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THE DESIGN AND OPTIMISATION OF COLD ASPHALT EMULSION MIXTURES

By

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ABSTRACT

Road structures are important to the survival of nations. As the cost for the rehabilitation and maintenance of highways soars, civil engineers and administrators face the ever present difficulty of meeting current resurfacing and rehabilitation needs. The deterioration of road structures under growing traffic weight and volume is occurring faster than agencies envisaged coupled with increasingly scarce and expensive new materials required. It is now apparent that for planning, design and construction for road structures, the most efficient and cost effective processes, materials and practices available must be appropriately considered.

The use of recycled materials as a sustainable alternative is gaining significant worldwide attention. The overall purpose of this research was to conduct an in-depth investigation and analysis into the design and optimisation of Cold Asphalt Emulsion Mixtures (CAEMs) incorporating high contents of Reclaimed Asphalt Pavements (RAP). To achieve the objectives of the research, four proportions of RAP aggregate materials in addition to Virgin Aggregates (VA) were used as categorised below:

- Category 1: 0% RAP (no RAP, 100% VA)
- Category 2: 50% RAP (50% RAP, 50% VA)
- Category 3: 85% RAP (85% RAP, 15% VA)
- Category 4: 95% RAP (95% RAP, 5% VA)

The effect of mixing and compaction temperatures at 5°C, 20°C and 32°C and how cement at 0%, 1% and 3% OPC influenced the CAEMs was also investigated.

This study presents a practical mix design procedure to act as a guideline for CAEMs incorporating high RAP contents by identifying critical parameters for the various categories of CAEMs which stemmed from the fact that currently there is no universally accepted mix design. The proposed mix design guideline is presented in this thesis. The effect of accelerated curing was investigated to study the effects of temperature, curing duration, conditioning and the influence of cement on the CAEMs. The research showed that an increase in curing temperature results in an increase in the stiffness and strength of the CAEMs. The thesis presents results on the mechanical and performance properties which provided vital information on expected performance of CAEMs incorporating high contents of RAP for use as a road base material. The research was able to highlight the purported effects of residual binder in RAP which could contribute positively to the mechanical and performance properties of the CAEMs. This points to the fact that treating RAP as "black rock" is not the right approach. The RAP needs to be evaluated for its inherent properties and suitability for purpose. The stiffness and strength were investigated using the Indirect Tensile Stiffness Modulus (ITSM) and Indirect Tensile Strength (ITS) tests which proved useful in ranking them. The addition of 1% OPC improved the stiffness of Categories 1-3 mixtures by 32% with Category 4 having the highest increase at 89%. The inclusion of 3% OPC, more than doubled the stiffness values. The Indirect Tensile Fatigue Test (ITFT) was used to investigate the fatigue characteristics. Results showed that if the CAEMs with cement at 1% and 3% experienced strains in the region of 200$\mu\varepsilon$, they tend to fail suddenly soon after crack initiation due to reduced flexibility of the CAEMs. This was more pronounced for the CAEMs at 3% OPC.

Resistance to permanent deformation was investigated using the Vacuum Repeated Load Axial Test (VRLAT) which showed that the mixing and compaction temperature influenced the permanent deformation characteristics of the CAEMs. Increasing OPC content to 1% for Categories 2 and 3 resulted in a decrease in permanent strains of 47% and at 3% OPC, the decrease in permanent strains was 54%. Wheel Tracking Test (WTT) was conducted to ascertain the susceptibility of the CAEMs to deform under loading, investigate crack propagation and number of cycles to failure. The test showed that the performance of the specimens was affected by the test temperature. Increased test temperatures resulted in an increased rate of rutting and eventual failure of the specimens. The test further highlighted the positive benefits of adding cement to the mixtures which resulted in reduced strains and an increased number of cycles to failure for the CAEMs. Structural design and modelling was conducted using KENLAYER which was able to account for the non-linearity of the CAEMs. This was crucial in having a total overview of these mixture types. Although, the structural design was based on practical hypothetical layer thicknesses, the results provided useful insight into the structural capabilities of the CAEMs. The RAP CAEMs generally had lower horizontal tensile strain values in comparison to the VA CAEMs. The design charts showed that an increase in the thickness of the base course and surfacing layer resulted in an increase in the overall fatigue life of the pavement structure. Overall, evaluating the complete findings of this research, CAEMs produced with high RAP contents especially at 50% and 85% RAP had considerably enhanced mechanical and performance properties and are suitable for inclusion as a base material for reconstruction and rehabilitation.

