the mix constituents so that the mix constituents do not take water from the foam. This would be the case if a dry concrete base mix is used. Kunhanandan Nambiar and Ramamurthy (2008) concluded from their investigation into the fresh properties of foamed concrete that there exists a band of water/solids ratio for which the concrete mix is stable. A stable mix in this case can be defined as a mix that is not too flowable to allow air bubbles to escape from the cement paste and not too dry so that the bubbles are broken. If the base mix contains too much water the air bubble would then make its way to the top of the paste and escape. When an air entraining agent is used, the conditions for air bubble formation would have to be similar to those conditions in which pre-formed foam is used. If a bubble were to form during the mixing stage, depending on the nature of the air-entraining (powder or fluid), certain conditions would have to be met. These conditions can be summarized as follows: (1) enough water must be present so that the air entraining agent can stabilize the bubbles formed during mixing, (2) the concentration of the air-entraining agent in the solution must be sufficient for air entrainment. As discussed by Zhang (1996) there exist dosage levels at which AEAs are ineffective in stabilizing air bubbles formed during the mixing stage. If there is too little water in the mix then the air-entraining agent is absorbed with the water by the cement for the hydration process. If there is too much water in the mix the bubbles escape from the mix to the surface and tend to form a layer of bubbles on the surface of the concrete. During trial mixing the author observed this phenomenon. When too much water was added to the concrete mix water would form a layer on the top surface of the concrete after consolidation. At a later stage, when the water has evaporated and setting has occurred, significant dimensional change can be observed. As a result of this the top layer of the concrete is extremely brittle and even with light touching the top layer would fall apart.

The proposed methodology to determine the water demand of the mix constituents rests upon the two conditions postulated for air entrainment. Simply stated, enough water needs to be present for the cement and other mix constituents and enough air entraining agent must be present as well. Therefore, to simplify the matter, it was chosen that the water demand of the sand shall be taken as the amount of water required to saturate the sand. This can easily be done and is described in the methodology presented in section 3.2.2.1, after which a modified version of the method used by Kearsley and Mostert (2005) to determine the water demand of the cement is given.

3.2.2.1 METHOD FOR DETERMINING THE WATER DEMAND OF THE FILLER

The water demand of the filler can be determined by the following steps:

1. Take a sample of filler of known mass

2. Add water to the sample until it is saturated
3. Calculate the amount of water added
4. Determine the water/filler ratio

It should be noted that the sample must not be super saturated. This condition, illustrated in Figure 6c, is characterised by puddles of water forming on the surface of the sand. If puddles form on the surface of the sand, the water can be drained from the sample until no puddles are present, or a new sample must be used. If too little water is present, Figure 6b, the filler will be dry and depending on the filler, will exhibit a display of two colours, a wet and dry appearance. Figure 6c shows the condition for which the water demand of the filler is satisfied, if a concrete mix were to be made with the filler in this condition, the hardened concrete mix will exhibit no dimensional instability as the air bubble will not escape but be trapped in the mix. If a concrete mix were to be made with the filler in the super saturated condition, Figure 6d, bubbles escape and dimensional instability occurs in the hardened concrete.

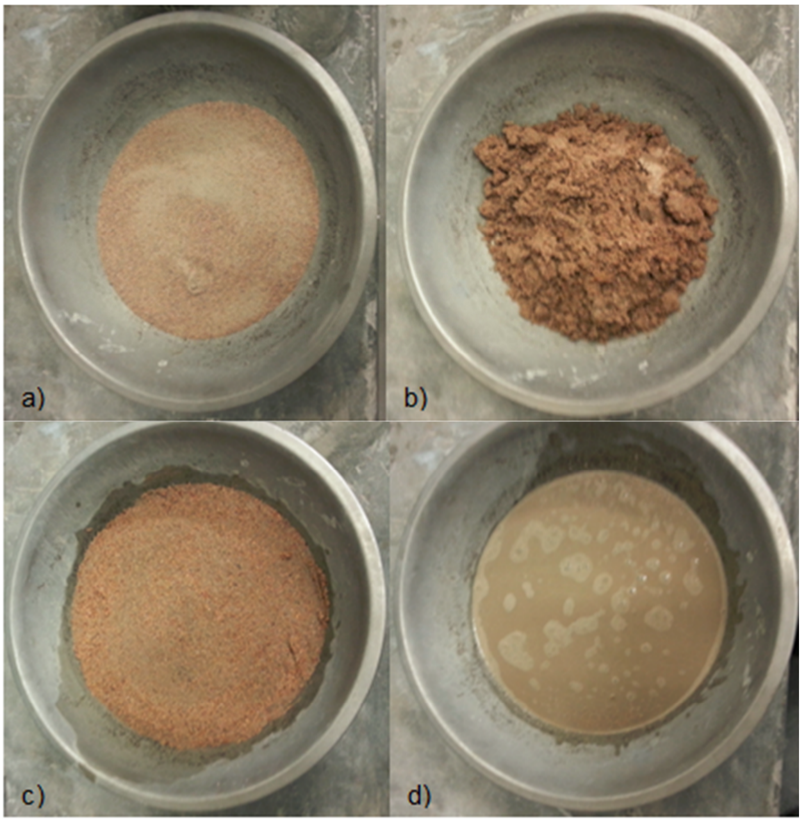

Figure 6: Series of pictures illustrating the influence the water has on filler

3.2.2.2 METHOD FOR DETERMINING THE WATER DEMAND OF THE CEMENT

The method used to determine the water/cement (w/c) ratio is based on the spreadability of the concrete base mix based on the results of the ASTM flow turn table test. A concrete base mix is defined as the cellular concrete mix before air entrainment takes place. For this method, a filler/cement ratio is chosen and a base mix is proportioned accordingly. The flow turn table test is conducted on the base mix and the fresh properties such as consistency, workability, flow ability, and spreadability of the concrete base mix is recorded. The spreadability range can be obtained from satisfying both conditions for air-entrainment. From this range an optimum value of spreadability is chosen and the water demand of the sand can be calculated from section 3.2.2.1 *Method for determining the water demand of the filler*. Since the total volume of water of the concrete base mix is known, the water demand of the cement can be calculated by subtracting the water demand of the filler from the total water in the concrete base mix. The w/c ratio is then calculated as the ratio of the water demand of the cement to the mass of the cement in the base mix. The methodology can be summarised by the following steps:

1. Based on the mix design parameters choose filler/cement (f/c) ratio
2. Proportion the concrete base mix by varying the water content of the mix
3. For each concrete base mix proportioned, record the results of consistency, workability, flow ability, and spreadability
4. Find the optimum spreadability value and calculate the water demand of filler with the known water/sand (w/s) ratio calculated from section 3.2.2.1
5. Calculate the water demand of the cement and determine the water/cement (w/c) ratio

The optimum value of spreadability is defined as the spreadability value of the concrete base mix that is not too dry so that air bubbles cannot form and not too wet so that air bubbles escape the concrete matrix. The mix evaluation in APPENDIX A shows how the optimum spreadability value is obtained for the different concrete mixes.

3.3 MIX DESIGN

3.3.1 MIX DESIGN FORMULATION

In the design of aerated concrete the mass of all the constituents must equal the mass of the target density of the mix, which leads to equation (3.1a). Equation (3.2a), equates the total volume of all constituents to 1000 litres, from which the required volume of air (V_a) to be entrained into the mix can be determined. The following equations can therefore be proposed for the mix design procedure; these equations are similar to the equations proposed by Kearsley and Mostert (2005). Also, a representation of the proposed mix design equations can be found in Table 1.

$$\rho_m = x_c + x_c(w/c) + x_c(w/s)(s/c) + x_c(w/a)(a/c) + x_c(s/c) + x_c(a/c) + RD_{AEA}V_{AEA} \quad (3.1a)$$

$$1000 = x_c/RD_c + x_c(w/c) + x_c(w/s)(s/c) + x_c(w/a)(a/c) + x_c(s/c)/RD_s + x_c(a/c)/RD_a + V_a + V_{AEA} \quad (3.2a)$$

<table>
<tr><th colspan="2">Total mass of the mix [kg]</th><th colspan="2">Total volume of the mix [l]</th></tr>
<tr><td rowspan="4">Target density of the mix</td><td>Cement [kg]</td><td rowspan="4">1000 [l]</td><td>Air [l]</td></tr>
<tr><td rowspan="2">Water</td><td>Cement [l]</td></tr>
<tr><td>Water [l]</td></tr>
<tr><td>Filler [kg]
(Sand, fly-ash)</td><td>Filler [l]
(Sand, fly-ash)</td></tr>
</table>

Table 1: Representation of the mix design equations

where:

ρ_m = the target density in kg/m^3

x_c = cement content in kg/m^3

(w/s) = water/sand ratio

(w/c) = water/cement ratio

(w/a) = water/ash ratio

(s/c) = sand/cement ratio

(a/c) = fly-ash/cement ratio

RD_c = relative density of cement

RD_s = relative density of sand

RD_a = relative density of fly-ash

RD_{AEA} = relative density of the air-entraining agent

V_{AEA} = volume of the air-entraining agent in litres

V_a = Volume of air to be entrained in litres

Equations (3.1a) and (3.2a) are set up specifically for when the w/c, w/s and w/a ratios are known. Alternatively, the water demand of the mix constituents need not be determined individually but as a whole. The resulting water demand is called the water solids ratio. The definition of the water/solids ratio is the ratio of the water demand of the mix to the mass of the solid constituents of the mix. If the water/solids ratio is used, equations (3.1b) and (3.2b) need to be modified accordingly, then the following equations govern the design of aerated concrete.

$$\rho_m = x_c + x_c\,(w/s^*)(f/c) + x_c\,(f/c) + RD_{AEA}V_{AEA} \tag{3.1b}$$

$$1000 = x_c/RD_c + x_c((s/c)/RD_s + (a/c)/RD_a) + x_c(w/s^*)(f/c) + V_a + V_{AEA} \tag{3.2b}$$

$$f/c)_r = (s/c) + (a/c) \tag{3.3}$$

where

(f/c) = filler/cement ratio

(w/s^*) = water/solids ratio

As a first attempt, the volume and mass term of the AEA may be omitted from equation (3.1 a), (3.1b) and (3.2a) and (3.2b). However, if the contributions of these terms are great relative to the other constituents of the mix, the terms cannot be omitted.

3.3.2 MIX DESIGN PROCEDURE

Air entrainment is dependent on the type and dosage of the air entraining agent, batch volumes and mixing, cement, aggregate, temperature, cement extenders, duration of mixing, speed of mixing and other chemical admixtures. These parameters directly influence the mix design process because the mix design is dependent on the amount of air entrained in the mix. In order to keep some of the dependent parameters unchanged during the mix stage, a mixing procedure needs to be adopted to ensure that the mixing procedure produces repeatable results. Since the amount of AEA needed to entrain a certain volume of air in the concrete mix is not known beforehand, a trial and error method is required. Therefore two mix design procedures are given, one that is conducted in a small mixer, similar to that used in bakeries, and one that is conducted in a pan mixer found in laboratories. Figure 7 shows the two mixers used in the mix design procedure. On the left in Figure 7 is the bakery mixer that has a capacity of 10 litres and on the right of the figure is the pan mixer which is used for larger mix volumes of 25L.

Figure 7: Bakery mixer and Pan mixer

3.3.2.1 TRIAL MIX (BAKERY MIXER)

The purpose of the small mixer is so that trial mix can be conducted on small scale. From the data accumulated on the small mix, the mix can be modified and used on a larger scale. The mix design procedure followed can be summarized as follows:

1. Choose a filler/cement (f/c) ratio, sand/cement (s/c), ash/cement (a/c)
2. Use the methods described in section 3.2.2.1 and 3.2.2.2 to determine the water/cement (w/c) ratio, water/sand (w/s) ratio and water/ash (w/a) ratio
3. Proportion a concrete base mix
4. Choose dosage of AEA
5. Mix the mix constituents for 2 minutes and gradually add the mixing water
6. Add AEA and start mixing for 10 minutes
7. Measure the density

3.3.2.2 FINAL MIX (PAN MIXERS)

After trial mixes have been conducted on the small mixer, the final mix is scaled up to a larger volume for mixing in the larger pan mixers. Depending on the volume of concrete needed a mix is proportioned weighed off and mixed in a fashion similar to that used for trial mixes. The only variation is that the time factor cannot be kept the same as the mixers have different actions and speeds at which they operate. It has been reported throughout the literature that the mixing speed influences the amount of air entrained into the concrete. Generally, with an increase in mixing speed there is an increase in the amount of air entrained into the concrete. However, there exists a “critical” mixing speed that coincides with the maximum optimum mixing speed. The optimum mixing speed is defined as the mixing speed that produces the maximum amount of entrained air in the concrete. Further increases in mixing speed beyond the critical mixing speed results in a decrease in the amount of air entrained in the concrete. The effect of the variation will be discussed later in the results section but for now the methodology used in the mix procedure of the final concrete mixes is given. The method can be summarized as follows

1. From the trial mix method all the parameters and mix constituents are known. Mix the dry material for 2 minutes
2. Add water and mix for another 2 minutes
3. Check the concrete base mix then mix for 30 seconds and add AEA.
4. Measure the density

For the final mix in step 1, all of the mix design parameters are known. The dry material is mixed for 2 minutes and checked to ensure that there is a state of homogeneity in the dry materials. In step 2 the water is then added gradually to the mix and the concrete base mix formed is mixed for another 2 minutes and then checked to ensure that it contains no dry material. In step 3 the concrete base mix is then mixed for 30 seconds, after which the AEA is added. The concrete mix is allowed to mix for a period of time then the mixing is stopped. The density is then obtained by filling a container of known volume and measuring the mass. This is accepted as the wet density of the concrete mix.

It should be noted that with the small bakery mixer, the concrete was mixed for 10 minutes after the AEA was added to the concrete base mix. However, due to the difference in mixing actions of the blades and mixing speed of the bakery mixer and the pan mixer the time parameter could not be kept the same for the final mix. The time required for the concrete mixes to reach their design target densities for the final mixes showed no correlation for the 2000 kg/m^3 target design density. For the design target density of 2000 kg/m^3 the time required for the mix to reach this density was in the range of 45 seconds to 1 minute and 15 seconds. For both the 1800 kg/m^3 and 1600 kg/m^3 target design densities the time required for the mix to reach these densities was in the range of 8 to 9 minutes.

3.4 MIX DESIGN CONSIDERATIONS

As stated earlier, the three performance requirements that must be met in the design of lightweight concrete are compressive strength, workability and relative density. In cellular concrete, irrespective of the type of application in mind, the relative density is the factor that controls the design of the concrete. For instance, if the concrete were to be used for structural purposes, the density range in which compressive strengths which are considered to be structurally adequate, needs to be established first. Reviewing literature indicates that, certain strengths are only achievably within certain density ranges. If the concrete were to be used for non-structural applications, density would still be the controlling factor because all of the properties of lightweight concrete depend on the density of the concrete. In both cases considered, the workability of the concrete is considered last but its importance should not be underestimated as it plays a major role in air entrainment. The factors that influence the three performance requirements therefore need to be considered. They are the filler content, filler/cement ratio, sand/cement ratio, ash/sand ratio, and cement content. Only the influence of these parameters on the compressive strength and density will be discussed as section *3.2 Water demand of the mix constituents'* deals with the water demand which essentially covers the workability issue of the concrete without addressing it explicitly.

3.4.1 DENSITY AND STRENGTH CONSIDERATIONS IN THE MIX DESIGN PROCESS

In the design of cellular concrete the target density is chosen and a concrete mix is proportioned accordingly. Depending on the type of application in mind, structural or non-structural, the concrete density is the main consideration in the mix design process. Besides density, strength plays an important role in the concrete as well. Therefore, this section highlights some of the aspects that can be considered during the mix design procedure to guide the designer to a mix that conforms to the three design performance criteria.

The strength of cellular concrete is influenced by the density, filler and curing regime used. Kearsley and Mostert (2005) conducted an investigation into the design of foamed concrete with high fly ash content. In the investigation the authors studied the effects of density and the utilization of high fly-ash contents on the compressive strength of steam cured foamed concrete. From the investigation they concluded that density is the main factor that contributes to the strength and that the fly-ash content had little influence on the strength of the concrete. Also, it was found that doubling the cement content for a particular density, (in their work particularly for 1250 kg/m^3) only increased the compressive strength by 12 per cent. This could be classified as uneconomical from a financial point of view.

Other than increasing the density, the use of fine filler has been reported to increase the compressive strength of aerated concrete. Kunhanandan Nambiar and Ramamurthy (2006) investigated the effect of filler on the properties of concrete. The authors found that the use of fine filler in foamed concrete yielded a greater compressive strength than those made with coarse filler. As a result, the use of fly-ash would greatly increase the compressive strength of the concrete because this acts both as very fine filler and some of the fly-ash reacts with the cement hydration products to form cementitious material that also contributes to the strength of the concrete. It should be noted that the use of fly-ash increases the required dosage of air entraining agent. This fact was investigated by Zhang (1996) who concluded that the use of fly-ash may increase the dosage by two or more times.

3.4.2 TARGET DESIGN DENSITY VERSUS DRY DENSITY

As discussed earlier, in the mix design of cellular concretes, the target design density dictates the mix design procedure. However, the target design density of the concrete mix is not the same as concrete density after setting has taken place. In all cases, the 'final' density of the concrete after setting is much lower than the design target density and because the properties of cellular concrete depend on the dry density of the concrete it is worthwhile to know the relationship between casting density and dry density. Kearsley and Mostert (2005) produced a linear equation that related the casting density to the dry density of foamed concrete in the range of 600 kg/m^3 and 1200 kg/m^3.The equation, given as equation (3.4), shows that in order to achieve a certain dry density the target design density should at least be 100 kg/m^3 more than the required dry density.

$$\rho_m = 1.034\rho_{dry} + 101.96 \qquad (3.4)$$

where:

ρ_m = the target density in kg/m^3

ρ_{dry} = dry density in kg/m^3

CHAPTER 4 - EXPERIMENTAL DESIGN

4.1 INTRODUCTION

Unlike normal concrete, cellular (aerated, foamed) concrete consists only of cement, water, and sand and can be defined as concrete with 20 per cent by volume or more air entrained into the concrete. Due to the high levels of air voids in the concrete a characteristic compressive strength of 25 MPa or more is difficult to achieve, hence the uses of cellular concrete have mostly been limited to non-structural applications. However, recently there has been renewed interest in its structural use. Jones and McCarthy (2005) discussed the potential of foamed concrete as a structural material. They examined some of the key early age engineering and durability properties and concluded from the data that foamed concrete is indeed viable for structural uses.

In the concrete construction industry a simple way to produce cellular concrete is to add an air entraining agent (AEA) to the concrete mix. Due to the wide range of AEA's available commercially, a design approach using an arbitrary AEA for producing aerated concrete with a certain density has to be developed. This section is therefore divided into two sections; section (1) describes the methodology followed to produce cellular concrete with a certain density using a commercially available AEA (discussed in Chapter 3); section (2) deals with the hardened concrete and uses characterisation testing such as the mechanical strength tests (compression, tensile (indirect)), and the modulus of elasticity) to evaluate some of the properties of cellular concrete.

4.2 EXPERIMENTAL DESIGN

Aggregate, in concrete, makes between 60 and 70 per cent of the volume of concrete. For this reason, the properties of aggregate have a significant effect on the properties of concrete in the fresh state and in the hardened state. For instance, in the fresh state, the consistency and workability of the concrete is affected by the sand grading and content in the mix. It has been reported that sand with a high content of fine material causes the mix to be sticky which decreases the workability of the concrete. When there is a lack of fine material in the sand, the mix may become too flowable and is prone to segregation. In the hardened state, the elastic stiffness of the concrete is affected by the properties of the aggregate. If the aggregate has a low stiffness then the concrete will also have a low stiffness. In lightweight concrete, these effects seem to be exaggerated and should be considered carefully in the mix design process. For instance, it is known that the composite

E-modulus of concrete is affected by the porosity of the concrete and the volume and E-modulus of the aggregate and paste in the concrete mix, for the same volume of concrete lightweight concretes would have a higher porosity and less volume of the aggregate and paste in the concrete mix than normal weight concrete would have. Hence a lower E-modulus is expected for the lightweight concrete. The properties of aerated concrete, which is a lightweight concrete that consists only of cement, filler (sand and/or fly-ash), water, and air voids will greatly be affected by the properties of the filler. Therefore, an investigation into the effects that the filler has on the concrete is warranted.

In order to investigate the effect the filler has on the concrete two experiments were designed and performed. Experiment 1 investigates the influence the filler has on the concrete and experiment 2 investigates the strength development of aerated concrete. In each of these experiments, the following standard tests were conducted on the hardened concrete; compressive strength test and tensile splitting test. The elastic modulus was also determined on the hardened concrete. The following details are discussed under each experiment; parameters under investigation, methodology of the experiment, sample preparation and experimental program.

4.2.1 INFLUENCE THE FILLER HAS ON THE CONCRETE

The term filler in cellular concrete is used to describe the fine material used in the concrete mix. The different types of fillers used in cellular concrete are sand and fly-ash. Strictly speaking, fly-ash is not a filler because it has cementitious properties. But when used in large volumes, as in cellular concrete, it is considered to be a filler because only a certain percentage of the fly-ash reacts with the products of cement hydration and the rest acts as filler (Kearsley and Mostert, 2005). Other than having cementitious properties, fly-ash may have several beneficial characteristics for concrete, for instance improved durability of the concrete in terms of covercrete density, reduced alkalinity whereby alkali-silicate reaction may be limited or prevented. As fine filler it also assists in reduced bleeding of the concrete. Sand on the other hand is used as the primary filler in aerated concrete. Normally sand which consists of a large percentage of fines is used because there have been reports that fine sands enable a higher air content than when coarse sands are used (Mielenz et al., 1958). Thus, the filler influences the cellular concrete in both the fresh state and in the hardened state. The fresh state properties that are affected by the filler include the workability, consistency and the air content. In the hardened state, the mechanical properties, which include the compressive and tensile strength, and the engineering properties such as the elastic modulus are affected by the filler. The diagram in Figure 8 shows the cellular concrete properties that are influenced by the filler.

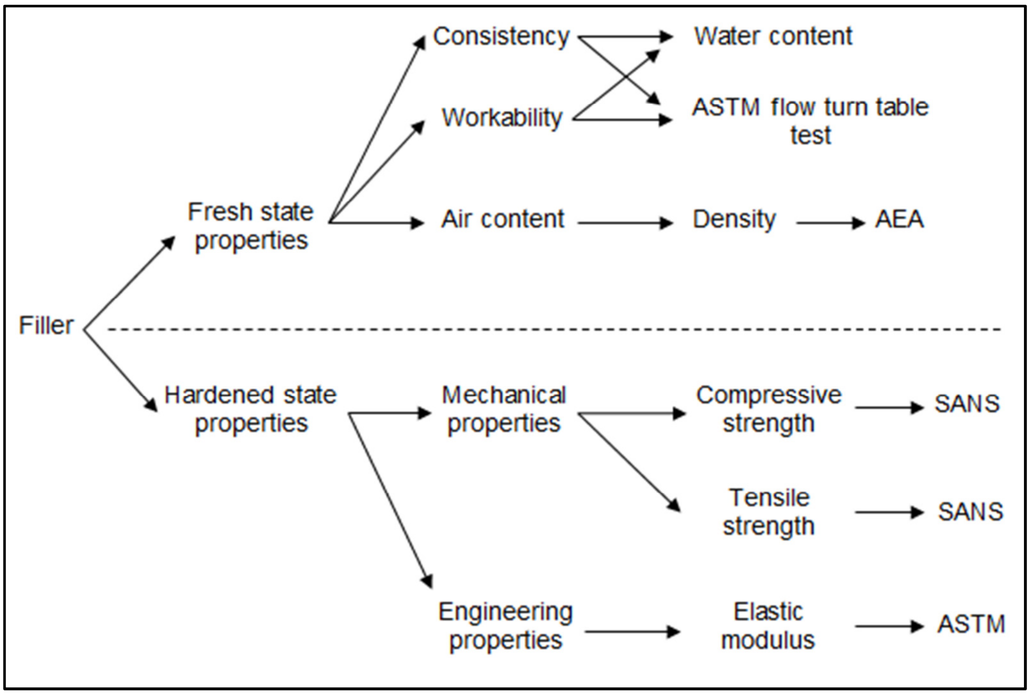

Figure 8: Diagrammatic representation of experiment 1: the role of filler in cellular concrete

Figure 8 depicts a flow diagram that shows how the concrete is affected by the choice of filler. As an added feature, the last arrow from each factor shows how this factor is determined or evaluated. In the fresh state properties, the workability and the consistency are evaluated using the ASTM flow turn table and water content of the concrete mix. The air content is calculated from the measured density and is evaluated by the amount of air entraining agent (AEA) as a percentage of the total binder in the concrete mix. Such a calibration process is required, because the indications of dosage on design sheets provided by AEA manufacturers' are indeed mere indications of the required dosage of AEA as a percentage of the total binder in the concrete mix. For the hardened state properties, the mechanical properties are obtained by performing the compression strength test and tensile strength test as described by the appropriate South African National Standard (SANS). The elastic modulus of the concrete is evaluated using the ASTM standard. A brief overview of the tests performed on the concrete is given in the next sections.

In this experiment, one test was performed on the fresh concrete and three tests were performed on the hardened concrete. Also, the weight of the fresh concrete was taken to calculate the density. The test performed on the fresh concrete was the flow turn table for hydraulic cements as per ASTM C 260/ C 230. A description of the test setup was given in section 3.2.2 and shall not be discuss here any further.

4.2.1.1 AIR CONTENT

Various methods exist for the determination of the air content in the fresh concrete. The pressure method as described in ASTM C231 is a standard test method that can be used to determine the air content in the fresh concrete. It was attempted to measure the air content of the concrete mix in accordance with ASTM C231 but due to the large percentage of air in the concrete mix the apparatus started to leak water. Any further attempts were avoided and an alternative method to calculate the air content was needed. It was decided to calculate the air content based on the mix proportions and the measured density.

PROCEDURE FOR DETERMINING THE AIR CONTENT BASED ON MIX PROPORTIONS

The following steps outline the methodology followed to determine the air content of the concrete mix.

1. Follow the procedures outlined in *SANS 5861-2 - Sampling of freshly mixed concrete* to fill a container of known volume with freshly mixed concrete.
2. Measure the weight of the concrete and calculate the density of the fresh concrete mix.
3. Calculate the required volume of air needed in the concrete mix to produce a concrete density equal to the measured density from the known masses and volumes of all the mix constituents water, sand, cement, fly ash (if included), AEA, air based on their solid densities and the proportions used in the mix
4. Finally, calculate the air content of the concrete mix by dividing the calculated volume of air in the concrete mix by the total volume of the concrete mix

$$\rho_{measured} = \frac{m_w + m_s + m_a + m_{AEA}}{V_w + V_s + V_a + V_{AEA} + V_{a(req)}} = \frac{m_{tot}}{V_{tot} + V_{a(req)}} \quad (4.1)$$

$$\text{Air content (\%)} = 100 \times \frac{V_{a(req)}}{V_w + V_s + V_a + V_{AEA} + V_{a(req)}} = 100 \times \frac{V_{a(req)}}{V_{tot} + V_{a(req)}} \quad (4.2)$$

where:

m_w, m_s, m_a and m_{AEA} are the masses of the mix constituents' water, sand, fly-ash and AEA in kg and m_{tot} is sum of the masses of the mix constituents in kg.

V_w, V_s, V_a and V_{AEA} are the volumes of the mix constituents' water, sand, fly-ash and AEA in litres m^3 and V_{tot} is sum of the masses of the mix constituents in kg. $V_{a(req)}$ is the volume of air needed in the concrete mix to produce a concrete density equal to the measured density in m^3

4.2.1.2 MECHANICAL PROPERTIES

The three tests performed in the hardened state of the concrete are the compression strength test, tensile strength test and the modulus of elasticity. The strength tests were done in accordance with SANS standards and the modulus of elasticity in accordance with ASTM standards.

COMPRESSIVE STRENGTH

Concrete compressive strength is one of the three performance criteria specified in the design of lightweight concrete and should be considered equally important as the other performance criteria. (i.e. density and workability) In the design of concrete, the most common requirement is that the characteristic compressive strength of the concrete must be equal or higher than the required compressive strength specified by the designer. The properties of hardened concrete are time dependant, therefore, any test method that is performed on the concrete is done at a certain age. The concrete compressive strength test is no exception and is usually tested 28 days after casting the concrete. In design standards, the 28 day compressive strength is used, statistically recalculated from the ultimate strength of several specimens (a minimum of 3) per test to the concrete characteristic strength usually based on the 5 percentile and is considered as the concrete final compressive strength for design purposes. The crushing test can also be performed on the concrete at different ages. For instance, it is not uncommon to test the compressive strength of the concrete at the age of 7 days. This is done because it is normally accepted that the concrete will reach 70 per cent of its final compressive strength at 7 days. Testing at different ages in general provides information of concrete strength evolution, essential for practical decisions about stripping and post-tensioning in construction stages.

The concrete compressive strength test was performed in accordance with *SANS 5863:2006 Concrete tests – Compressive strength of hardened concrete on concrete.* The concrete test specimens were cast in 100 mm cube moulds. The test was performed on 6 concrete specimens per mix, of which 3 specimens were tested at the age of 7 days after casting and the other 3 at the age of 28 days. The average of the three specimens was taken as the concrete compressive strength at that age.

For the compression strength test cubical or cylindrical specimens can be used. Standard dimensions for cubical specimens include cubes of side lengths 100 mm and 150 mm. For cylindrical specimens a diameter to length ratio of 2 is normally required. Higher ratios are also acceptable but it should be noted that the slenderness of the specimen may affect the results from the compression test. *SANS 5863:2006: Concrete tests - Compressive strength of hardened concrete* makes provision for the use of both cylindrical and cubical specimens. The standard describes the procedure to test the specimens to obtain the compressive strength of the concrete. A summary of the procedure is presented in the following section.

To perform the compressive strength test SANS 5863:2006 requires that a constant load of 0.3 ± 0.1 MPa/s be applied to the concrete specimen until failure is reached. For cubes of side length 100 mm this loading rate converts to 3 ± 1 kN/s and for cylindrical specimens of diameter 100 mm and length 200 mm the loading rate is 0.75 ± 0.25 kN/s. Stellenbosch University makes use of a 350 ton Contest material testing machine to apply the required rate to the specimens. A setup of the test with a cubical specimen is shown in Figure 9. The load corresponding to failure of the concrete specimen is recorded, in newtons (N), and makes use of equation (4.3) to calculate the compressive strength of the concrete. The area A of the specimen is calculated from the measured dimensions of the specimen and is given in square millimetres. To measure the dimensions of the specimen a caliper vernier was used. The two dimensions were taken across the centreline of the specimens at 90 degrees at the top of the specimens.

$$\sigma_{comp} = \frac{F}{A} \tag{4.3}$$

where

σ_{comp} = compressive strength of the concrete in megapascal (MPa)

F = load at failure in newtons (N)

A = cross-sectional area of the specimen measured in square millimetres (mm^2) on which the load is applied

Figure 9: Contest compressive strength testing machine with cubical specimen

TENSILE STRENGTH

Although, the concrete tensile strength is not one of the performance criteria in the design of lightweight concrete, it is still an important property of the concrete that should be determined on the hardened concrete. This value is useful to know because it can be used to estimate when cracking will occur in the concrete under a given load. Cracks should be prevented or limited in width in reinforced concrete because this could lead to corrosion of the reinforcement. The three most common tests used to determine the tensile strength of concrete are the direct tensile strength test, flexural strength test and the splitting strength test. Both the splitting and flexural tests are known to be indirect tensile tests, of which the results are usually higher than those obtained from the direct tensile test. This is due to stress / strain gradients in the splitting and flexural test specimens, and therefore non-uniform stress distributions which lead to inclusion of toughness and even confinement in the strength obtained from these tests. Conversion from splitting test results to direct tensile strength results is rather well established and included for instance in Eurocode 2 (BS EN1992-1-1:2004). Due to its simplicity, it was decided that the tensile splitting strength test would be used.

The concrete tensile splitting test was done in accordance with *SANS 6253:2006 Concrete tests – Tensile splitting strength of concrete*. The standard makes provision for the use cylindrical, cubical or prismatic specimens. Standard dimensions for cubical specimens include cubes of side lengths 100 mm and 150 mm. For cylindrical specimens a diameter to length ratio of 2 is normally required. *SANS 6253:2006: Concrete tests – Tensile splitting* *strength of concrete* makes provision for the use of both cylindrical and cubical specimens. It was decided that cubical specimens would be used for the tensile splitting strength test. The concrete test specimens were casted in 100 mm cube moulds. The test was performed on 3 concrete specimens 28 days after casting. The average of the three specimens was taken as the concrete tensile strength.

To perform the tensile splitting strength test SANS 6253 prescribes that a constant load of 0.3 ± 0.1 MPa/s be applied to the concrete specimen until failure is reached. Stellenbosch University makes use of a Zwick Z250 material testing machine to apply the required rate to the specimens. A setup of the test with a cubical specimen is shown in Figure 10. The load corresponding to failure of the concrete specimen is recorded, in newtons (N), and makes use of equation (4.4) to calculate the tensile strength of the concrete. The dimension of the specimen is measured using a caliper vernier and is given in millimetres. The dimension is taken across the centreline of the specimens perpendicular to the direction of loading on both sides of the specimen.

$$\sigma_{split} = \frac{2F}{\pi a^2} \tag{4.4}$$

where

σ_{split} = tensile strength of the concrete in megapascal (MPa)

F = load at failure in newtons (N)

a = is the length of the cube in millimetres (mm)

Figure 10: Tensile splitting test

4.2.1.3 MODULUS OF ELASTICITY

For the static modulus of elasticity test cubical or cylindrical specimens can be used. However, before specimen shapes and dimensions are chosen, care should be taken as the setup and material testing machine of the test may restrict the size of the specimen. It is common practice to use cylindrical specimens in the test for the static modulus of elasticity but cubical specimens may also be used. The preference of cylindrical specimens is due to their aspect (length to diameter) of usually 2, which ensures a reasonably uniform and uniaxial stress distribution in the central part, from which the stiffness can be determined most accurately. In cubes, the friction at the loading plates, causes lateral confinement in the specimens, which influences deformation as well as strength in the specimen. For this reason cylinders were preferred in this project. Due to the height restriction imposed by the setup and testing machine, it was decided that cylindrical specimens of diameter size 100 mm and length of 200 mm would be used. *ASTM C469-02: Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression* describes the procedure to determine the static modulus of elasticity on cylindrical concrete specimens. A summary of the procedure is presented in the following section.

To perform the compressive strength test ASTM C49-02 requires that a constant load of 241 ± 34 kPa/s be applied to the concrete specimen until failure is reached. The code also requires that two sensing devices that measure deformation be used on two diametrically opposed lines on the midheight of the specimen. A slight variation from the standard was made here; instead of two, three sensing devices that measure deformation were used spaced 120° apart on the midheight of the specimens. A 350 ton Contest material testing machine was used to apply the required loading rate to the specimens, a load cell was used to measure the load applied to the specimen, and three linear variable differential transducers of type HBM W10 were used to measure the deformation. The data was recorded and logged by a data logger. Before the test commenced and actual measurements were recorded, the concrete specimens were loaded at least twice to a load between 30 and 40 per cent of the estimated ultimate load of the concrete specimen. A setup of the test with a cylindrical specimen is shown in Figure 11. By dividing the load recorded by the data logger with the cross sectional area the load is converted to the compressive stress in the concrete. The average of the three deformation measurements recorded by the data logger was taken and the strain corresponding to this deformation can be obtained by dividing it by the gauge length. The gauge length is the length over which the deformation was recorded. The stress strain diagram can then be plotted and the required stress and strain values can be read off the graphs or can be found in the data. The values are then used in equation (4.4) to calculate the static modulus of elasticity.

$$E_c = \frac{\sigma_1 - \sigma_0}{\varepsilon_1 - \varepsilon_0} \tag{4.5}$$

where

E_c = Elastic Modulus (GPa)

σ_1 = Stress corresponding to 40 % ultimate load (MPa)

σ_0 = Stress corresponding to 0.005 % strain (MPa)

ε_1 = Strain corresponding to stress at 40 % ultimate load (mm/mm)

ε_0 = 0.005 % strain (mm/mm)

Figure 11: Modulus of elasticity test setup with cylindrical specimen

4.2.1.4 PARAMETERS UNDER INVESTIGATION AND METHODOLOGY

In this experiment, the following parameters were under investigation, the effect the filler has on the mix design procedure, the effect the filler has on the strength of the concrete, the effect the filler has on the AEA, and the effect of curing (wet, i.e. submerged in water vs. wrapped in plastic after stripping) on the strength of the concrete. For this reason, four aerated concrete mixes were designed, two mixes using Malmesbury sand with a filler/cement (f/c) ratio of 1.25 and 1.50 and two mixes using Phillippi sand with f/c ratio of 1.25 and 1.50. The design target densities of the mixes were chosen as 1600 and 1800 kg/m^3. The mixes are presented in Table 2.