ACKNOWLEDGEMENTS

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I would like to express my deepest gratitude to my supervisors Dr. N.H. Thom and Dr. T. Parry for the great help, advice, guidance, insight, assistance and close supervision that motivated and kept me grounded. My sincere thanks go to Dr G.D Airey for his positive insight during the PhD reviews and also to Dr. J. Grenfell for his assistance in setting up the Wheel Tracker. Significant part of this research was based on laboratory experiments and I would like to extend my thanks and appreciation to all the technicians including Richard Blakemore, Martyn Barrett, Anthony Beska, Nancy Hodge, Jon Watson and Lawrence Pont. I would like to thank my friends and fellow researchers at Nottingham Transportation Engineering Centre (NTEC) with whom I shared many enjoyable moments and experiences.

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DECLARATION

The research described in this thesis was conducted at the Nottingham Transportation Engineering Centre, University of Nottingham between February 2012 and June 2015. I declare that the work is my own and has not been submitted for a degree at another university.

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1. INTRODUCTION

1.1 Overview

The recycling of asphalt pavements has been an important option in road maintenance and construction since the oil crisis in the mid-1970s which resulted in an increase in the cost of construction especially the price of bitumen. In recent years, the benefits of using RAP has seen a considerable increase due to a greater awareness of the environmental problems and the need to conserve materials for future generations. The use of CAEMs provides a sustainable alternative to conventional Hot Mix Asphalts (HMAs). The process involves milling an existing pavement and reusing the RAP with the inclusion of bitumen emulsion to rehabilitate and construct new asphalt pavements all without the use of heat. The RAP usually has high quality, well graded aggregate materials coated with bitumen that can be effectively reused in new asphalt mixtures.

The use of CAEMs has certain advantages over hot bituminous road mixtures with respect to financial, environmental, energy and logistical savings. CAEMs over time have evolved in their application and use in various locations around the world including the United States, Sweden, Lithuania, United Kingdom, South Africa and Australia with various agencies in charge of road networks proposing and implementing the inclusion of RAP in mixtures for asphalt pavements albeit at various proportions for road applications. This is not to say that CAEMs are without their short-comings which include low early life strength, long curing times, high air void contents.

1.2 Statement of Problem

The use of CAEMs has grown widely helping to reduce and conserve virgin materials for future generations. The increased use of CAEMs prompts research into the material, mechanical and performance properties of these mixtures especially at high contents of RAP.

There are many factors that influence the performance of CAEMs including aggregate properties, particle size distribution, bitumen emulsion, binder content, material compatibility, moisture content, mixing procedure, compaction characteristics, curing regime and low early life strength as CAEMs gain strength with time. These factors present a unique challenge in developing a mix design method for CAEMs resulting in the classification of these materials as 'inferior' or 'second class' in some quarters.

This study aims to bridge the gap by addressing most of the issues raised and goes a step further by investigating CAEMs incorporating high contents of RAP up to 95%. This research simulates as realistically as possible what should be expected from CAEMs with the view that results from the research will provide information for better understanding of the mechanical and performance properties of CAEMs. A practical and consistent mix design procedure to act as a guideline for CAEMs as a base course material is developed. In the end, the study attempts to show that the findings on the behaviour of CAEMs with high RAP content can be successfully applied to the structural design of pavements.

1.3 Objectives and Scope of the Research

The main objective of this research is to design and optimise CAEMs incorporating high contents of RAP. The study aims to better understand how mixing and compaction temperatures influence these mixture types. The influence of cement at 1% and 3% OPC in comparison to CAEMs without cement are investigated. The study aims to propose a practical and consistent mix design procedure to act as a guideline for mixtures incorporating high RAP contents.

In achieving the objectives, detailed tests are conducted to obtain the mechanical and performance properties of the CAEMs investigated. It is envisaged that this research would proffer effective guidelines, analysis, solutions and increased confidence for the design of CAEMs incorporating high RAP contents. To achieve the stated objectives of the research, these major factors were considered as stated below:

1. Detailed evaluation and literature review of mix design and structural design procedures used by various organisations and agencies.
2. Developing a practical and effective mix design procedure to act as a guideline for CAEMs with high RAP content.
3. Material selection and evaluation including trial tests for suitability and applicability.
4. Investigate the influence of accelerated curing on the CAEMs.
5. Investigate the mechanical and performance characteristics of CAEMs.
6. Conduct structural design and analysis using the mechanistic (non-linear elastic) approach for the CAEMs.

1.4 Thesis Outline

Chapter	Description
Chapter 1	This chapter presents an overview of the research. The background information, the research objectives, scope and structure of the thesis are presented in this chapter.
Chapter 2	This chapter is a literature review of previous research studies on cold mix asphalts focusing specifically on CAEMs. The chapter presents a brief history of bitumen emulsions, composition of bitumen emulsions, mix and structural design considerations for CAEMs. Material properties, compaction and curing techniques are reviewed.
Chapter 3	Presents a study of the mix design procedure for CAEMs. The chapter presents the process involved in producing the CAEMs which included: material characterisation, evaluation of mix design procedures, compaction and curing characteristics. Initial mechanical and performance tests were conducted to understand the behaviour and quickly rank the various categories of CAEMs.
Chapter 4	This chapter investigated the curing mechanism of CAEMs taking into account the effect of temperature, curing conditioning and curing duration. The influence of cement at the different curing regimes was evaluated and presented in this chapter.
Chapter 5	This chapter detailed results of the mechanical and performance properties of the CAEMs produced using high contents of RAP with and without the use of cement (0%, 1% and 3% OPC). The study also looked at the compaction characteristics of these mixture types focusing on the purported effects of the mixing and compaction temperatures. The results and analysis are presented in this chapter.
Chapter 6	Presents the Wheel Tracking Test (WTT). The results, analysis and findings are stated in this chapter.
Chapter 7	This chapter focused on the structural design and analysis using the mechanistic (non-linear elastic) approach for the CAEMs.
Chapter 8	States the main findings, conclusions and future recommendations.

2. LITERATURE REVIEW

2.1 Introduction

This chapter presents a general overview and introduction into the current state of cold mix asphalts in the industry. It starts with a brief introduction into pavement structures, the process of obtaining RAP, the various types of recycling including foamed asphalt mixtures. The chapter then focuses on the use of CAEMs with bitumen emulsions which includes a brief history of bitumen emulsions, review into its composition and characteristics, mix and structural design considerations for CAEMs. Material properties, compaction and curing techniques are also reviewed.

A pavement is defined as a structure comprised of natural ground with varying layers on top capable of spreading loads for many years and millions of traffic load applications. A typical pavement structure with varying layers is depicted below in Figure 2-1.

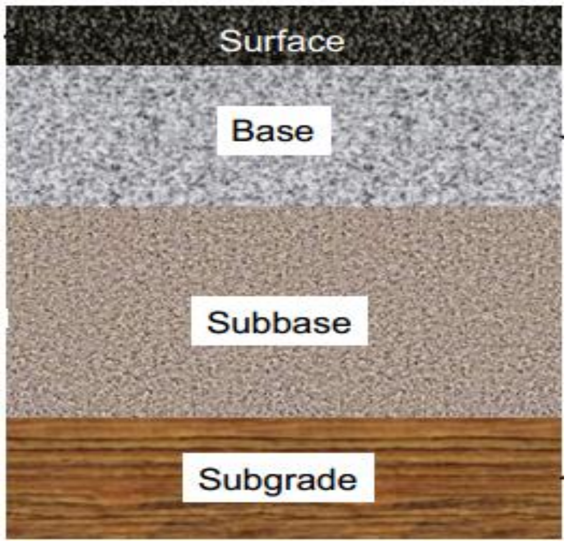
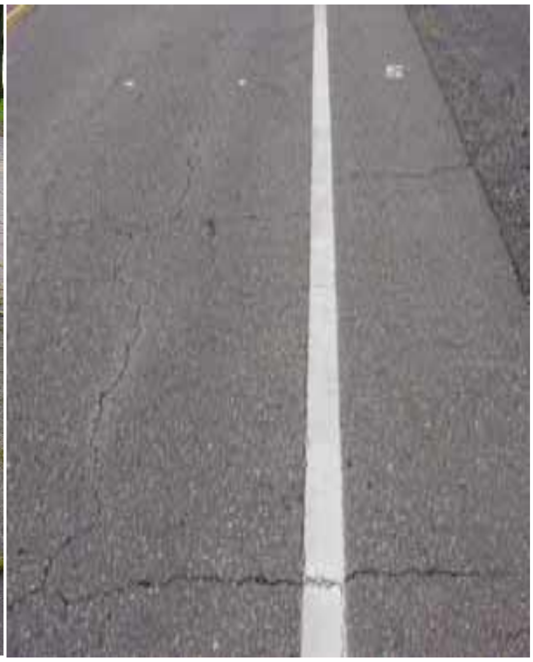