	Malmesbury Sand		Phillippi Sand	
f/c	1.25	1.5	1.25	1.5
s/c	1.25	1.5	1.25	1.5
a/c	0.0	0.0	0.0	0.0
	1600 kg/m^3	**1800 kg/m^3**	**1600 kg/m^3**	**1800 kg/m^3**
Cement(kg)	621.5	625.3	607.5	620.5
Water (kg)	232.4	234.5	232.4	248.2
Sand (kg)	759.4	938	759.4	930.8
Fly-ash(kg)	0.00	0.00	0.00	0.00
AEA (kg)	1.62	1.83	0.62	0.41
Air (l)	294.1	210.4	286.9	202.5

Table 2: Mix composition of Experiment 1

4.2.1. SAMPLE PREPARATION AND EXPERIMENTAL PROGRAM

After mixing, the fresh density of the concrete was measured as described in section 4.2.1.1 The concrete was then poured into moulds, kept protected in the laboratory environment and demoulded 24 hours later. Half of the specimens were submerged in water at a constant temperature of 23±1 ℃ to cure for 7 days and 28 days respectively. The other half were wrapped in plastic and stored in a room for 7 days and 28 days respectively. The compressive strength of the lightweight concretes was determined at the ages 7 days and 28 days. Also, the tensile strength was determined using an indirect method (splitting test) and the elastic modulus was determined at 28 days.

4.2.2 STRENGTH DEVELOPMENT IN AERATED CONCRETE

Having established the role of filler on aerated concrete, this phase of the research project intends to study several other mechanisms of strength of aerated concrete, including filler/cement ratio (f/c), water/cement ratio (w/c), sand/cement ratio (s/c), water/solids ratio (w/s) and fly-ash/cement ratio (a/c). These are the parameters chosen in the mix design phase. This phase is illustrated in Figure 12.

In the mix design of cellular concrete density is the controlling factor. The strength of the concrete is influenced by the density and other factors that seem to have a less significant effect on the strength of the concrete. In the design of cellular concrete, the strength of the concrete is not determined by the water/cement (w/c) ratio. Instead, the density dominates the strength, and for a given density, a concrete strength within a certain range is expected.

Although, density has been reported to be the most significant factor influencing the strength of the concrete, it is worthwhile to investigate the effect these parameters may have, no matter how insignificant they may seem to be.

The experimental design for this phase of the program is illustrated in Figure 12. These factors include the target density of the concrete mix, filler/cement (f/c) ratio, sand/cement (s/c) ratio, and ash/cement (a/c) ratio. Figure 12 shows a flow diagram depicting the factors influencing the concrete strength. The overall outcome of the thesis is to produce structural lightweight aerated concrete. The literature suggests that this can be achieved with concrete densities of 1600 kg/m^3 and higher. Higher strengths have been reported for lower densities but those experiments made use of curing regimes that are linked to high energy use such as steam curing, autoclaving and high temperature curing. Therefore it was decided that three different design target densities will be used in this investigation that could produce structural lightweight aerated concrete. The design target densities chosen were 1600 kg/m^3, 1800 kg/m^3, and 2000 kg/m^3. Strictly speaking, a concrete density of 2000 kg/m^3 falls outside the definition of cellular concrete which was defined earlier. However, to investigate the effect the density has on the compressive strength of the concrete it is vital that a third data set is present in order to see if some kind of trend manifests.

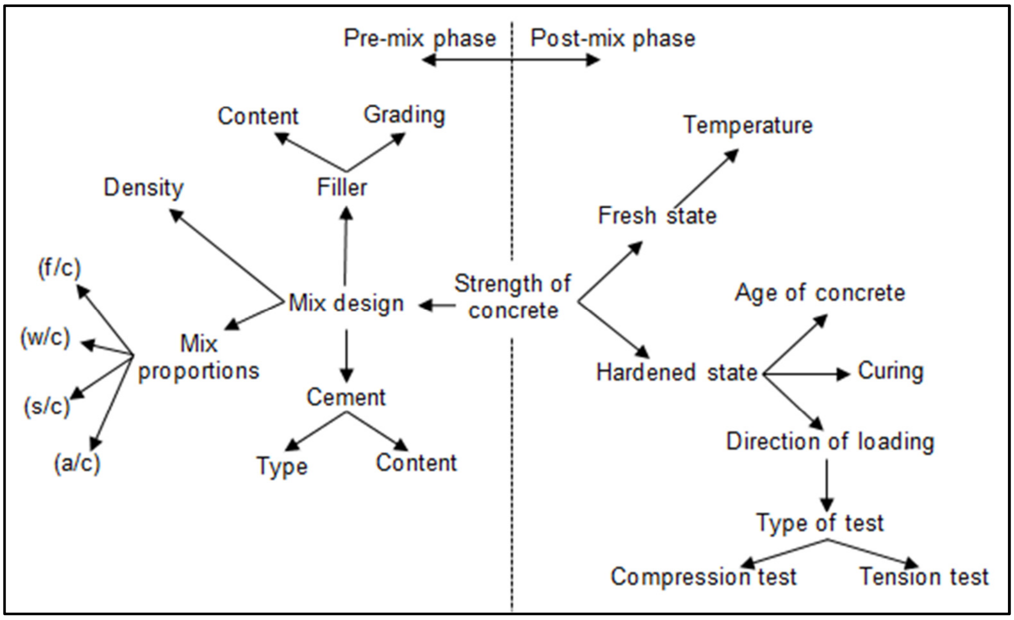

Figure 12: Flow diagram of the factors influencing the strength of concrete

Kunhanandan Nambiar and Ramamurthy (2006) investigated the influence of filler on the properties of foamed concrete. One of the observations of the experiment was that the use of a fine filler lead to a higher compressive strength to density ratio. Kearsley and Mostert

(2005) observed that doubling the cement content leads to an increase of 12 per cent in the compressive strength of the concrete with a density of 1250 kg/m^3 and believed that this increase in compressive strength increase with density. To investigate these observations, two factors (f/c ratio, cement content), it was decided to use two different fillers, one coarse filler and one fine filler, available locally, Malmesbury sand and Phillippi sand respectively. The f/c ratio was also varied in order to see the effect of the cement content of the compressive strength of the concrete. The f/c ratios used in this investigation are 1.0, 1.5 and 2.0. No fly-ash was used during this investigation. The concrete mix compositions used in this investigation are presented in Table 3.

	Malmesbury Sand			Phillippi Sand		
f/c	1.0	1.5	2.0	1.0	1.5	2.0
s/c	1.0	1.5	2.0	1.0	1.5	2.0
a/c	0.0	0.0	0.0	0.0	0.0	0.0
2000 kg/m^3						
Cement(kg)	860.0	700.5	590.0	855.0	690.0	575.0
Water (kg)	279.5	248.7	230.1	290.7	276.0	276.0
Sand (kg)	860.0	1050.8	1180.0	855.0	1035.0	1150.0
Fly-ash(kg)	0.00	0.00	0.00	0.00	0.00	0.00
AEA (kg)	0.29	0.33	0.26	0.26	0.11	0.11
Air (l)	121.7	131.5	136.6	114.6	114.1	107.4
1800 kg/m^3						
Cement(kg)	774.0	630.0	531.0	769.0	620.5	517.2
Water (kg)	251.6	223.7	207.1	261.5	248.2	248.3
Sand (kg)	774	945.0	1062.0	769.0	930.8	1034.4
Fly-ash(kg)	0.00	0.00	0.00	0.00	0.00	0.00
AEA (kg)	0.74	0.68	0.58	0.66	0.31	0.25
Air (l)	209.3	218.1	222.8	202.9	202.5	196.5
1600 kg/m^3						
Cement(kg)	687.0	560.0	472.0	683.0	551.5	460.0
Water (kg)	223.3	198.8	184.1	232.2	220.6	220.8
Sand (kg)	687.0	840.0	944.0	683.0	827.3	920.0
Fly-ash(kg)	0.00	0.00	0.00	0.00	0.00	0.00
AEA (kg)	2.12	1.26	1.27	1.50	0.97	0.47
Air (l)	296.2	304.7	308.9	290.9	290.8	285.9

Table 3: Mix compositions for Experiment 2

Thus far, the factors that influence the strength of the concrete in the mix design phase have only been considered. Now, the factors that influence the strength in the fresh state and hardened state need to be considered. In the fresh state, the temperature affects the early age strength development of concrete. However, this could not be considered because no test could be designed and performed on the concrete due to the fact that the climate room was not functioning at the time. Therefore, it was decided to limit the scope of the investigation and only consider the factors that influence the strength of the concrete in the hardened state. These factors include curing, specimen size, direction of loading and the age of the concrete at the time of testing.

According to Neville (1998) curing of the concrete is the name given to procedures promoting the hydration of cement and consists of a control of temperature and the movement of moisture from and into the concrete. Various methods of curing exist, they include submerged water curing, moist curing, dry curing, air curing, steam curing, autoclaving, high temperature curing, etc. Since the world is becoming more energy efficient and methods consuming or using less energy tend to be more favourable than others, it was decided not to use autoclaving processes, but normal water curing. Specimens were submerged in water of which the temperature is kept at 23±1°C. The previous experiments in phase 1 (section 4.2.1) utilized both submerged water curing and wrapped curing and found that the strength difference between samples curing in the water and those curing by wrapping the concrete in plastic was small but the submerged water curing yielded a slightly higher compressive strength for the hardened concrete. Therefore, either method is acceptable to use for cellular concrete. Therefore, the concrete specimens of phase 2 were water cured until the time of testing. For each test conducted the samples were tested within 30 minutes upon removal from the water.

Specimen size, age and direction of loading also influence the strength of the concrete in the hardened state. The direction of loading can either cause tensile or compressive forces in the concrete and is dependent on the type of test conducted on the concrete, as elaborated in section 4.2.1. It was decided that the compression strength test would dominantly be used in phase 2, having established the splitting strength for various densities of cellular concrete in phase 1 already. As in phase 1 (section 4.2.1) the specimen size chosen for the compression test is 100 mm cubes. In phase 2, the evolution of strength was studied in greater detail. Twelve specimens were prepared for each concrete mix for the compression strength test. At the concrete ages of 7 days, 14 days, 21 days and 28 days three specimens were tested; the average of the results of the three specimens was taken as the compressive strength of the concrete at that age. Some splitting tests were performed in phase 2. Also for these tests, cubic specimens of 100 mm side lengths were chosen, as was the case in phase 1. The test was performed on three specimens at a concrete age of 28 days. The average of the results of the three specimens was taken as the tensile strength of the concrete at the age of testing. For the elastic modulus, cylindrical specimens of diameter to length ratio of 2 were used. The diameter of the specimens used was 100 mm with a length of 200 mm. Three specimens were used to perform the test at a concrete age of 28 days. The average of the results of the three specimens was taken as the elastic modulus of the concrete at the age of testing.

CHAPTER 5 - RESULTS AND DISCUSSION

5.1 INTRODUCTION

In this section the results of the experiments performed on the concrete are presented and discussed. The results of the mix design methodology are discussed first. Secondly, the results from experiment 1 of the influence the filler has on the concrete, and lastly the results from experiment 3, strength development of the concrete over time, are discussed.

5.2 MIX DESIGN

Under the mix design experiment the following factors were under investigation: water demand of the concrete mix, water demand of the mix constituents, and the amount of AEA needed to produce aerated concrete of a certain density. The water demand of the mix is directly derived from the water demand of the mix constituents and therefore shall be discussed together in the same section. The results of the proposed method to determine the water demand of the mix constituents are presented under this section.

5.2.1 WATER DEMAND OF THE MIX CONSTITUENTS

5.2.1.1 WATER DEMAND OF THE FILLER

For the water demand of the filler the method described in section 3.2.2 was followed. The results of the two fillers used are presented in Table 4. The results show that the fine sand (Phillippi sand) requires more water to be saturated than the coarse sand (Malmesbury sand). A reason for this is that the fine sand has a larger surface area per volume than the coarse sand.

	Phillippi sand	Malmesbury sand
Weight (kg)	3.00	3.00
Water (kg)	0.65	0.45
water/sand (w/s)	0.217	0.15

Table 4: Water/sand ratio of the different fillers used

5.2.1.2 WATER DEMAND OF THE CEMENT

In Table 5, the water demand of the cement is tabulated and given as the water/cement ratio (w/c) for the different fillers and filler/cement ratios (f/c) used in the investigations. The water/solids ratios (w/s*) used is also given in Table 5. The results can also be seen in Figure 13. The method proposed to determine the water demand of the cement is based on the spreadability of the concrete base mix. It was found that the optimum range of spreadability is 180 to 196 mm. For this range of spreadability the concrete base mix was not too flow able so that air bubbles can escape and not to dry so that the bubbles break. The mix evaluation data sheet can be found in Appendix A showing the data of the optimum spreadability range only. Figure 34 is given, in Appendix A, showing the results of the flow turn table conducted on the concrete base mix.

From Figure 13 and Table 5 it can be seen that the w/c ratio decreases as the f/c ratio increases. For the same f/c ratio the w/c ratio is much smaller when fine filler is used instead of coarse filler. It should be noted that this w/c should in no way be taken as the w/c for normal concrete and the relationship that exists between normal concrete compressive strength and w/c ratio does not hold for aerated concrete. Rather it should be viewed as the ratio of the “additional water above the threshold” required for the cement to satisfy the conditions for air entrainment to the total weight of the cement for a specific f/c ratio. Also, the amount of water obtained from the w/c ratio is not the total water in the concrete mix and should not be taken as the only amount of water that takes part in the hydration process.

The relationship that exists between f/c and w/c indicates that, if the results were extrapolated, at some value of f/c the required w/c would be small enough to ignore or stabilise at a threshold value. The following scenario is proposed to explain the result. Imagine sand is mixed with water and has a certain spreadability. Cement is then added to the mix checking the spreadability of the mix throughout the process. It will be found that the spreadability of the mix will remain the same until a certain volume of cement is added to the mix, at which level the spreadability will start to change. For this to be true a threshold has to exist. When the w/c ratio is small enough to be neglected or equal to zero, it can be assumed that the w/c ratio is under this threshold.

	Phillippi sand			Malmesbury sand		
f/c	1.0	1.5	2.0	1.0	1.5	2.0
Cement (kg)	2	2	2	2	2	2
Sand (kg)	2	3	4	2	3	4
Water (kg)	0.68	0.80	0.96	0.65	0.71	0.78
w/s*	0.17	0.16	0.96	0.1625	0.142	0.13
w/c	0.123	0.075	0.047	0.175	0.13	0.09

Table 5: Results of the w/s* and the w/c

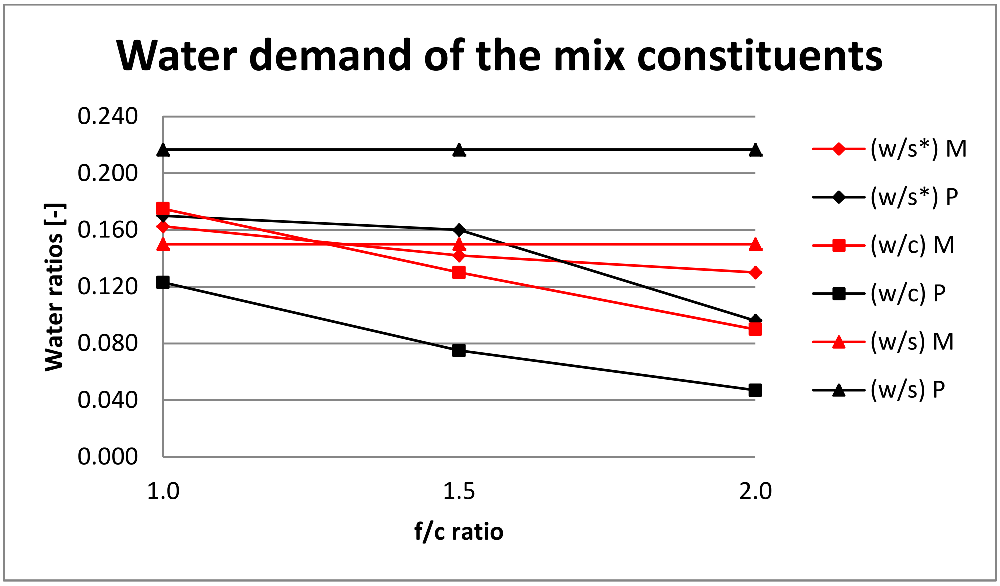

Figure 13: Water demand of the mix constituents

5.3 INFLUENCE OF THE FILLER ON THE CONCRETE

5.3.1 FRESH STATE PROPERTIES

5.3.1.1 EFFECT OF THE FILLER ON THE WATER SOLIDS RATIO

The water/solids (w/s*) ratio is influenced by the filler type (sand, fly-ash) and filler/cement (f/c) ratio. This can be attributed to the water demand of the mix constituents. From Table 5 it can be seen that as the f/c ratio is increased the w/s* ratio decreases. A reason for this is that as f/c ratio is decreased the fine particle size of the cement starts to dominate the dry mix constituents and the required w/s ratio increases. Also, the water demand of the sand dominates when the f/c ratio is increased and the w/s ratio therefore decreases.

5.3.1.2 EFFECT OF THE FILLER ON THE AEA

Modern day AEA's are designed to react with the binder (cement) in the concrete mix, thus for comparison reasons the results should be compared using the ratio of the AEA to the binder. Comparing the mix compositions of the two fillers used, for sand with a poor grading (Phillippi Sand), the ratio of AEA to binder is much lower than that used for sand with a good grading (Malmesbury Sand). This could be attributed to the fact that there may already exist voids in the poorly graded sand because the particle sizes are similar. In the well graded sand, voids may exist but on a smaller scale compared to the poorly graded sand hence a greater ratio of AEA to binder is needed to produce voids for the same density.

5.3.2 MECHANICAL PROPERTIES OF THE CONCRETE

5.3.2.1 COMPRESSIVE STRENGTH OF THE CONCRETE

Figure 14 - Figure 17 depict the results of the compressive strength of the concrete for a filler/cement ratio of 1.25, and 1.5 for a design density of 1600 kg/m^3 and 1800 kg/m^3 at the concrete ages of 7 days and 28 days. In each of the figures the results of the compressive strength is plotted against the density. The type of curing regime used, wrapped curing (WC) and submerged water curing (SC), is also indicated.

The compressive strengths at 7 days and 28 days for the Phillippi sand (poorly graded, fine sand) irrespective of the curing regime used in this investigation, are higher than the concrete containing Malmesbury sand (well graded, coarse sand) in all cases. Kunhanandan Nambiar and Ramamurthy (2006) had a similar result and attributed the difference in strength to the fact that the air bubbles are smaller and more uniformly distributed in fine sand whereas in coarse sand the air bubbles tend to form large air bubbles by clustering.
The results indicate that the concrete compressive strength in these two density classes is not significantly different for the two curing types applied in this research (submerging specimens in water, versus wrapping them in plastic after stripping).