Figure 2-1: A Typical Pavement Structure

Pavements are designed to provide durable all-weather travelling surfaces for safe and speedy movement of vehicles carrying people and goods with a reasonable level of comfort to its users according to Fwa (2003). Road pavements are valuable assets as they constitute primary means of communication and transportation of people, goods and services on a daily basis. The design and construction of pavements requires a fundamental understanding of materials as once they are open to traffic loads, the pavements gradually deteriorate with time, traffic load applications and change in climatic conditions. Lack of maintenance on the pavement could result in rutting, fatigue cracking and other distress types that eventually result in an unacceptable ride quality for users. Typical pavement failure modes are depicted in Figure 2-2. Ibrahim (1998) stated that a pavement is considered to have failed when the deformation of its components is large enough to cause either an uneven riding surface or cracking of the pavement due to repeated stresses over a prolonged period of time. In his research, he identified the major causes of permanent deformation (rutting) to include densification of materials and accumulation of shear deformation due to the repeated action of shear and tensile stresses caused by factors including poor mix design, weak foundation and excessive traffic loadings. Soenen et al., (2003) defined fatigue as failure resulting from the repetitive action of traffic loadings on the pavement that develops into the formation of cracks. They identified the various phases in the fatigue failure process that include the initiation of minor hairline cracks followed by the development of micro cracks that leads into wider macroscopic cracks that combine to form a network of fatigue cracks. The final stages of these macroscopic cracks then propagate through the depth of the pavement resulting in failure with loss of structural integrity of the pavement.

Longitudinal and transverse cracking are depicted in Figure 2-2 (a) and (b) while rutting due to densification and accumulation of vertical shear deformation in the wheel path is depicted in Figure 2-2 (c). Potholes are shown in Figure 2-2 (d).



(a) Longitudinal and Alligator Cracking

(b) Transverse Cracking

(c) Rutting

(d) Potholes

Figure 2-2: Typical Pavement Defects

Failure modes in pavements are usually avoided through proper design, adequate maintenance, rehabilitation and reconstruction solutions as a pavement structure needs to be able to withstand repeated loads from vehicle axles and the effects of climatic changes (moisture and temperature) on the pavement structure in an economical and safe manner over a long period of time. To achieve this, the proper design of the pavement needs to be adequately thought through. Fwa, (2003) stated that the functional requirements of pavements are achieved through careful consideration and selection of pavement type, materials, structural thickness design for pavement layers, surface drainage and ride quality. This forms a standard benchmark for consideration in the initial design of pavements. In recent times, considerable research is been done with respect to the rehabilitation and construction of roads using cold mix asphalts. This development is due to emphasis on the benefits of rehabilitating and upgrading existing pavements in an economic, sustainable and environmentally friendly manner. Brundtland (1987) defines sustainable development as the "ability of the present generation to meet its needs without compromising the ability of future generations to meet their own needs". The increased use of recycled asphalt pavements enables the construction industry to construct and rehabilitate road structures without unduly depleting natural resources. Recycling of existing pavements has become an increasingly important factor in the UK for maintenance of highways. The use of CAEMs is developing in the United Kingdom as well as in many other countries as a result of the changes in how materials are specified which allow for innovation and alternatives to virgin aggregates. There is a shift from strict empirical design methods to performance based specifications which permit alternative innovative options with the appropriate mechanical and performance properties based on performance (Merrill et al., 2004).

2.2 Recycling of Asphalt Pavements

Epps et al., (1980) highlighted the fact that recycling of asphalt pavements is a logical and practical way to preserve diminishing supplies of construction materials and to help reduce the cost of rehabilitating existing road networks. Brown (2000) stated that if properly designed and constructed, recycled asphalt pavements perform as well if not better than pavements built with new virgin materials. Chen et al., (2009) highlighted the inconsistent performance of recycled asphalt mixtures as hampering widespread application of these mixtures. Albeit, in recent times, the use of recycling asphalt pavements has been on the increase. To illustrate this, In America, the asphalt pavement industry recycles approximately 73 million tons of materials annually, which doubles the combined total for recycled paper, glass, plastic and aluminium. Recycling of asphalt pavements can be categorised broadly into hot and cold mix recycling and within these categories hot in-place, hot in-plant, cold in-place and cold in-plant recycling as categorised by ARRA (2001).

2.2.1 Hot Mix Recycling

Hot mix recycling is defined as a process that combines RAP with virgin aggregates, bitumen and sometimes recycling agents to produce hot mix asphalt. The RAP may be obtained by pavement milling with a rotary drum, cold milling machine or from a ripping/crushing operation (ARRA, 1992). Mucinis et al., (2008) stated that hot mix recycling is the most common method of recycling asphalt pavements and can perform as well as mixtures with entirely new materials.