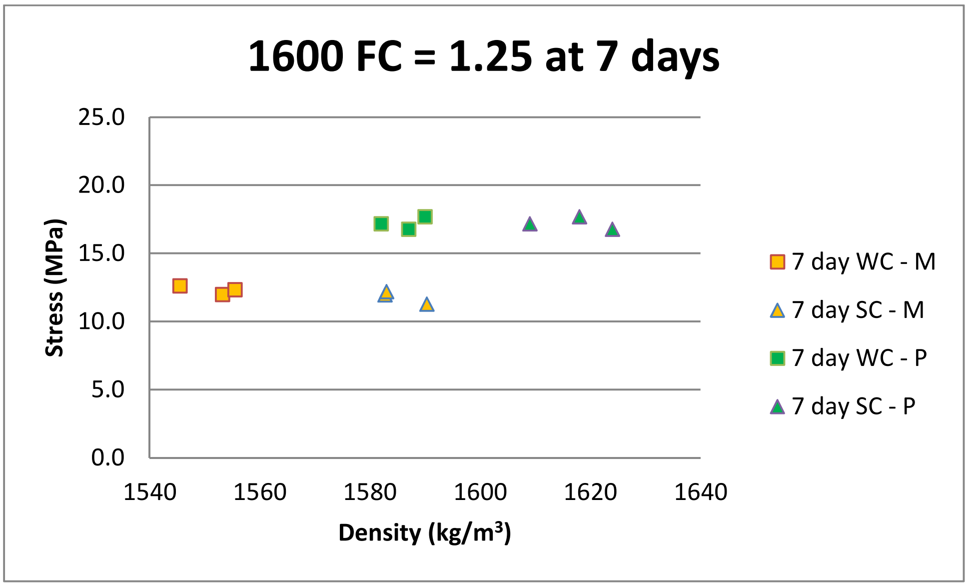

Figure 14: Compressive strength at 7 days for f/c = 1.25

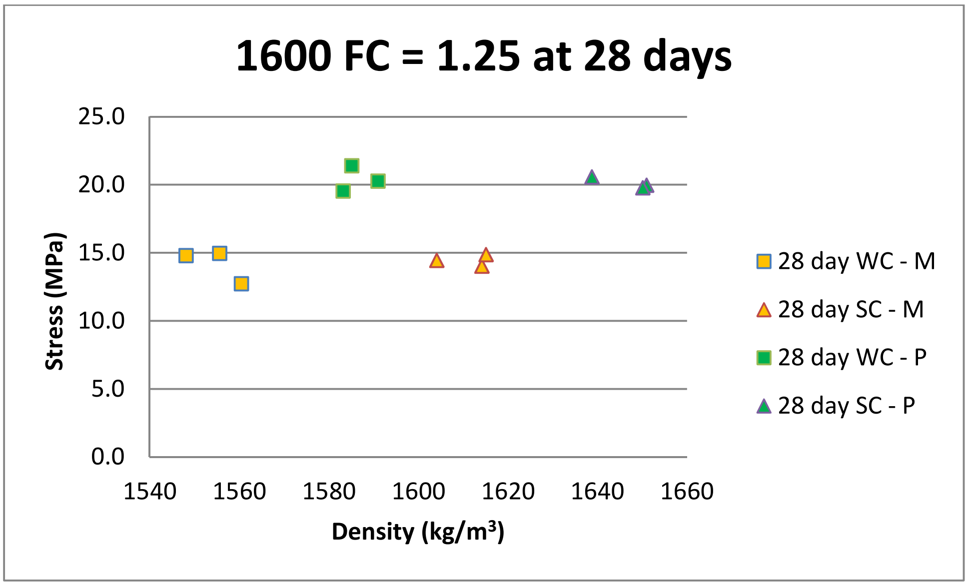

Figure 15: Compressive strength at 28 days for f/c = 1.25

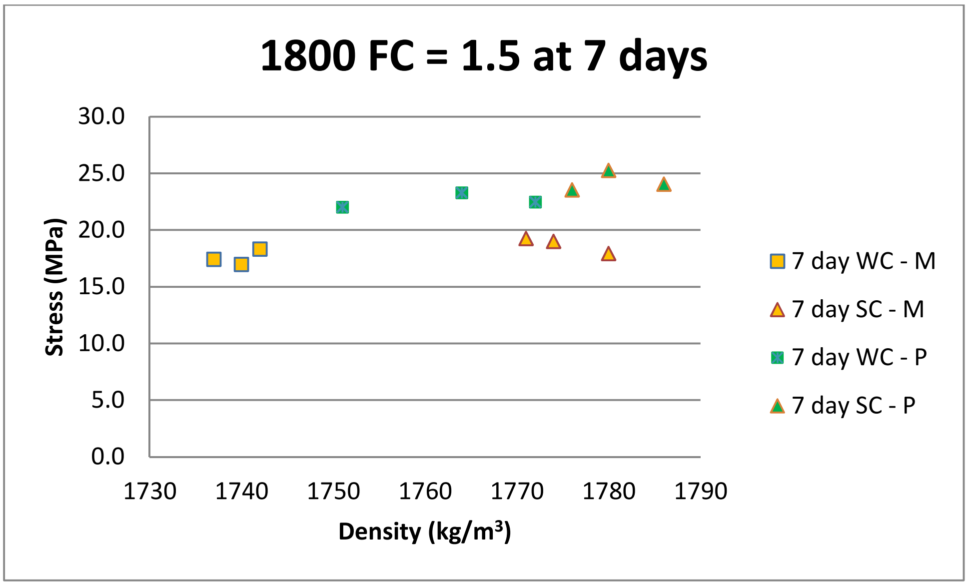

Figure 16: Compressive strength at 7 days for f/c = 1.5

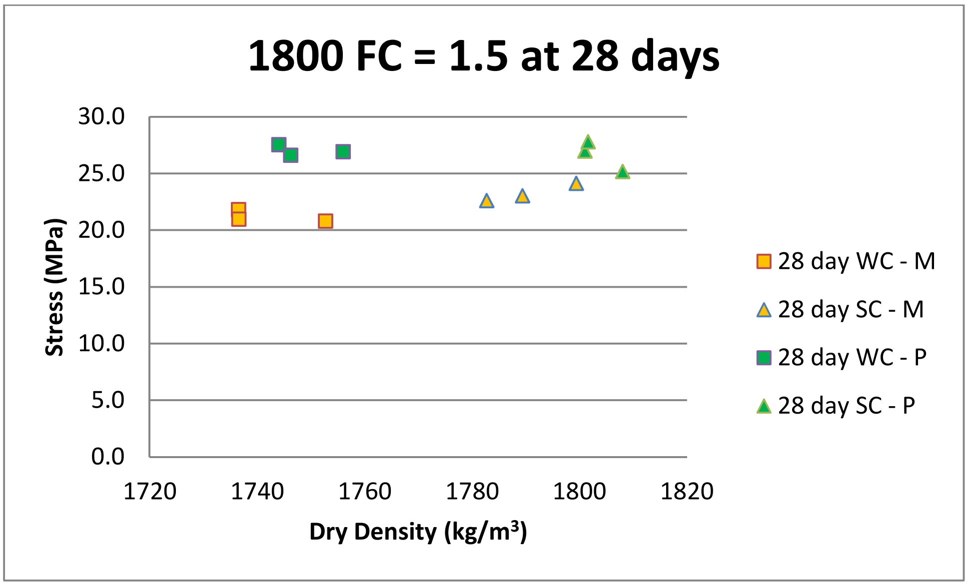

Figure 17: Compressive strength at 28 days for f/c = 1.5

	Phillippi sand				Malmesbury sand			
Density	1600 kg/m^3		1800 kg/m^3		1600 kg/m^3		1800 kg/m^3	
f/c	1.25		1.50		1.25		1.50	
Curing	SC	WC	SC	WC	SC	WC	SC	WC
	18.1	17.2	24.0	22.0	12.0	12.0	19.3	17.4
7 days	17.8	16.8	25.3	23.3	12.2	12.4	17.9	17.0
	17.8	17.7	23.6	22.5	11.3	12.6	19.0	18.3
	20.0	20.3	25.2	26.9	14.0	12.7	22.6	21.8
28 days	19.8	21.4	27.0	27.6	14.9	15.0	23.1	20.8
	20.6	19.6	27.8	26.6	14.4	14.8	24.1	21.0
Average								
7 days	17.9	17.2	24.3	22.6	11.8	12.3	18.7	17.6
28 days	20.1	20.4	26.7	27.0	14.5	14.2	23.3	21.2
CoV(%)								
7 days	0.74	2.69	3.62	2.88	4.03	2.46	3.76	3.88
28 days	2.11	4.60	5.05	1.73	2.91	8.79	3.36	2.56

Table 6: Compressive strength of the concrete at 7 and 28 days

5.3.2.2 TENSILE SPLITTING STRENGTH OF THE CONCRETE

The results of the tensile splitting test of the concrete mixes at age 28 days are tabulated in Table 7. The concrete tensile splitting test was done according to SANS standards. In all cases the results show that the tensile strength of the concrete cured in water is greater than that of concrete cured by the wrapped method. Also, the concrete containing Phillippi sand (fine sand) has a higher tensile strength value than the Malmesbury sand (coarse sand).

For the 1800 kg/m^3 density concrete mix containing Phillippi sand the concrete strength class can be considered as C25. Comparing the value of the average tensile splitting strength to the value of tensile splitting strength for lightweight aggregate concrete of the same strength class given in BS EN 1992 1-1, the value of the tensile splitting strength is slightly higher but within a range of 1 MPa. The same observation was made for the 1600 kg/m^3 density concrete mix for Phillippi sand, 1800 kg/m^3 density concrete mix for Malmesbury sand for a concrete strength class of C20 and 1600 kg/m^3 density concrete mix for Malmesbury sand for a concrete strength class of C15.

	Malmesbury Sand				Phillippi Sand			
	1600 kg/m^3		1800 kg/m^3		1600 kg/m^3		1800 kg/m^3	
Curing	WC	SC	WC	SC	WC	SC	WC	SC
	2.05	2.00	2.29	2.61	2.35	2.83	2.87	2.79
	1.93	2.17	2.64	2.68	2.22	2.76	2.61	3.18
	1.98	2.12	2.38	2.87	2.58	2.75	3.01	2.85
Average	1.99	2.10	2.43	2.72	2.38	2.78	2.83	2.94
CoV(%)	3.0%	4.2%	7.5%	4.9%	7.6%	1.6%	7.2%	7.1%
BS EN 1992	1.49	1.49	1.88	1.88	1.76	1.76	2.18	2.18

Table 7: Results of the tensile splitting strength test in MPa

5.3.3 ELASTIC MODULUS OF THE CONCRETE

The results of the E-modulus test conducted at a concrete age of 28 days on all the mixes are given in Table 8. The results of the 1800 kg/m^3 Malmesbury sand mix was calculated from two LVDTs (instead of three, due to a faulty third LVDT), which resulted in a large variation in the results calculated. The difference that resulted in the average E-modulus between the dry/wrapped curing and the wet curing is 4.5 GPa, whereas in all other cases the difference is 0.5 GPa. The values of the E-modulus can also be compared to the E-modulus values for general purposes given for lightweight concrete of the same strength class in the BS EN 1992 1-1. For the same strength classes as discussed in the previous section, it was observed that the E-modulus of all the mixes is below the values given but within a range of 5 GPa. Note that the values given by the standard are nominal values, and are known to be different when different aggregate is used. Here, it is compared merely for illustration purpose.

	Malmesbury Sand				Phillippi Sand			
	1600 kg/m^3	1800 kg/m^3			1600 kg/m^3		1800 kg/m^3	
Curing	WC	WC	WC	SC	WC	SC	WC	SC
	10.0	10.5	18.0	15.5	12.0	14.5	17.0	-
	10.0	10.5	12.0	22.0	13.0	10.0	16.5	16.5
	10.5	10.5	11.0	16.5	12.5	13.5	17.5	18.0
Average	10.0	10.5	13.5	18.0	12.5	12.5	17.0	17.5
CoV(%)	2.8%	0.0%	27.7%	19.4%	4.0%	18.7%	2.9%	6.1%
BS EN 1992	14.0	14.0	19.5	19.5	15.0	15.0	20.0	20.0

Table 8: Results of the elastic modulus test in GPa

5.4 STRENGTH DEVELOPMENT OF THE CONCRETE

The topic of strength of aerated concrete has been elaborated in section 3.4 *Mix design considerations* and in section 4.2.2 *Strength development in aerated concrete*. A prior study, conducted on aerated concrete, has shown that the 7 day compressive strength of aerated concrete was greater than 70 per cent of the 28 day compressive strength of the concrete. Generally, in practice, it is accepted that the 7 day compressive strength is about 70 per cent of the 28 day compressive strength depending on the type of cement used (i.e. N - Normal, R - Rapid, S - Slow). An advantage of establishing a relationship between compressive strength and concrete age is that it can be very useful for contractors. For instance, during construction, contractors can use this relationship to estimate the time required for the concrete to gain sufficient strength so that the formwork can be removed and construction can continue without causing any damage to the concrete, . Therefore, it was decided to design an experiment that investigates the strength development of aerated concrete subject to continuous submerged water curing and the factors that influence the strength development of the concrete.

5.4.1 EXPERIMENTAL RESULTS

Figure 12 shows a flow diagram depicting the factors that influence the strength of the concrete. The results of the experiment will therefore be presented in such a fashion so as to show how these factors influence the mechanical properties (compressive strength, tensile strength) of the concrete. The factors influencing the strength of aerated concrete will be presented and discussed in the following sequence: density, mix proportions (f/c, s/c) and filler type (Malmesbury sand, Phillippi sand). The other factors, such as the cement type and curing are not discussed here. However it should be noted that a higher quality of cement would produce a higher strength to density ratio and for the curing it was concluded from the results of the compressive strength of the concrete that it was not significantly affected by two types of curing regimes used (submerged water curing, wrapped curing). The influence the factors have on the strength of the concrete is evaluated by the compression strength test. The results of the tensile strength test and the modulus of elasticity are also given.

5.4.2 THE EFFECT OF DENSITY ON THE CONCRETE

5.4.2.1 COMPRESSIVE STRENGTH

Four figures are used to illustrate the influence that the concrete density has on the compressive strength of the concrete. Figure 18 - Figure 19 depict the results of the compressive strength test for the 9 different concrete mixes at a concrete age of 28 days for the two different fillers used and Figure 20 - Figure 21 depict the results of the compressive strength test for the 9 different concrete mixes at a concrete age of 21 days. The letters M and P denote the different filler (i.e. M - Malmesbury sand, P - Phillippi sand) used as a mix constituent in the concrete mixes respectively. The result of the compressive strength of each concrete cube is indicated in the figures, as well as the compressive strength of the concrete at the age of testing, taken as the average of 3 concrete cubes. A regression line is plotted through the compressive strength of the concrete made from the same f/c ratio. The regression line with the R^2 value closest to the value of 1 was taken as the regression line that best represents the trend of the compressive strength data.

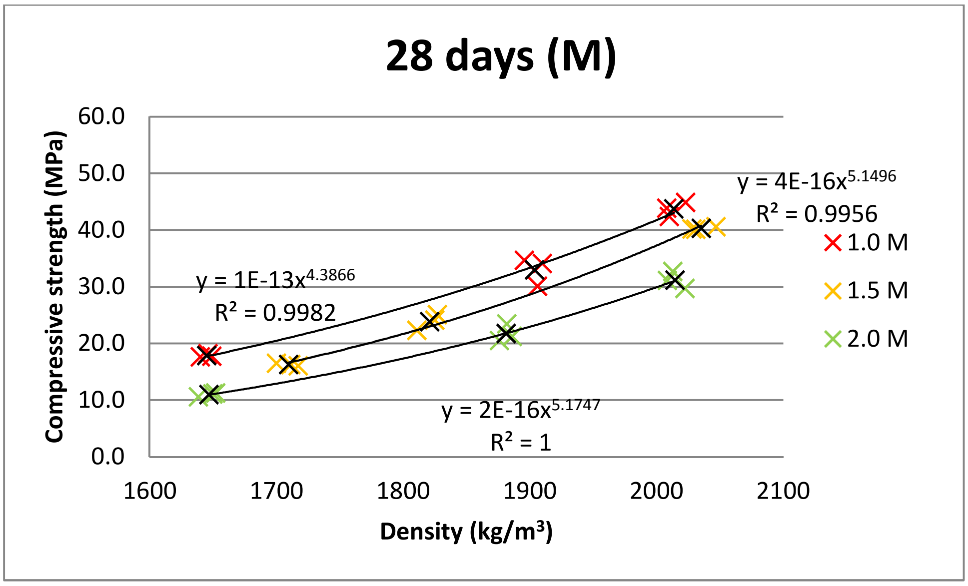

Figure 18: Density versus compressive strength (Malmesbury sand - 28 days)

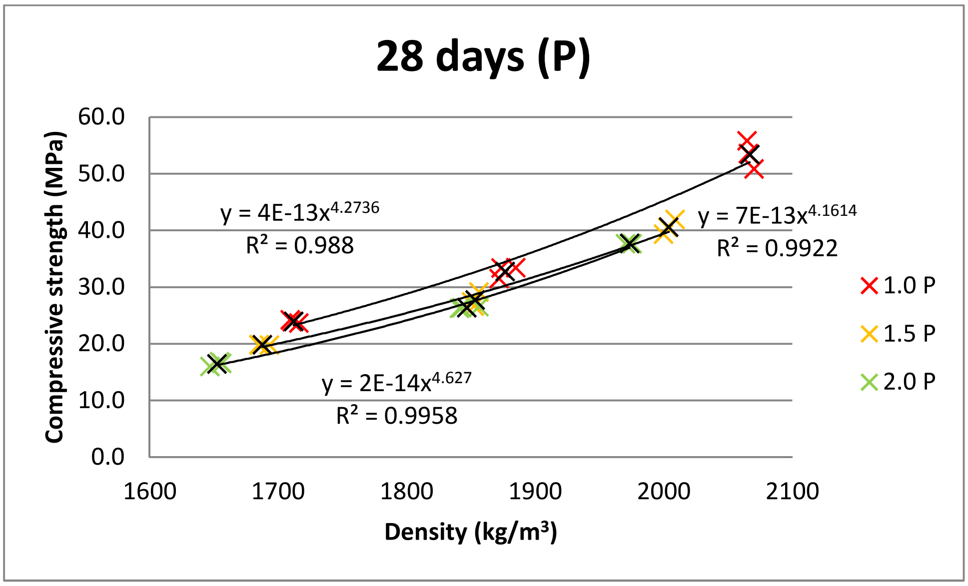

Figure 19: Density versus compressive strength (Phillippi sand - 28 days)

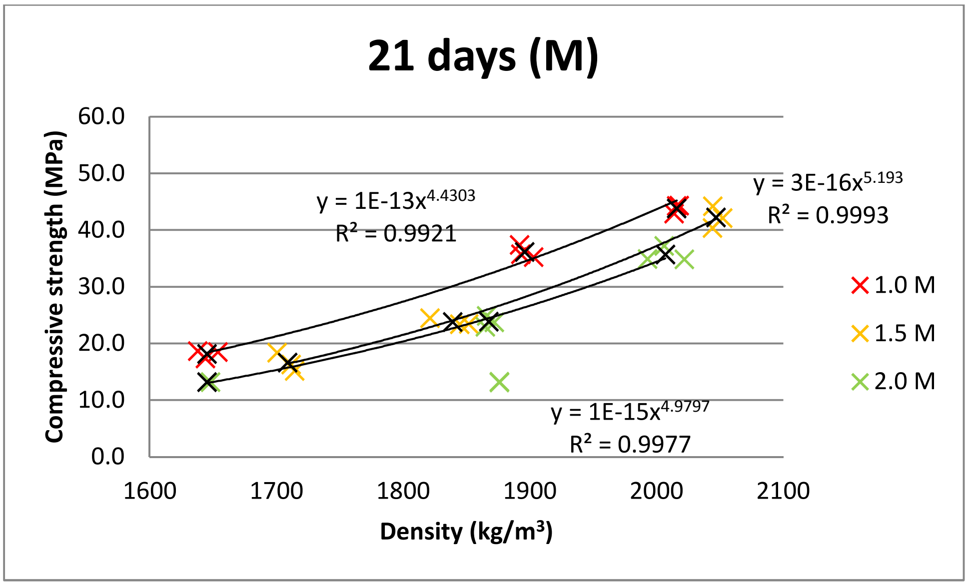

Figure 20: Density versus compressive strength (Malmesbury sand - 21 days)

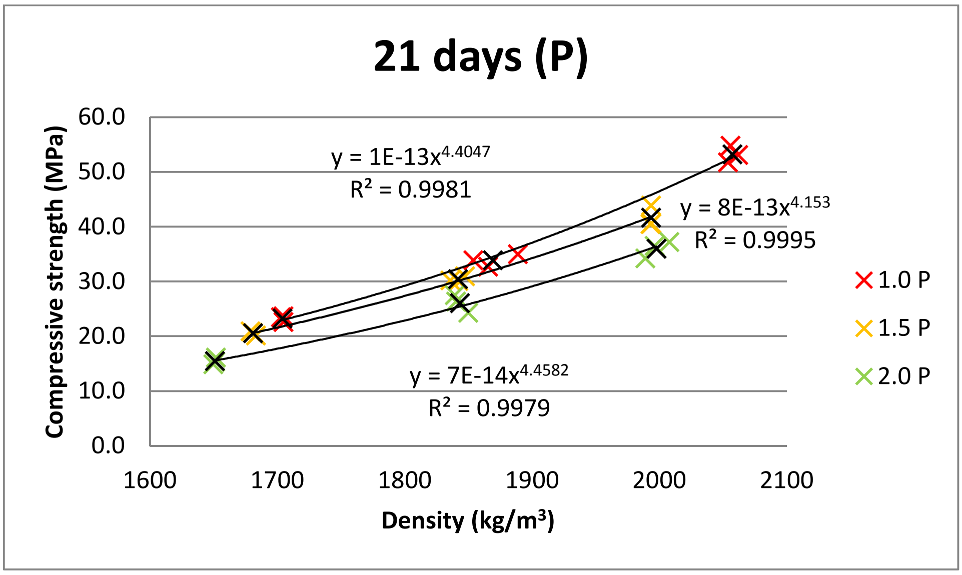

Figure 21: Density versus compressive strength (Phillippi sand - 21 days)

From Figure 18 - Figure 21 it can be seen that with an increase in concrete density there is a non-linear increase in concrete compressive strength. Regression analysis shows that the trends that best describe the relationship between concrete density and concrete compressive strength can be given as a power function. This power function written in standard form is given as equation (5.1). The coefficient and exponent used to describe the compressive strength given the density of the concrete and f/c ratio is given in Table 9 and Table 10 for the concrete at different ages. In all cases, the concrete mixes designed for a concrete density of 2000 kg/m^3 yielded the largest compressive strength out of the three chosen design target densities. The concrete mixes designed for 1800 kg/m^3 yielded the 2^{nd} highest compressive strength followed by the 1600 kg/m^3 concrete design target density mixes. Also, for concrete mixes designed for the same design target density, say 2000 kg/m^3, the results recorded for the compressive strength of the concrete were higher for low f/c. This trend is consistent throughout the data. The results displayed in Figure 18 and Figure 19 depict the results of the compressive strength test for the Malmesbury sand and Phillippi sand at a concrete age of 28 days, respectively. Comparing the results displayed in the two figures, it is observed that the results in Figure 19 are higher than those in Figure 18. This indicates the compressive strength for the concrete mixes designed for the same target densities with the same f/c ratio are higher for concrete mixes made with finer sand. The compressive strength to density ratio is therefore increased when fine sand is used as filler compared to using coarse sand as filler. This result can also be seen when comparing Figure 20 and Figure 21. The effect that the density has on the concrete compressive strength has only been shown here for concrete ages of 21 and 28 days because in nearly all cases the compressive strength of the concrete at the age of 21 days is higher than the compressive strength at 28 days. Also, it needed to be seen that the effect the concrete density has on the compressive strength does not change with age. For this reason, the effect the concrete density has on the concrete at the ages of 7 and 14 days has been omitted here, however, the figures have been included in APPENDIX B.

$$\sigma_{comp} = ax^b \tag{5.1}$$

where

σ_{comp} = the compressive strength of the concrete in MPa

a = the coefficient of the equation relating the density to the compressive strength, found in Table 9 and Table 10 depending on the type of filler used

x = the density of the concrete in kg/m^3

b = the exponent of the equation relating the density to the compressive strength, found in Table 9 and Table 10 depending on the type of filler used

Malmesbury sand									
f/c		1.0			1.5			2.0	
Age	a	b	R^2	a	b	R^2	a	b	R^2
7	$2x10^{-13}$	4.3138	0.9998	$7x10^{-17}$	5.3502	0.9872	$1x10^{-14}$	4.6399	0.9992
14	$5x10^{-15}$	4.8222	0.9956	$2x10^{-15}$	4.916	0.9989	$2x10^{-16}$	5.2365	0.9981
21	$1x10^{-13}$	4.4308	0.9921	$3x10^{-16}$	5.193	0.9993	$1x10^{-15}$	4.9797	0.9977
28	$1x10^{-13}$	4.3866	0.9982	$4x10^{-16}$	5.1496	0.9956	$2x10^{-16}$	5.1747	1.000

Table 9: Coefficients and exponents relating the density of the concrete to the compressive strength (Malmesbury sand)

Phillippi sand									
f/c		1.0			1.5			2.0	
Age	a	b	R^2	a	b	R^2	a	b	R^2
7	$5x10^{-12}$	3.9269	0.9966	$3x10^{-12}$	3.96	0.9961	$9x10^{-13}$	4.1078	1.000
14	$7x10^{-13}$	4.1836	0.9997	$1x10^{-14}$	4.717	0.9991	$4x10^{-13}$	4.2221	0.9905
21	$1x10^{-13}$	4.4047	0.9981	$8x10^{-13}$	4.135	0.9995	$7x10^{-14}$	4.4582	0.9979
28	$4x10^{-13}$	4.2736	0.988	$7x10^{-13}$	4.1614	0.9922	$2x10^{-14}$	4.827	0.9958

Table 10: Coefficients and exponents relating the density of the concrete to the compressive strength (Phillippi sand)

5.4.2.2 TENSILE SPLITTING STRENGTH

Two figures are used to illustrate the influence of density on the tensile splitting strength of the concrete. Figure 22 and Figure 23 show these results for the 9 different concrete mixes made with the two different fillers, respectively. The result of the tensile splitting test of each concrete cube is indicated in the figures, as well as the tensile splitting strength of the concrete, taken as the average of 3 concrete cubes. A regression line is plotted through the compressive strength of the concrete made from the same f/c ratio. The regression line with the R^2 value closest to the value of 1 was taken as the regression line that best represents the trend of the tensile splitting strength data.

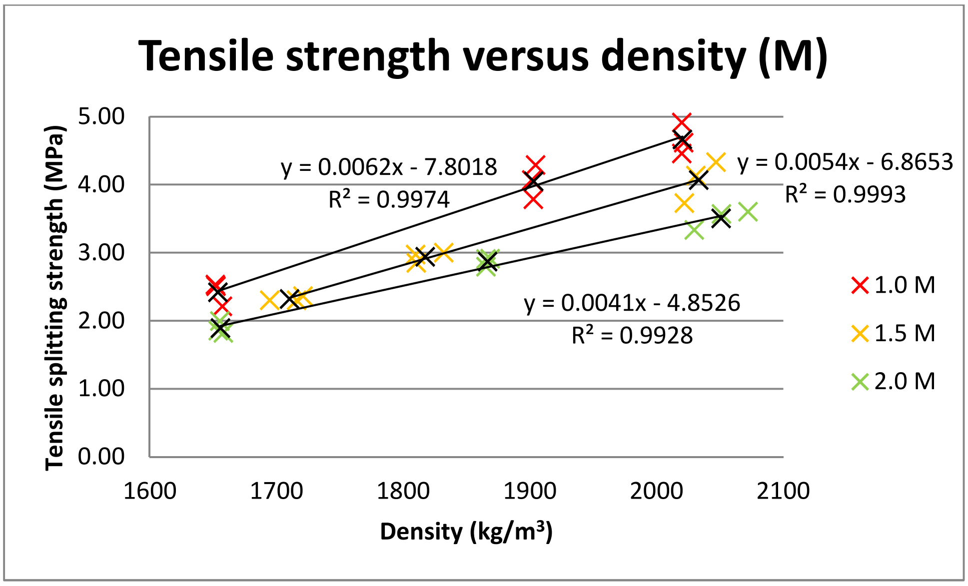

Figure 22: Tensile splitting strength versus density (Malmesbury sand)

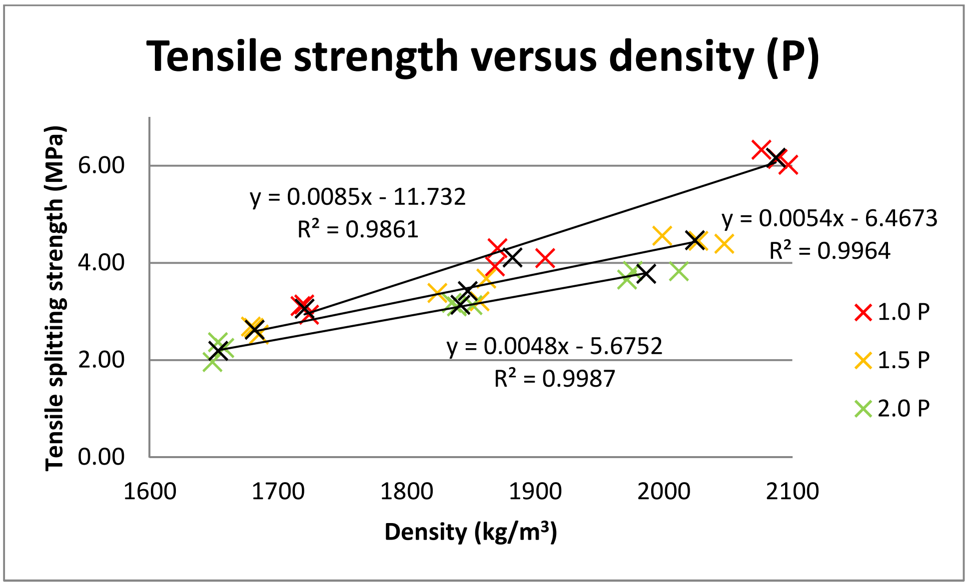

Figure 23: Tensile splitting strength versus density (Phillippi sand)

From Figure 22 - Figure 23 it can be seen that with an increase in concrete density there is a linear increase in the tensile splitting strength of the concrete. Regression analysis shows that a linear exists relationship between concrete density and concrete tensile splitting strength. The equation describing this relationship can be found in equation (5.2). It should be noted that no attempt was made to produce a tensile splitting strength prediction model and that this equation should not be taken as such but rather as a general equation showing how the tensile splitting varies with an increase in concrete density. The coefficient and exponent used to describe the tensile splitting strength given the density of the concrete and f/c ratio is given in Table 11 and Table 12 for the concrete. In all cases, a higher concrete density resulted in a higher tensile splitting strength. Another observation that can be made is that the concrete mixes designed for the same target concrete density but made with a lower f/c ratio also resulted in an increase in tensile splitting strength.

$$\sigma_{split} = ax + b \tag{5.2}$$

where

σ_{split} = concrete splitting strength in MPa

a = the coefficient of the equation relating the density to the tensile splitting strength, found in Table 11 and Table 12 depending on the type of filler used

x = density of the concrete in kg/m^3

b = the constant term of the equation relating the density to the tensile splitting strength, found in Table 11 and Table 12 depending on the type of filler used

Malmesbury sand			
f/c	1.00	1.50	2.00
a	0.0062	0.0054	0.0041
b	-7.8018	-6.8653	-4.8526
R^2	0.9974	0.9993	0.9928

Table 11: Coefficients and exponents relating the density of the concrete to the tensile splitting strength (Malmesbury sand)

Phillippi sand			
f/c	1.00	1.50	2.00
a	0.0085	0.0054	0.0048
b	-11.732	-6.4673	-5.6752
R^2	0.9861	0.9964	0.9987

Table 12: Coefficients and exponents relating the density of the concrete to the tensile splitting strength (Phillippi sand)

5.4.2.3 MODULUS OF ELASTICITY

Figure 24 and Figure 25 illustrate the effect the concrete density has on the modulus of elasticity of the concrete. Figure 24 shows the results of the elastic modulus of six different concrete mixes using Malmesbury sand as filler and Figure 25 shows the results of the elastic modulus of six different concrete mixes using Phillippi sand as filler. Three of the six concrete mixes were designed for a concrete design target density of 1600 kg/m^3 and the remaining concrete mixes were designed for a design target density of 1800 kg/m^3. The f/c ratios used for the concrete mixes were 1.0, 1.5 and 2.0, respectively. In both figures (Figure 24 and Figure 25) the elastic modulus of the concrete is plotted against the concrete density before testing, obtained by calculating the density from the known weight and volume of the concrete cylindrical sample. In both figures it is observed that with an increase in density there is an increase in the modulus of elasticity of the concrete. In Figure 24, for the Malmesbury sand, it seems that there is no way to predict which concrete mix, out of the three concrete mixes designed for the same target, but having a different f/c ratio would have the highest elastic modulus. This seems to be random because for the 1600 kg/m^3 design target density mixes the highest recorded elastic modulus is for the concrete made with a f/c ratio of 1.5 and for a design target density of 1800 kg/m^3 f/c ratio of 1.0 was recorded to be the highest of the three concrete mixes. However, in Figure 25 for the Phillippi sand, the concrete mixes made with f/c ratios of 1.0 were recorded to give the highest elastic modulus for the respective target densities. In should be noted that in all cases the concrete mixes with f/c ratios of 2.0 were recorded to give the lowest result for the modulus of elasticity for the concrete mixes.

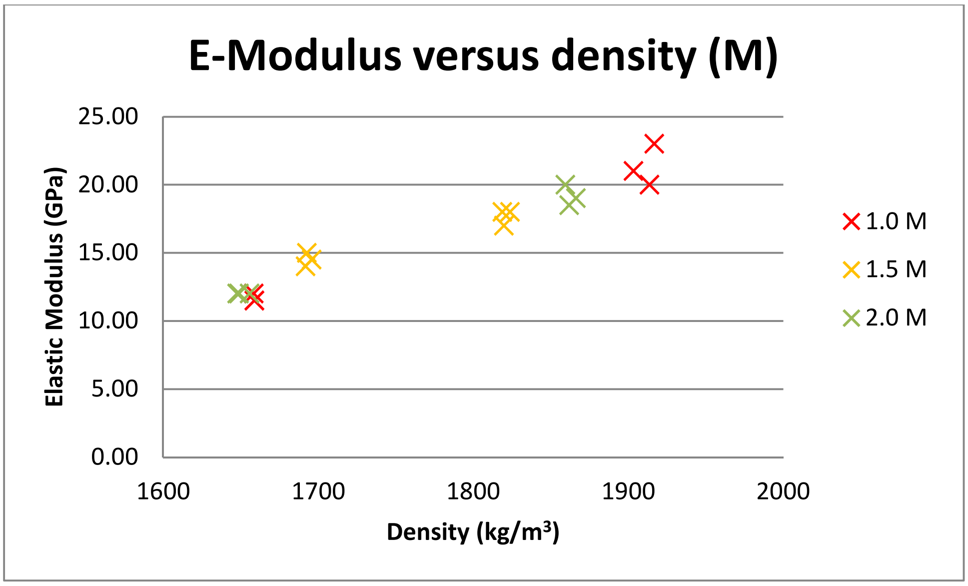

Figure 24: Elastic Modulus versus density (M)

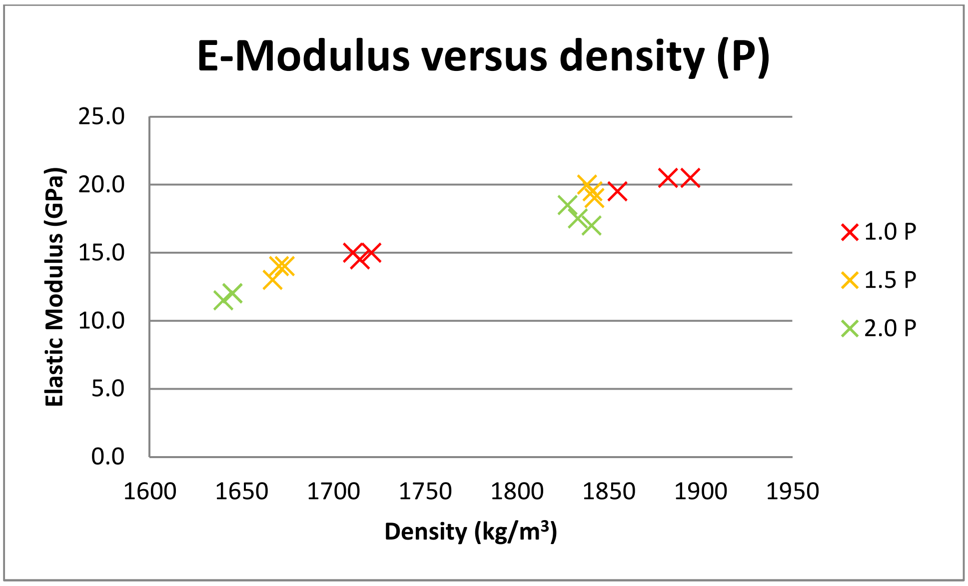

Figure 25: Elastic modulus versus density (P)

5.4.3 THE EFFECT OF THE FILLER ON THE CONCRETE

5.4.3.1 COMPRESSIVE STRENGTH

With the aid of six figures the effect that the filler has on the compressive strength of the concrete can be illustrated. Figure 26 - Figure 28 show the results of the compressive strength test of six different concrete mixes each at concrete ages of 7, 14, 21 and 28 days. For the six different concrete mixes depicted in the figures, three of the concrete mixes are designed for a specific design target density using Malmesbury sand as filler with f/c ratios of 1.0, 1.5 and 2.0. The remaining three mixes are designed for the same design target density as the first three mixes; however Phillippi sand is used as filler instead. Figure 26, Figure 27 and Figure 28 show the results of the compressive strength of the concrete at concrete ages of 7, 14, 21 and 28 days for design target densities of 2000 kg/m^3, 1800 kg/m^3 and 1600 kg/m^3. Figure 31, Figure 32 and Figure 33 shows the normalized result of the compressive strength of the concrete at various concrete ages for design target densities of 2000 kg/m^3, 1800 kg/m^3 and 1600 kg/m^3.