Their research highlighted the fact that the residual aged binder from the RAP could result in a reduction in the need for new binder in the mix. Guidelines as to where the recycled mix can be used in the pavement structure and the quantity to be included vary by clients and supervising agencies. McDaniel and Shah (2003) stated that some agencies allow 15-25% while others permit larger amounts of RAP of which adjustments in mix design and binder selection may be required.

2.2.2 Cold Mix Recycling

Cold mix recycling is a process of recycling without the use of heat where existing asphalt pavements are pulverised, mixed with new virgin aggregates and stabilising agents to produce a new material that is expected to meet the standards and requirements for its use as stated by Epps (1990). Cold recycling is an economical technique since the material does not need to be heated contributing to a reduction in energy, fuel and material consumption. There are differing opinions on its use and applications with Milton and Earland (2009) stating that the process can be conducted either in-plant or in-place at ambient temperatures with the addition of hydraulic binders such as cement and/or bituminous binders. Epps (1990) was of the opinion that cold mix recycling should only be used to form a base course for low to medium traffic volumes due to the fact that they are not structurally as strong as hot mixtures. Needham (1996) was of the view that cold mixtures can be used as a wearing course on all but the most highly trafficked roads. Carswell et al., (2008) stated that provided the cold recycled material can achieve the designed mechanical and performance properties, its potential use should not be limited which highlights the progress with respect to time and research effort of these mixture types.

2.3 Cold In-Place Recycling

Cold in-place recycling is a rehabilitation technique where existing asphalt pavement materials are reused and mixed in-place without the application of heat. The RAP is obtained by milling or crushing the existing pavement. Virgin aggregate, recycling agents or both are usually added to the RAP which is then laid and compacted (Wood et al., 1988). The use of cold in-place recycling can restore old pavement to the desired profile, eliminate existing wheel ruts, potholes, irregularities and rough areas. It can also eliminate transverse, reflective, and longitudinal cracks (ARRA, 1992). This method for the maintenance and rehabilitation of pavements promotes sustainability and helps in limiting the use of scarce materials that include gravel and crushed rock. Cold in-place recycling promotes a high production rate of asphalt mixtures resulting in cost savings, minimum traffic disruption, ability to retain original profile and environmental benefits all without the use of heat according to Loizos and Papavasiliou (2006). Kandhal (1984) expressed the fact that cold in-place recycling is more suitable than cold in-plant recycling particularly for secondary low-volume roads that are located at a considerable distance from a central plant as the process does not involve hauling RAP to a central plant followed by hauling the cold recycled mix back to the job site which could be cumbersome. Figure 2-3 shows the milling process for cold in-place recycling. Cold in–place recycling is carried out using specialised recycling machines comprising of a milling drum equipped with a large number of hardened steel picks. The drum rotates upwards, milling the material in the existing road as Figure 2-3 shows. As milling of the distressed pavement takes place, water is injected through a flexible hose spraying it straight into the mixing chamber.

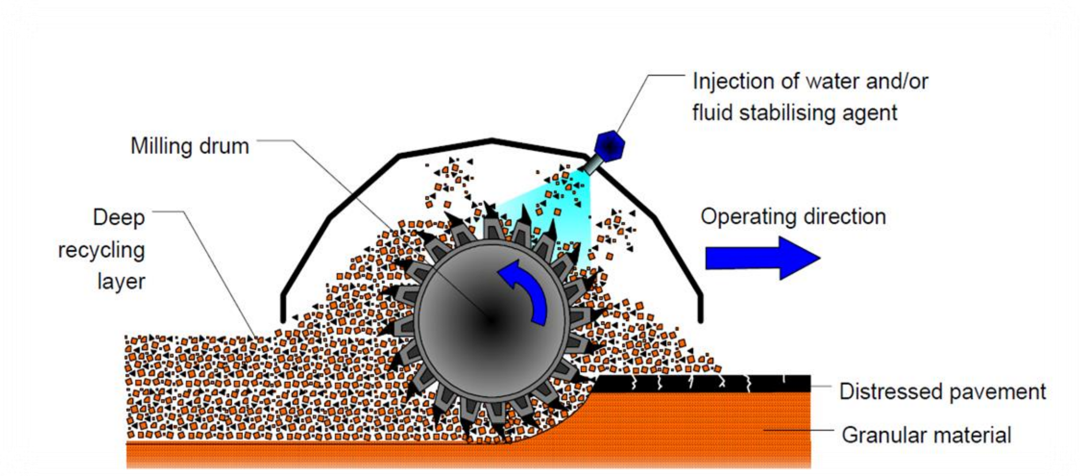


Figure 2-3: Cold in-place recycling – The Milling Process

The water is accurately measured through the use of a controlled microprocessor pumping system. This is important to ensure that the materials are thoroughly mixed together and design specifications are realised (Wirtgen, 2005).