In Figure 26, the results of the compressive strength test of six different concrete mixes designed for a concrete density of 2000 kg/m^3 are shown. Three of the concrete mixes were designed using Malmesbury sand with f/c ratio of 1.0, 1.5 and 2.0. Comparing the results of the three concrete mixes made with Malmesbury sand as filler shows that, the compressive strength of the concrete made with f/c ratio of 1.0 is the highest for the concrete ages of 14, 21 and 28 days of the three concrete mixes. At the concrete age of 7 days the compressive strength of the concrete made with f/c ratio of 1.5 was recorded to give the highest compressive strength. The compressive strength of the concrete mix made with f/c of 2.0 is recorded to be the lowest of the three concrete mixes at the concrete ages of 7, 14, 21, and 28 days.

For the three concrete mixes using Phillippi sand as filler, the compressive strength of the concrete mix made with f/c ratio of 1.0 was the highest for the concrete ages of 7, 14, 21 and 28 days. The concrete mix using f/c ratio of 1.5 gave the second highest compressive strength for all concrete ages and the concrete mix made with f/c of 2.0 yielded the lowest compressive strength of the three concrete mixes.

Comparing the results of the compressive strengths of the concrete mixes made with the same f/c ratio shows that the concrete mixes made with Phillippi sand gives a higher compressive strength for all concrete ages. For the case of the concrete mixes made with f/c ratio 1.5, the results from the compressive strength test yields compressive strengths for the two concrete mixes, 2000 1.5 M and 2000 1.5 P, that are relatively close to each other. Therefore, the line representing the results of the compressive strength of the concrete mix 2000 1.5 M is obscured by the line representing the compressive strength of the 2000 1.5 P concrete mix.

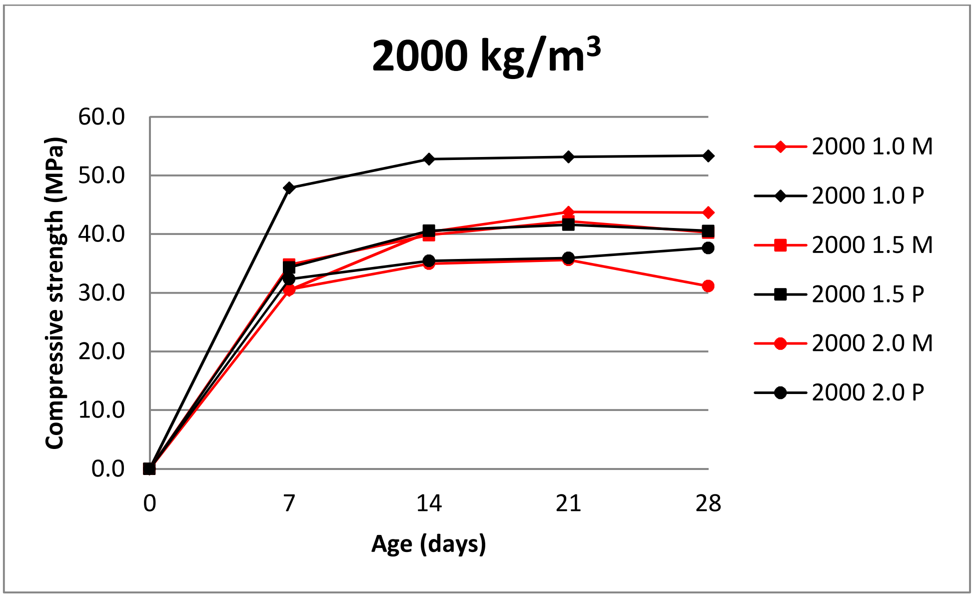

Figure 26: Concrete compressive strength over time for concrete target density of 2000 kg/m^3. Illustrating the influence of the filler on the compressive strength of the concrete

In Figure 27, the results of the compressive strength test of six different concrete mixes designed for a concrete density of 1800 kg/m^3 are shown. Three of the concrete mixes were designed using Malmesbury sand with f/c ratio of 1.0, 1.5 and 2.0. Comparing the results of the three concrete mixes made with Malmesbury sand as filler shows that, the compressive strength of the concrete made with f/c ratio of 1.0 is the highest for the concrete ages of 7, 14, 21 and 28 days of the three concrete mixes. At the concrete age of 7 days the compressive strength of the concrete made with f/c ratio of 1.5 was recorded to give the lowest compressive strength. The compressive strength of the concrete mix made with f/c of 2.0 yielded the second highest compressive strengths of the three concrete mixes at the concrete ages of 7, 14, 21 days. The compressive strength of the concrete mix made with f/c ratio of 1.5 gave the lowest compressive strength of the three concrete mixes at the concrete ages of 7, 14 and 21 days but gave the second highest compressive strength at the concrete age of 28 days.

For the three concrete mixes using Phillippi sand as filler, the compressive strength of the concrete mix made with f/c ratio of 1.0 was the highest of the concrete mixes at the concrete ages of 7, 14, 21 and 28 days. The second highest compressive strengths were recorded for the concrete mix made with f/c ratio of 1.5 and the concrete mix made with f/c of 2.0 yielded the lowest compressive strength of the three concrete mixes.

Comparing the results of the compressive strengths of the concrete mixes made with the same f/c ratio shows that those containing Phillippi sand give a higher compressive strength for the concrete mixes for all ages but at the concrete age of 21 days the concrete mix using Malmesbury sand as filler with f/c ratio 1.0 yielded a greater compressive strength than the concrete made using Phillippi sand as filler for the same f/c ratio.

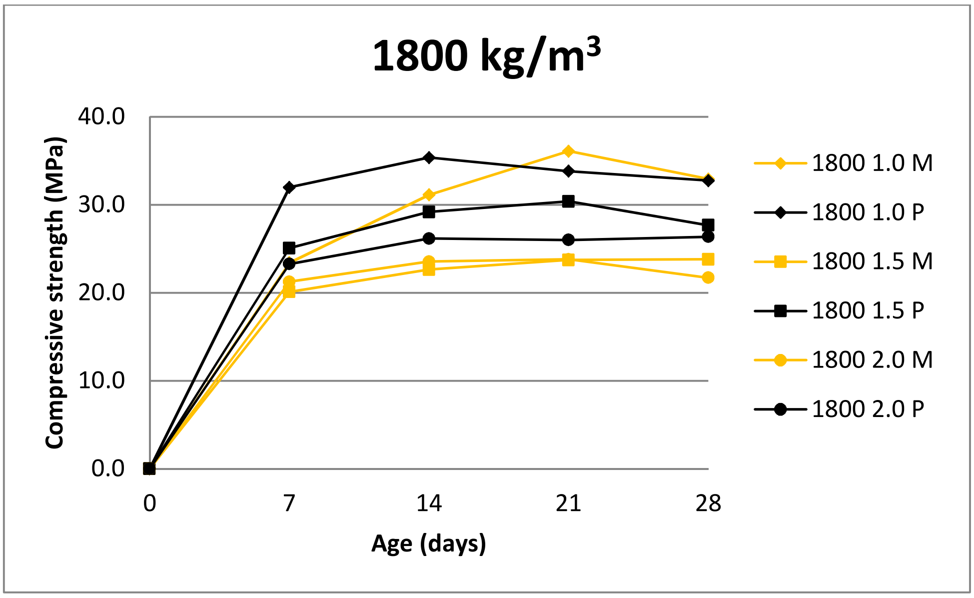

Figure 27: Concrete compressive strength over time for concrete target density of 1800 kg/m^3. Illustrating the influence of the filler on the compressive strength of the concrete

In Figure 28, the results of the compressive strength test of six different concrete mixes designed for a concrete design target density of 1600 kg/m^3 are shown. Three of the concrete mixes were designed using Malmesbury sand with f/c ratio of 1.0, 1.5 and 2.0. Comparing the results of the three concrete mixes made with Malmesbury sand as filler shows that, the compressive strength of the concrete made with f/c ratio of 1.0 is the highest for the concrete ages of 21 and 28 days of the three concrete mixes. At the concrete age of 7 and 14 days the compressive strength of the concrete made with f/c ratio of 1.5 was recorded to give the highest compressive strength. The compressive strength of the concrete mix made with f/c of 2.0 yielded the lowest compressive strengths of the three concrete mixes at the concrete ages of 7, 14, 21 and 28 days.

For the three concrete mixes using Phillippi sand as filler, the compressive strength of the concrete mix made with f/c ratio of 1.0 was the highest of the concrete mixes at the concrete ages of 7, 14, 21 and 28 days. The second highest compressive strengths were recorded for the concrete mix made with f/c ratio of 1.5 and the concrete mix made with f/c of 2.0 yielded the lowest compressive strength of the three concrete mixes.

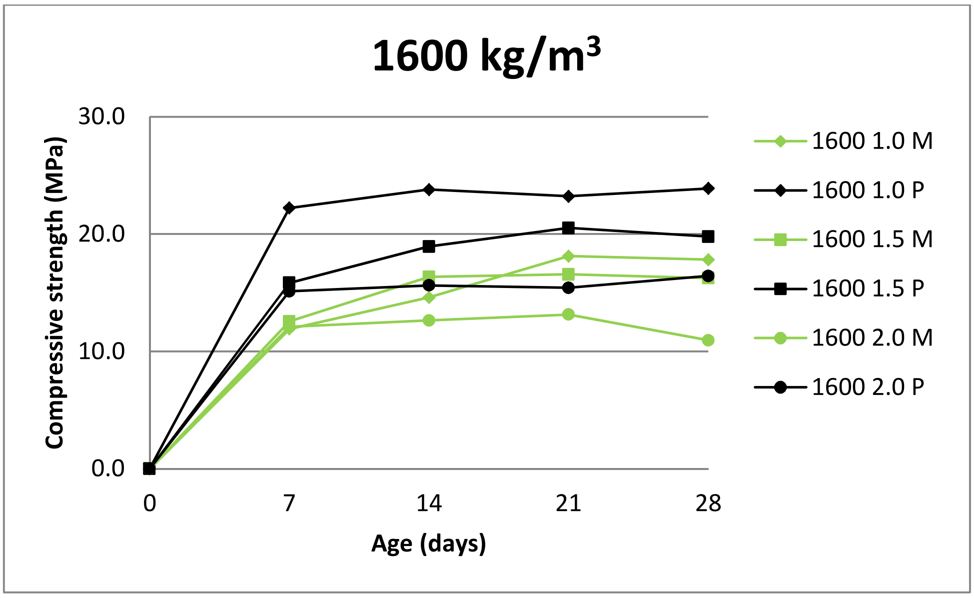

Figure 28: Concrete compressive strength over time for concrete target density of 1600 kg/m^3. Illustrating the influence of the filler on the compressive strength of the concrete

Comparing the results of the compressive strengths of the concrete mixes made with the same f/c ratio shows that the concrete mixes made with Phillippi sand give a higher compressive strength for all concrete ages. The results of the concrete compressive strength for the 9 concrete mixes made using Malmesbury sand as filler are tabulated in Table 13 and the results of the concrete compressive strength for the 9 concrete mixes made using Malmesbury sand as filler are tabulated in Table 14.

In Table 13 and Table 14 the results of the compressive strength, compressive strength averages and coefficient of variation (CoV) is given. In Table 13, for the Malmesbury sand, the CoV, expressed as a percentage, of all the concrete mixes are in the range of 0.5 to 10 except for the concrete mix designed for a design target density of 1600 kg/m^3 made with a f/c of 1.5 at the concrete age of 7 days, which resulted in a CoV of 26.43. This is clearly an outlier but the results of the compressive strength test for this mix do meet the acceptance criteria. It is normally accepted that the result from a sample of a data set that falls outside 2 standard deviations from the average of the data set is excluded from the results. This is not the case for this data set as the two extreme values are within 1.5 times the standard deviation. In Table 14, for the Phillippi sand, the CoV of the concrete mix are all in the range of 0.05 to 9.0. In this case there are no outliers.

Malmesbury sand									
Density	1600 kg/m^3			1800 kg/m^3			2000 kg/m^3		
f/c	1.00	1.50	2.00	1.00	1.50	2.00	1.00	1.50	2.00
	11.4	8.70	11.2	21.8	20.3	21.0	30.2	34.0	30.3
7 days	12.1	14.2	13.1	23.3	19.5	21.1	30.3	36.3	30.4
	12.2	14.7	12.0	25.1	20.5	21.6	30.9	34.1	31.0
	13.7	15.9	13.6	31.1	22.4	24.0	40.8	39.5	34.6
14 days	14.9	16.5	12.2	31.4	23.2	22.6	37.7	39.7	34.3
	15.2	16.6	12.2	30.9	22.3	24.1	42.5	40.3	36.0
	18.4	18.3	13.2	37.4	24.4	24.8	44.2	42.1	34.9
21 days	17.3	16.2	13.1	35.7	23.4	22.9	42.9	44.2	37.2
	18.6	15.1	13.1	35.2	23.4	23.7	44.3	40.3	34.8
	18.2	16.2	10.6	34.1	24.1	20.5	44.9	40.5	29.7
28 days	17.7	16.5	11.2	30.1	22.3	21.2	42.4	40.2	31.1
	17.6	16.0	11.1	34.6	25.0	23.4	43.8	40.1	32.7
Average									
7 days	11.9	12.6	12.1	23.4	20.1	21.3	30.5	34.8	30.6
14 days	14.6	16.4	12.6	31.1	22.6	23.6	40.3	39.9	35.0
21 days	18.1	16.6	13.1	36.1	23.7	23.8	43.8	42.2	35.6
28 days	17.8	16.2	11.0	32.9	23.8	21.7	43.7	40.3	31.1
CoV(%)									
7 days	3.74	26.43	7.79	6.92	2.58	1.44	1.23	3.66	1.26
14 days	5.48	2.27	6.42	0.79	2.15	3.45	6.06	1.10	2.52
21 days	3.85	9.78	0.52	3.14	2.50	4.01	1.82	4.56	3.78
28 days	1.81	1.37	3.29	7.59	5.89	6.93	2.84	0.55	4.85

Table 13: Results of the concrete compressive strength test using Malmesbury sand as filler

	Phillippi sand								
Density	1600 kg/m^3			1800 kg/m^3			2000 kg/m^3		
f/c	1.00	1.50	2.00	1.00	1.50	2.00	1.00	1.50	2.00
7 days	23.2	16.6	14.9	31.5	25.2	22.7	49.5	34.1	33.8
	21.4	14.2	15.8	31.9	25.0	23.3	47.1	34.3	31.7
	22.1	16.8	14.7	32.5	25.0	23.8	47.1	34.5	31.6
14 days	24.2	19.5	16.6	34.5	29.0	25.9	52.0	40.9	35.1
	23.4	19.1	15.0	35.0	29.8	25.8	54.0	39.4	36.3
	23.8	18.3	15.3	36.5	28.8	26.9	52.4	41.5	35.0
21 days	23.6	20.1	14.9	32.7	30.2	26.3	53.1	40.5	37.2
	23.5	20.9	16.1	33.8	30.1	27.5	54.7	43.8	34.2
	22.5	20.6	15.3	34.9	30.9	24.3	51.7	40.5	36.4
28 days	23.6	19.8	16.6	31.5	26.6	26.5	50.8	26.6	37.6
	23.9	19.8	16.8	33.3	29.1	26.2	53.5	29.1	37.7
	24.2	19.8	16.0	33.4	27.3	26.4	55.8	27.3	37.6
Average									
7 days	22.2	15.8	15.1	32.0	25.1	23.3	47.9	34.3	32.4
14 days	23.8	18.9	15.6	35.4	29.2	26.2	52.8	40.6	35.5
21 days	23.2	20.5	15.4	33.8	30.4	26.0	53.2	41.6	35.9
28 days	23.9	19.8	16.4	32.7	27.7	26.4	53.4	27.7	37.7
CoV(%)									
7 days	4.10	8.96	3.80	1.44	0.54	2.32	2.82	0.58	3.83
14 days	1.77	3.24	5.58	2.93	1.88	2.35	1.94	2.66	2.00
21 days	2.51	1.83	3.99	3.34	1.54	6.14	2.86	4.64	4.27
28 days	1.21	0.12	2.54	3.34	4.65	0.56	4.66	4.65	0.07

Table 14: Results of the concrete compressive strength test using Phillippi sand as filler

5.4.3.2 TENSILE SPLITTING STRENGTH

Table 15 and Table 16 show the results of the tensile splitting strength of the concrete for the two different fillers used, Malmesbury sand and Phillippi sand respectively. In each table, the following information can be found: the tensile splitting strength of each concrete cube obtained by conducting the tensile splitting strength test in accordance with *SANS 6853:2006 - Concrete test - Tensile splitting strength of concrete*, the tensile strength of the concrete which is taken as the average of three concretes, and the coefficient of variation.

In order to highlight the influence that the filler has on the tensile splitting strength of the concrete, the results of the tensile splitting strength test of the concrete made with different f/c ratios are compared to each other. That is to say, the results of the different f/c ratios are compared to each other for the same concrete target density. This comparison shows that the concrete mixes made with f/c of 1.0 yield higher tensile splitting strength than the concrete mixes made with f/c of 1.5 and 2.0. An f/c of 2.0 yields the lowest tensile splitting strength for the respective concrete design target densities. Therefore, the concrete mixes made with f/c 1.0 yield the highest tensile splitting strength followed by f/c ratios of 1.5 and 2.0. This is true for all three chosen design target densities used in this investigation and is independent of the choice of filler used.

Comparing the results in Table 15 and Table 16 for Malmesbury sand and Phillippi sand shows that the results of the tensile splitting strength of the concrete mixes, designed for the same design target density and same f/c ratio using Phillippi sand as filler, yield higher tensile splitting strength values. This result is consistent throughout the data.

Malmesbury Sand									
Density	1600 kg/m^3			1800 kg/m^3			2000 kg/m^3		
f/c	1.00	1.50	2.00	1.00	1.50	2.00	1.00	1.50	2.00
	2.51	2.30	1.99	3.78	2.85	2.79	4.91	3.73	3.60
	2.53	2.36	1.85	4.07	2.97	2.91	4.62	4.33	3.57
	2.21	2.30	1.82	4.29	3.00	2.90	4.45	4.13	3.33
Average	2.42	2.32	1.89	4.04	2.94	2.87	4.66	4.06	3.50
CoV(%)	7.36	1.48	4.98	6.26	2.73	2.32	4.97	7.56	4.21

Table 15: Results of the concrete tensile splitting strength test using Malmesbury sand as a filler

Phillippi Sand									
Density	1600 kg/m^3			1800 kg/m^3			2000 kg/m^3		
f/c	1.00	1.50	2.00	1.00	1.50	2.00	1.00	1.50	2.00
	3.10	2.66	1.95	4.29	3.20	3.10	6.32	4.39	3.65
	2.93	2.68	2.25	3.92	3.38	3.13	6.13	4.45	3.83
	3.14	2.52	2.36	4.09	3.67	3.17	6.01	4.55	3.83
Average	3.06	2.62	2.19	4.10	3.42	3.13	6.16	4.46	3.77
CoV(%)	3.68	3.42	9.65	4.51	6.91	1.20	2.56	1.82	2.66

Table 16: Results of the concrete tensile splitting strength test using Phillippi sand as a filler

5.4.3.3 MODULUS OF ELASTICITY

Table 17 and Table 18 show the results of the modulus of elasticity of the concrete mixes made for concrete design target densities of 1600 kg/m^3 and 1800 kg/m^3 with f/c ratios of 1.0, 1.5 and 2.0 for the Malmesbury sand and Phillippi sand respectively. The results of the modulus of elasticity for the Malmesbury sand, in Table 17, indicate that the f/c ratio has no clear influence on the modulus of elasticity, because the results fluctuate. For instance, for the 1600 kg/m^3 mix with f/c of 1.0 the E-modulus of the concrete is found to be 11.67 GPa. the E-modulus then increases to 14.50 GPa for the f/c ratio of 1.5, but decreases to 12.17 GPa for the f/c of 2.0. In this case the highest E-modulus result was recorded for the concrete mix made with f/c 1.5 followed by f/c 2.0 and then f/c 1.0. The opposite happened when the concrete density was increased to 1800 kg/m^3. The concrete made with f/c ratio of 1.0 was recorded to have the highest E-modulus, 21.33 GPa, of these three concrete mixes. The concrete mix with f/c ratio of 1.5 had the lowest E-modulus, 17.67 GPa and the concrete mix with f/c of 2.0 recorded the second highest E-modulus, 19.17 GPa, of the three concrete mixes.

The result of the modulus of elasticity of the concrete mixes made with Phillippi sand, Table 18, follows a certain trend. Irrespective of the concrete design density, the modulus of elasticity seems to increase with a decrease in f/c ratio. The highest E-modulus results are therefore recorded for the concrete mixes made with f/c 1.0 followed by f/c 1.5 and finally f/c 2.0.

Comparing the results of the modulus of elasticity of the concrete mixes with the same design density and same f/c ratios shows no clear trend of the influence that the filler type has on the modulus of elasticity of the concrete because some of the concrete mixes made using Malmesbury sand as filler, Table 17 and Table 18, yield E-modulus values greater than those recorded for the Phillippi sand for the same concrete mix design parameters, and vice versa. Note that the differences in the results never exceed a margin of 5 GPa.

Malmesbury Sand						
Density	1600 kg/m^3			1800 kg/m^3		
f/c	1.00	1.50	2.00	1.00	1.50	2.00
	12.00	14.00	12.00	20.00	18.00	18.50
	11.50	14.50	12.00	23.00	18.00	19.00
	11.50	15.00	12.50	21.00	17.00	20.00
Average	11.67	14.50	12.17	21.33	17.67	19.17
CoV(%)	0.02	3.45	2.37	0.07	3.27	3.98

Table 17: Results of the modulus of elasticity for the different concrete mixes using Malmesbury as filler

Phillippi sand						
Density	1600 kg/m^3			1800 kg/m^3		
f/c	1.00	1.50	2.00	1.00	1.50	2.00
	14.50	13.00	12.00	20.50	19.50	17.00
	15.00	14.00	12.00	19.50	19.00	18.50
	15.00	14.00	11.50	20.50	20.00	17.50
Average	14.83	13.67	11.83	20.17	19.50	17.67
CoV(%)	1.95	4.22	2.44	2.86	2.56	4.32

Table 18: Results of the modulus of elasticity for the different concrete mixes using Phillippi sand as filler

5.4.4 THE EFFECT OF THE CONCRETE AGE ON THE PROPERTIES OF THE CONCRETE

5.4.4.1 COMPRESSIVE STRENGTH

Figure 29 and Figure 30 show the compressive strength of the concrete versus concrete age. The compressive strength of the concrete was taken as the average of three cubes. A compressive strength of 0 MPa has been assumed for the concrete at the age of zero days. This assumption does in no way have any influence on the interpretation of the results of the compressive strength because no remarks are made about the strength development of the concrete between the concrete ages of zero and 7 days. In each figure (Figure 29 and Figure 30) the results of the compressive strength test at the concrete ages of 7, 14, 21, and 28 days can be found. These results give an indication of how the concrete strength develops over time under continuous submerged water curing conditions. In Figure 29 the results indicate the compressive strength of the concrete for the 9 different concrete mixes made using Malmesbury sand as filler. The legend shows the concrete density and the f/c ratio used in the concrete mixes. In Figure 30 the results indicate the compressive strength of the concrete for the 9 different concrete mixes using Phillippi sand as filler. The letters M and P denote the different filler (i.e. M - Malmesbury sand, P - Phillippi sand) used as a mix constituent in the concrete mix respectively.

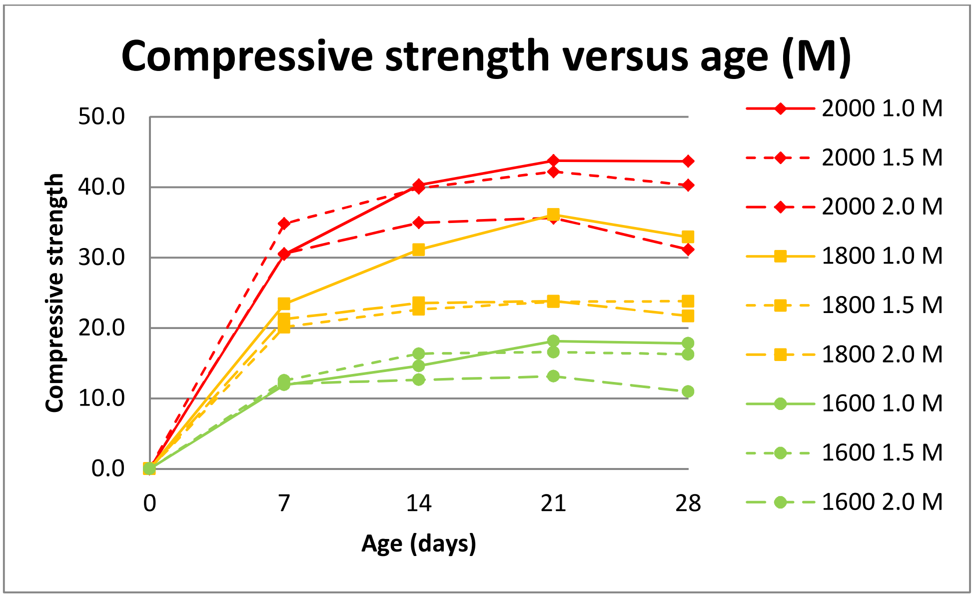

Figure 29: Compressive strength versus concrete age (Malmesbury sand)

In Figure 29 it can be seen that there exists a non-linear relationship between concrete compressive strength and concrete age and that with an increase in concrete density there is an increase in the concrete compressive strength. The results indicate that the rate of strength gain in concrete compressive strength is low after the concrete age of 7 days. However, some of the results indicate that not only does the strength gaining process stop but the concrete seems to be losing strength after a concrete age of 21 days when the concrete is subjected to continuous water curing. This can be seen in the results of the compressive strength of the following concrete mixes: 2000 1.5 M, 2000 2.0 M, 1800 1.0 M, 1800 2.0 M, 1600 1.5 M and 1600 2.0 M. The result of the concrete compressive strength of these concrete mixes at the age of 28 days is less than the compressive strength of the concrete at one of the following ages: 7, 14 and 21 days. For the concrete mixes designed for 2000 kg/m^3 the strength loss increases with an increase in f/c ratio; this is also true for the 1600 kg/m^3 concrete mixes. However this is not true for the concrete mixes designed for 1800 kg/m^3 as the 1800 kg/m^3 with f/c of 1.0 shows a greater loss in compressive strength than the 1800 kg/m^3 with f/c of 2.0.

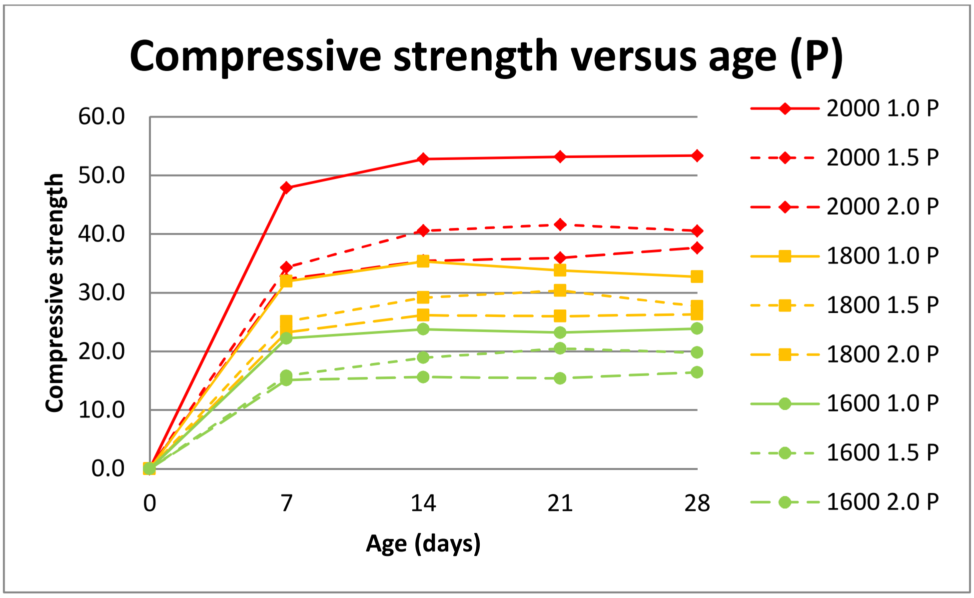

Figure 30: Compressive strength versus concrete age (Phillippi sand)

In Figure 30 it can be seen that there exists a non-linear relationship between concrete compressive strength and concrete age and that with an increase in concrete density there is an increase in the concrete compressive strength. In almost all cases the results of the compressive strength test show that the rate of concrete compressive strength gain decreases quite significantly after 7 days, resulting in a small increase in the compressive strength of the concrete between 7 and 28 days. Also, for the following concrete mixes the results of the 28 day compressive strength of the concrete is smaller than the results of the compressive strength at an earlier age: 2000 1.5 P, 1800 1.0 P, 1800 1.5 P and 1600 1.5 P. For this type of filler the f/c ratio does not seem to exaggerate the loss of compressive strength of the concrete. For the 1800 1.0 concrete mix the compressive strength of the concrete starts after 14 days and for the rest of the concrete mixes for which compressive strength loss is reported after 21 days.

Comparing the results shown in Figure 29 to the results shown in Figure 30, the Malmesbury sand mixes show a much higher strength development after 7 days compared to that of the Phillippi sand mixes. Also, more concrete mixes made using Malmesbury sand as filler have yielded 28 day compressive strengths that are less than the compressive strengths at earlier ages. Note that the sample size of three specimens per test must be increased in future work to confirm the trend of reduced strength with curing age. Continuous water curing for aerated concrete appears to be detrimental to the concrete after the concrete age of 14 or 21 days depending on the filler used. The long term strength should also be determined, but under more realistic curing regimes which better simulate environmental exposure of structural concrete.

In order to show the effect the filler has on the compressive strength development with concrete age, the results of the compressive strength test shown in Figure 26 - Figure 28 have been normalized with respect to the concrete compressive strength at 28 days. The results of the normalized compressive strengths of the concrete mixes are presented in Figure 31 to Figure 33 for concrete densities of 2000 kg/m^3, 1800 kg/m^3 and 1600 kg/m^3. The figures will therefore show the relationship between the compressive strength of the concrete at the concrete age of 7, 14 and 21 days with that of the compressive strength at 28 days, which is usually considered for design.

Figure 31 depicts the normalized compressive strength of the concrete mixes designed for a target density of 2000 kg/m^3. The results of six different concrete mixes are presented in Figure 31, three concrete mixes made using Malmesbury sand as filler with f/c ratios of 1.0, 1.5 and 2.0 and three concrete mixes made using Phillippi sand as filler with f/c ratios of 1.0, 1.5 and 2.0. From the figure it can be seen that the 7 day compressive strength as a factor of the 28 day compressive strength, for the concrete mixes made using Phillippi sand as filler, is approximate 0.85 to 0.90. Also, as a factor of the 28 day compressive strength of the concrete, the 14 day and 21 day compressive strength is approximately 0.94 to 1.00 and 0.95 to 1.03. For the concrete mixes made using Malmesbury sand as filler a trend showing that the rate of compressive strength development increases with decreasing f/c ratio. The concrete mix with f/c ratio of 2.0 gives the highest factor of 0.98 of the three concrete mixes followed by 0.86 and 0.70 for f/c 1.5 and 1.0 at the concrete age of 7 days. At the concrete age of 14 and 21 days the factors are in the range of 1.12 to 0.92 and 1.14 to 1.00, respectively.

Figure 31: Concrete compressive strength over time for concrete target density of 2000 kg/m^3. Illustrating the influence of the filler on the compressive strength of the concrete (Normalized)

Figure 32 depicts the normalized compressive strength of the concrete mixes designed for a target density of 1800 kg/m^3. The results of six different concrete mixes are presented in Figure 32, three concrete mixes made using Malmesbury sand as filler with f/c ratios of 1.0, 1.5 and 2.0 and three concrete mixes made using Phillippi sand as filler with f/c ratios of 1.0, 1.5 and 2.0. From the figure it can be seen that the 7 day compressive strength as a factor of the 28 day compressive strength, for the concrete mixes made using Phillippi sand as filler, is approximate 0.88 to 0.98. Also, as a factor of the 28 day compressive strength of the concrete, the 14 day and 21 day compressive strength is approximately 0.99 to 1.05 and 0.99 to 1.10. For the concrete mixes made using Malmesbury sand as filler, the concrete mix with f/c ratio of 2.0 gives the highest factor of 0.98 of the three concrete mixes followed by 0.86 and 0.70 for f/c 1.5 and 1.0 at the concrete age of 7 days. At the concrete age of 14 and 21 days the factors are in the range of 1.12 to 0.92 and 1.14 to 1.00, respectively.

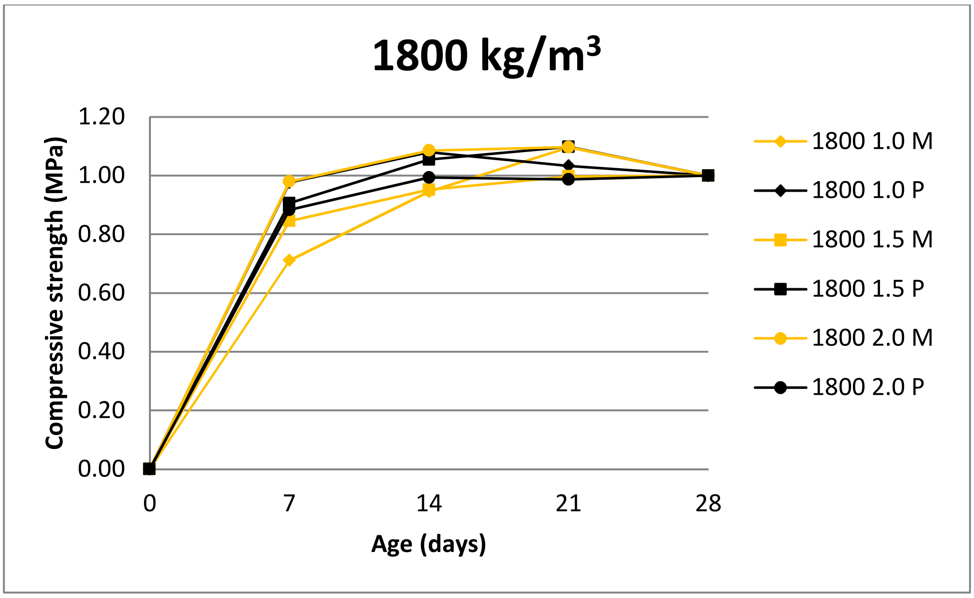

Figure 32: Concrete compressive strength over time for concrete target density of 1800 kg/m^3. Illustrating the influence of the filler on the compressive strength of the concrete (Normalized)

Figure 33 depicts the normalized compressive strength of the concrete mixes designed for a target density of 1600 kg/m^3. The results of six different concrete mixes are presented in Figure 33, three concrete mixes made using Malmesbury sand as filler with f/c ratios of 1.0, 1.5 and 2.0 and three concrete mixes made using Phillippi sand as filler with f/c ratios of 1.0, 1.5 and 2.0. From the figure it can be seen that the 7 day compressive strength as a factor of the 28 day compressive strength, for the concrete mixes made using Phillippi sand as filler, is approximate 0.85 to 0.90. Also, as a factor of the 28 day compressive strength of the concrete, the 14 day and 21 day compressive strength is approximately 0.94 to 1.00 and

0.95 to 1.03. For the concrete mixes made using Malmesbury sand as filler a trend showing that the rate of compressive strength development increases with decreasing f/c ratio. The concrete mix with f/c ratio of 2.0 gives the highest factor of 1.10 of the three concrete mixes followed by 0.77 and 0.67 for f/c 1.5 and 1.0 at the concrete age of 7 days. At the concrete age of 14 and 21 days the factors are in the range of 1.15 to 0.82 and 1.20 to 1.02, respectively.