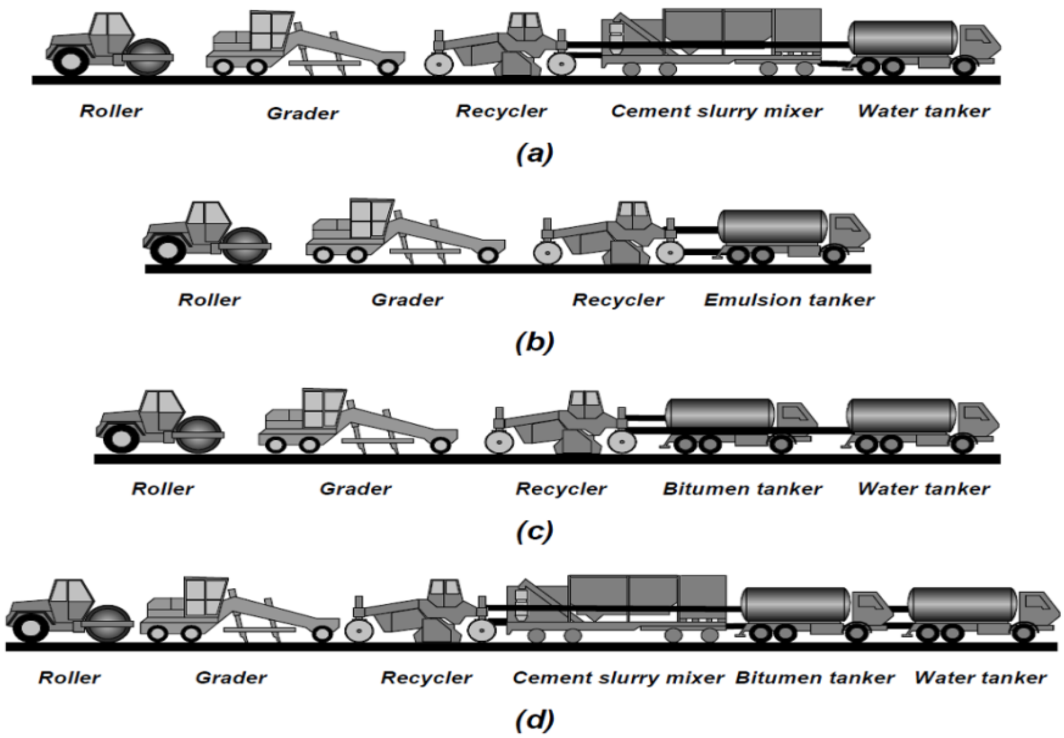


Figure 2-4: Typical configurations for a recycling train (a) cement (b) bitumen emulsion (c) foamed bitumen (d) combination of either bitumen emulsion or foamed bitumen and cement (Lewis and Collings, 1999).

The set-up of the specialist recycling equipment used varies depending on the application, location type of stabilising agent, road dimension and design specifications. Lewis and Collings (1999) categorised typical set-up and configurations of recycling trains as shown in Figure 2-4. There are many cold recycling machines available with production from various manufacturers that include Wirtgen, Roadtec and Caterpillar. Figure 2-5 shows a Roadtec RX-900 in operation.

Figure 2-5: Cold in–place recycling train (Roadtec, 2015)

Usually, a bituminous overlay is placed on the recycled base layer and this ranges from a thin surface layer for lightly trafficked road to a combination of one or more layers of hot mix asphalt courses for heavily trafficked roads in order to attain design specifications (Jitareekul, 2009). Milling techniques are constantly improving with improved screening and processing whereby substantial portions of underlying unbound materials are recycled and reused to produce cold mix asphalts (ARRA, 1992).

2.4 Classification of Cold Recycled Materials

Figure 2-6 presents a classification method based on stabilising agents for recycled materials.