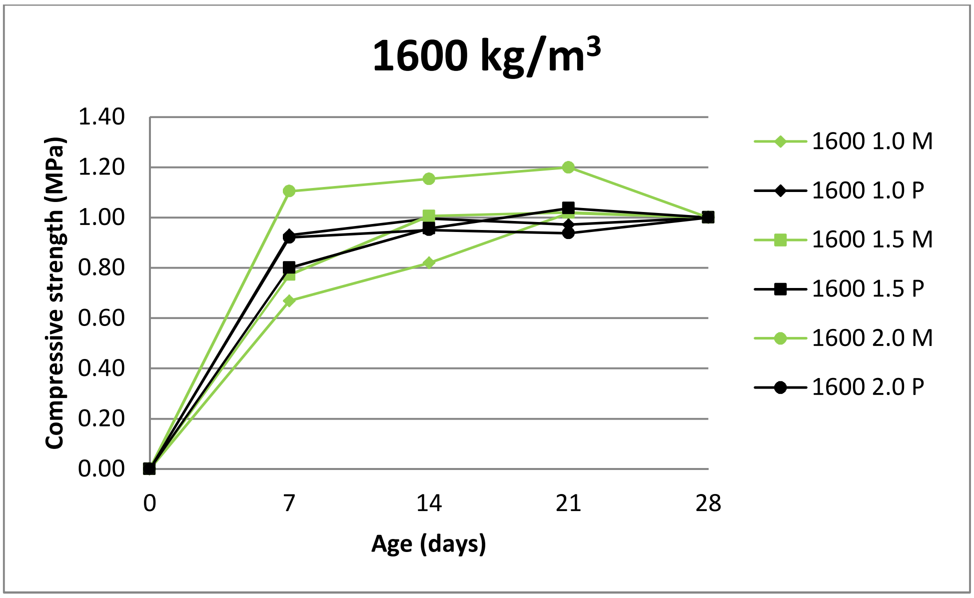

Figure 33: Concrete compressive strength over time for concrete target density of 1600 kg/m^3. Illustrating the influence of the filler on the compressive strength of the concrete (Normalized)

The results depicted in Figure 31 - Figure 33 of the normalised compressive strength for the concrete densities of 2000, 1800, 1600 kg/m^3, versus concrete age are tabulated in Table 19 and Table 20 for the Malmesbury sand and Phillippi sand. Comparing the results of the concrete mixes designed for the same design target density but using different f/c ratio indicates that the rate of compressive strength gain increase with increasing f/c ratio for the concrete mixes designed for a design target density of 2000 kg/m^3 and 1600 kg/m^3 when Malmesbury sand is used, in Table 19. The concrete mixes designed for 1800 kg/m^3 showed the same trend however at the concrete age of 14 days, the concrete mixes made with f/c 1.0 yields a factor that equals that of the concrete mix made with f/c of 2.0.

From Figure 29 and Figure 30 it appears to be predominantly the f/c greater than 1.0 mixes that have higher compressive strength at ages smaller than 28 days. Concrete mixes with design target density of 1800 are the exception, for which also f/c of 1.0 mixes have lower strength at 28 days than earlier ages. Also, from Figure 29 and Figure 30, and Figure 31 - Figure 33, it appears that the overshoot in strength at early age (and subsequent reduced strength at 28 days) is worst for the highest f/c ratio of 2.0. In fact, there is a suggestion of consistency in higher early age strength for higher f/c ratio. It is predominantly Malmesbury sand mixes that show this phenomenon. Phillippi sand mixes have fast strength development, and then retain their strength (flat line from about 14 days onward).

Malmesbury sand									
Density	1600 kg/m^3			1800 kg/m^3			2000 kg/m^3		
f/c	1.0	1.5	2.0	1.0	1.5	2.0	1.0	1.5	2.0
Age									
7	0.67	0.77	1.10	0.71	0.85	0.98	0.70	0.86	0.98
14	0.82	1.01	1.15	0.95	0.95	1.09	0.92	0.99	1.12
21	1.02	1.02	1.20	1.10	1.00	1.10	1.00	1.05	1.14
28	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Table 19: Normalised compressive strength of the concrete mixes using Malmesbury sand as filler

Phillippi sand									
Density	1600 kg/m^3			1800 kg/m^3			2000 kg/m^3		
f/c	1.0	1.5	2.0	1.0	1.5	2.0	1.0	1.5	2.0
Age									
7	0.93	0.80	0.92	0.98	0.91	0.88	0.90	0.85	0.86
14	1.00	0.96	0.95	1.05	1.05	0.99	0.94	1.00	0.94
21	0.97	1.04	0.94	1.10	1.10	0.99	1.00	1.03	0.95
28	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Table 20: Normalised compressive strength of the concrete mixes using Phillippi sand as filler

CHAPTER 6 - CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

The main aim of the thesis is to produce aerated concrete with sufficient compressive strength in order to be classified as structural concrete. Aerated concrete is a form of lightweight concrete which is made with cement, sand, water and 20 per cent by volume or more air entrained into the concrete mix. This type of lightweight concrete is less popular than the more common lightweight aggregate, which as the name suggests is made like normal concrete however the aggregate used in the concrete mix has a relative density that is less than that used for normal weight concrete. The task of producing structural lightweight concrete was split into several subtasks, the first of which was to find a suitable concrete mix design approach that can yield aerated concrete mixes that are as close as possible to the design target density. The second task was designed to investigate the influence the filler has on the properties of the concrete and the third task to investigate the factors affecting the strength development of aerated concrete. The conclusions drawn from the experiments performed under these tasks are presented in this section.

The mix design of aerated concrete differs vastly from that of normal concrete in the sense that the water/cement ratio is not the controlling factor but the concrete design target density is the controlling factor. A concrete mix design approach therefore needed to be developed for aerated concrete made with an air-entraining agent (AEA). The factors that influence the mix design approach are cement, filler, and AEA used. The mix design of aerated concrete consists of two parts: (1) finding a suitable concrete base mix for air entrainment, and (2) finding the required amount of AEA needed in the concrete mix to produce the concrete design target density. For the concrete base mix, the water demand of the concrete mix must be found. For air-entrainment in concrete the water content of the mix must give the mix a certain consistency otherwise air bubbles will either make their way to the surface of the concrete and escape or the air bubbles will break during the mixing process. With the use of the ASTM flow turn table the water demand of the concrete base mixes was determined. From this the water demand of the mix constituents was calculated. Also, due to the complex nature of air entrainment in concrete a trial and error method to find the amount of AEA to produce aerated concrete with a certain design target density had to be adopted. The following conclusions can be drawn from the mix design approach adopted:

- The optimum water demand of the concrete base mix corresponds to a spreadability value in the range of 180 to 196 mm

- Air entrainment is only possible when the water demand of the concrete mix is satisfied
- The amount of AEA required to produce aerated concrete with a specific design target density can only be found by a trial and error method because of the complex nature of air-entrainment

The second task undertaken was to investigate the influence the filler has on the properties of the concrete. Also, the difference between continuous water curing and wrapped curing was investigated. Based on the results obtained from the experiments the following conclusions can be made:

- The properties of the filler have a significant effect on the design of aerated concrete
- The finer the sand the greater the water demand of the concrete mix
- A filler with a fine particle size distribution yields a higher strength than a filler with a coarse particle size distribution
- The use of fine filler resulted in an increase in the amount of air entrained in the concrete mix. This is indicated by the use of less amount of AEA to percentage of binder used.
- Curing the concrete in water resulted in a higher compressive strength except in cases where free moisture played a role.
- The strength difference between the concrete cured in water compared to that cured by wrapping with plastic is small. Therefore, the effect the two curing methods used in this investigation have on the strength of the concrete is negligible.

The third task focused on the strength development of aerated concrete. The experiment designed under this task can basically be thought of as a parameter study of the mix design parameters on the concrete strength. These parameters include the following: filler/cement ratio (f/c), sand/cement ratio (s/c), ash/cement ratio (a/c), and concrete age. To simplify the experiments it was decided not to use fly-ash during the investigation. Therefore, the s/c ratio is equal to the f/c ratio in all concrete mixes. Based on the results of the experiments the following conclusions can be drawn:

- With an increase in the design target density there is a non-linear increase in compressive strength shown to be best regressed with a power law, and a linear increase in concrete tensile strength
- With a decrease in f/c there is an increase in concrete compressive strength and tensile splitting strength
- With an increase in concrete design target density there is an increase in the modulus of elasticity of concrete.

- No clear trend was found with f/c ratio for the modulus of elasticity
- The values found for the modulus of elasticity for the different concrete mixes were lower than the nominal values given for lightweight aggregate concrete in EC2
- In nearly all cases the 28 day compressive strength of the concrete yielded values that are lower than the compressive strength of the concrete at younger ages

6.2 RECOMMENDATIONS

Structural aerated lightweight concrete can be produced, with strength applicable for structural application. The material stiffness, as reflected by the E-moduli, is lower than corresponding strength normal weight concrete, and has to be considered in structural design. This study did not include dimensional stability, in terms of creep and shrinkage, but this will be the next step in this research program. Based on the work presented in this thesis the following recommendations are presented for future studies

- Tests to determine long term dimensional stability, shrinkage and creep, to confirm that the material is useful, or at least quantify the shrinkage and creep for the concrete to be taken into consideration in structural design
- Tests to determine the reason for the high early age strength but low 28 day strength
- Further studies of optimisation of structural response which includes, utilisation of high volumes of fly-ash for potentially increasing the strength to density ratio and investigate the potential use of fibres in aerated concrete to overcome problems associated with steel reinforcement in aerated concrete

CHAPTER 7 - REFERENCES

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APPENDICES

APPENDIX A1 - MALMESBURY SAND

Water demand of the filler

Item	Mass[kg]	RD	Vol (l)
Sand	3.00	2.65	1.132
Water	0.45	1.00	0.45
Total	3.45	-	1.582

Water/solids ratio (w/s*):

(w/s*) =	0.15	Malmesbury sand

Binder used	Surebuild 42.5 N			OPC 52.5 N	X
Filler/cement ratio:	1.0	X	1.5	2.0	
Filler used:	Malmesbury sand	X	Phillippi sand		

Concrete base mix composition

Item	Mass[kg]	RD	Vol (l)
Cement	2.00	3.14	0.637
Sand	2.00	2.65	0.755
Total	4.00	-	1.392

(w/s*)	Water (l)	Workability			ASTM flow turn table		
		Yes	No	Describe	φ_1 (mm)	φ_2 (mm)	φ_{ave} (mm)
0.1625	0.650	X		Medium	197	195	196

Water/cement ratio (w/c):

(w/c) =	0.175	OPC 52.5 N

Binder used	Surebuild 42.5 N		OPC 52.5 N		X
Filler/cement ratio:	1.0	1.5	X	2.0	
Filler used:	Malmesbury sand	X	Phillippi sand		

Concrete base mix composition

Item	Mass[kg]	RD	Vol (l)
Cement	2.00	3.14	0.637
Sand	3.00	2.65	1.132
Total	5.00	-	1.392

(w/s*)	Water (l)	Workability			ASTM flow turn table		
		Yes	No	Describe	φ_1 (mm)	φ_2 (mm)	φ_{ave} (mm)
0.142	0.710	X		Medium	190	191	191

Water/cement ratio (w/c):

(w/c) =	0.130	OPC 52.5 N

Binder used	Surebuild 42.5 N		OPC 52.5 N		X
Filler/cement ratio:	1.0	1.5		2.0	X
Filler used:	Malmesbury sand	X	Phillippi sand		

Concrete base mix composition

Item	Mass[kg]	RD	Vol (l)
Cement	2.00	3.14	0.637
Sand	4.00	2.65	1.509
Total	6.00	-	2.146

(w/s*)	Water (l)	Workability			ASTM flow turn table		
		Yes	No	Describe	φ_1 (mm)	φ_2 (mm)	φ_{ave} (mm)
0.13	0.780	X		Medium	185	190	187

Water/cement ratio (w/c):

(w/c) =	0.090	OPC 52.5 N

APPENDIX A2 - PHILLIPPI SAND

Water demand of the filler

Item	Mass[kg]	RD	Vol (l)
Sand	3.00	2.65	1.132
Water	0.65	1.00	0.65
Total	3.65	-	1.782

Water/solids ratio (w/s*):

(w/s*) =	0.217	Phillippi sand

Binder used	Surebuild 42.5 N				OPC 52.5 N	X
Filler/cement ratio:	1.0	X	1.5		2.0	
Filler used:	Malmesbury sand				Phillippi sand	X

Concrete base mix composition

Item	Mass[kg]	RD	Vol (l)
Cement	2.00	3.14	0.637
Sand	2.00	2.65	0.755
Total	4.00	-	1.392

(w/s*)	Water (l)	Workability			ASTM flow turn table		
		Yes	No	Describe	φ_1 (mm)	φ_2 (mm)	φ_{ave} (mm)
0.17	0.680	X		Medium	190	190	190

Water/cement ratio (w/c):

(w/c) =	0.123	OPC 52.5 N

Binder used	Surebuild 42.5 N			OPC 52.5 N	X
Filler/cement ratio:	1.0	1.5	X	2.0	
Filler used:	Malmesbury sand			Phillippi sand	X

Concrete base mix composition

Item	Mass[kg]	RD	Vol (l)
Cement	2.00	3.14	0.637
Sand	3.00	2.65	1.132
Total	5.00	-	1.392

(w/s*)	Water (l)	Workability			ASTM flow turn table		
		Yes	No	Describe	φ_1 (mm)	φ_2 (mm)	φ_{ave} (mm)
0.16	0.800	X		Medium	192	185	188

Water/cement ratio (w/c):

(w/c) =	0.075	OPC 52.5 N

Binder used	Surebuild 42.5 N			OPC 52.5 N	X
Filler/cement ratio:	1.0	1.5		2.0	X
Filler used:	Malmesbury sand			Phillippi sand	X

Concrete base mix composition

Item	Mass[kg]	RD	Vol (l)
Cement	2.00	3.14	0.637
Sand	4.00	2.65	1.509
Total	6.00	-	2.146

(w/s*)	Water (l)	Workability			ASTM flow turn table		
		Yes	No	Describe	φ_1 (mm)	φ_2 (mm)	φ_{ave} (mm)
0.16	0.960	X		Medium	184	188	186

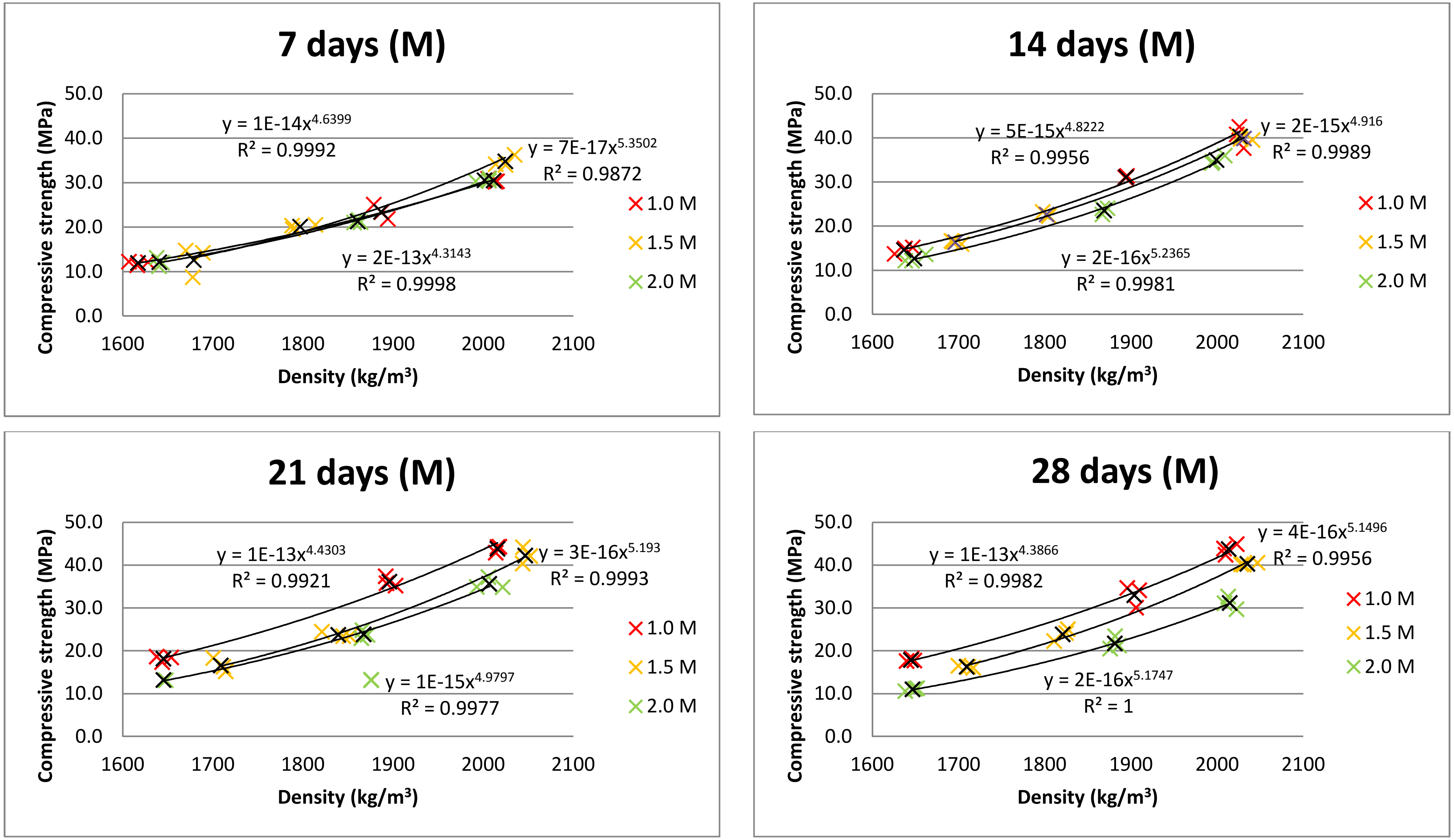
Water/cement ratio (w/c):

(w/c) =	0.047	OPC 52.5 N

APPENDIX A3 - ASTM FLOW TURN TABLE

Figure 34: ASTM flow turn table illustrating the spreadability of two concrete base mixes. The concrete base mix on the left has a good consistency. The concrete base mix on the right is too flowable

APPENDIX B1: COMPRESSIVE STRENGTH VERSUS DENSITY - MALMESBURY SAND



APPENDIX B2: COMPRESSIVE STRENGTH VERSUS DENSITY - PHILLIPPI SAND