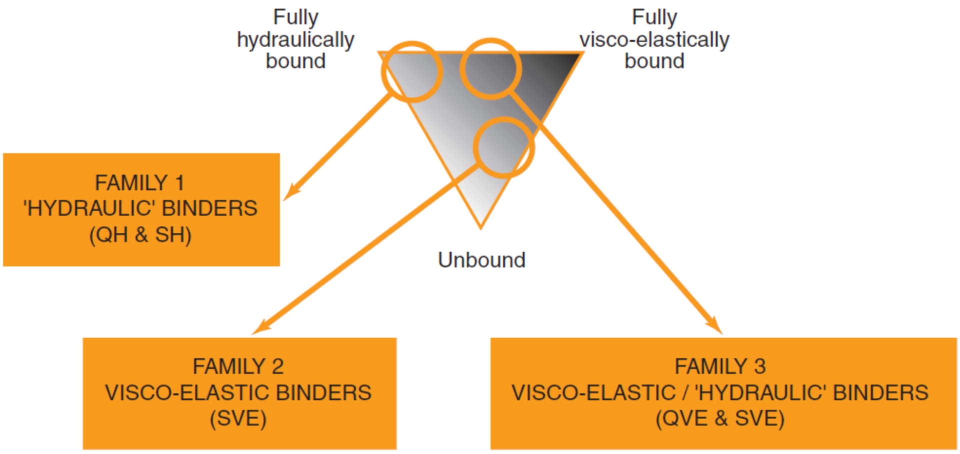


Figure 2-6: Classification of Cold Recycled Materials (Merrill et al., 2004)

The apexes correspond to fully hydraulically bound, fully visco-elastically bound and unbound materials while recycled materials using combinations of binders and curing behaviour are characterised by areas within the chart of which four material types can be obtained and defined as:

1. Quick Hydraulic (QH): Hydraulic binders only including cement.
2. Slow Hydraulic (SH): Hydraulic binders only which include Pulverised Fly Ash (PFA), Lime and Granulated Blast Furnace Slag (GBS). Bituminous binders and cement are excluded from this category.
3. Quick Visco-Elastic (QVE): Bituminous and hydraulic binders including cement.
4. Slow Visco-Elastic (SVE): Bituminous binder only or a mixture of bituminous and hydraulic binders excluding cement.

2.5 Materials for Cold In-Place Recycling

Jitareekul (2009) stated that materials for recycling asphalt pavements can be categorised into three main components that include RAP, aggregate substructure and stabilising agents.

2.5.1 Reclaimed Asphalt Pavement (RAP)

RAP is the pulverized excavated material that has been recovered usually by milling that is used as an aggregate material for the rehabilitation and maintenance of roads. The use of RAP as an alternative to new virgin aggregate materials is gaining worldwide attention as a sustainable, economic, widely available and environmentally friendly option. The RAP to be used should be properly tested and characterised to ascertain its properties that include the gradation, moisture content, density, elongation and flakiness index, the residual binder content, compatibility, penetration and softening point of the residual binder in the RAP.

2.5.2 Aggregate Substructure

In most cold in-place recycling, the entire pavement layer can be milled and pulverised together with a proportion of underlying material and then treated to produce a new stabilised road base. A percentage of virgin aggregates is added if the in-place material is not sufficient or to the right standard to provide the desired depth of the treated layer (Epps, 1990).

2.5.3 Stabilising Agents

Stabilising agents aid in binding the materials together providing an increase in strength, durability and water resistance of the pavement (Epps et al., 1980). Stabilising agents can be categorised into hydraulic and bituminous stabilising agents. Bitumen emulsion and foamed bitumen are bituminous stabilising agents. Cement, hydrated lime, pulverised fly ash, ground granulated blast furnace slag are typical examples of hydraulic stabilisers.

2.5.3.1 Stabilising with cement

Modarres et al., (2011) stated that adding cement to recycled mixes with bitumen emulsion leads to the production of a crystallized pozzolanic structure that increases cohesion, stiffness and resistance to rutting in the mix. Merrill et al., (2004) stated that materials bound with cement cure more quickly than with other types of hydraulic binder and visco-elastic materials. It is typical for 1% to 4% cement to be added in cold in-place recycled mixtures. Giuliani and Rastelli (2004) stated that cement acts as an effective regulator in the breaking phenomenon of the bitumen emulsion. Brown and Needham (2000) found in their research that the three key mechanical properties of stiffness modulus, resistance to permanent deformation and fatigue strength are all improved by the addition of OPC and this could be explained by various mechanisms that include improved rate of emulsion coalescence after compaction, cement hydration and enhancement of binder viscosity.

2.5.3.2 Stabilising with foamed bitumen

Foamed bitumen is produced by injecting compressed air and small amounts of water droplets of about 1%-5% of the mass of bitumen under high pressures (typically 5 bars) into hot bitumen at temperatures ranging from 140°C-180°C in an expansion chamber transforming the bitumen into foam as depicted in Figure 2-7.

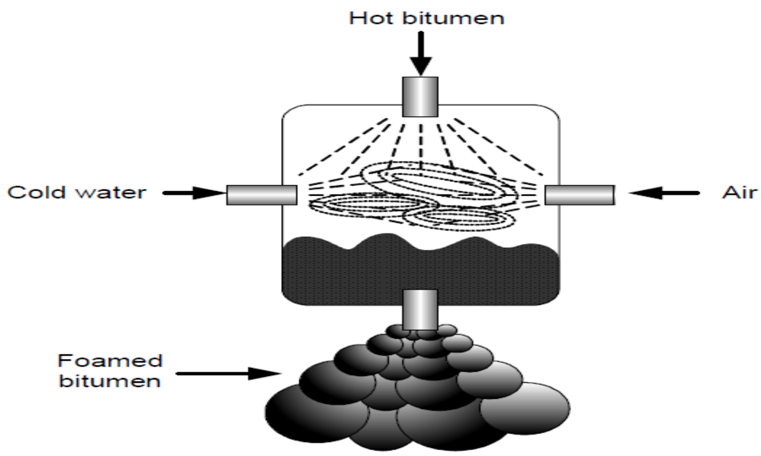


Figure 2-7: The Production of Foamed Bitumen

Foamed bitumen can be used as a stabilising agent with a variety of aggregate materials. Acott (1979), Ruckel et al., (1980) and Lee (1981) conducted studies on foamed asphalt mixtures using virgin materials while Van Wijk et al., (1983), Brennen et al., (1983) and Akeroyd (1988) made use of RAP. Maccarrone et al., (1994) introduced a new “FOAMSTAB” process that had superior fatigue characteristics, less sensitivity to extreme weather and rapid curing. Cement or lime is normally added in conjunction with the foamed bitumen to improve the properties of the mixtures. The foaming process enables normal grades of bitumen to be mixed with cold, wet recycled material without first having to emulsify them.

The use of foamed bitumen has been on the increase in cold recycling with researchers including Jenkins (2000), PIARC (2003), Kim and Lee (2006), Kim et al., (2007), Sunarjono, (2008) and Jitareekul (2009) conducting extensive research and investigations with foamed bitumen. The main advantages are: foamed bitumen makes use of standard penetration grade bitumen utilising only a small percentage of water typically about 2% by mass of bitumen. Foamed bitumen can be placed, compacted and opened to traffic almost immediately, foamed bitumen remains workable for extended periods and can be stockpiled for later use (Wirtgen, 2004). Improvements in the technology of foaming bitumen make its use very attractive. Figure 2-8 shows a laboratory scale foamed bitumen plant. Foamed bitumen was laid on the A43 in Northamptonshire which performed for over 12 years.

Figure 2-8: A Wirtgen Foam Bitumen Rig

2.6 Recycling with Bitumen Emulsion

2.6.1 Introduction

Bitumen emulsion is a dispersion of fine minute droplets of bitumen into water manufactured by using emulsifying agents to emulsify bitumen in water. A major objective of using bitumen emulsions is to obtain a product that can be used without heating (AEMA, 2009).

2.6.2 Brief History of Bitumen Emulsions

The 20th century saw the development of the first emulsions and its use for road construction started around 1920. It was mainly used in spray sealing applications and to prevent dust from ever increasing traffic on roads as stated by Le Coroller (1999). There was a decline in its use at the end of World War II due to a rise in the use of hot mix asphalt to withstand increasing traffic loads and volumes. This did not persist for long and factors including the energy crisis in the early 1970s prompted the conservation of oil shifting focus to the use of bitumen emulsions over hot bitumen as they use less energy with reduced pollution and emit little to no greenhouse gases making them environmentally friendly and sustainable. Bitumen emulsions are capable of coating cold and damp aggregate surfaces which reduce the amount of fuel required for heating and drying of aggregates. This has resulted in a renewed increase in demand for bitumen emulsions as detailed by AEMA (2009).

The major uses of bitumen emulsions are stated in Table 2-1.

Major Uses of Bitumen Emulsion		
Recycling	**Surface Treatments**	**Other Uses**
▪ In-place ▪ In-plant	▪ Surface dressing ▪ Micro-surfacing ▪ Fog sealing ▪ Sand sealing ▪ Cape sealing	▪ Stabilisation (soil and base) ▪ Maintenance patching ▪ Bond and Tack coats ▪ Dust palliatives ▪ Prime coats ▪ Crack filling ▪ Protective coatings

Table 2-1: Major Uses of Bitumen Emulsions (AEMA, 2009)

Le Coroller (1999) gives an indication of the world consumption level of bitumen emulsions in Table 2-2.

Country	Annual Production of Bitumen (Millions of Metric Tons)	Consumption per head (kg/person)	Bitumen used in Emulsions (%)
United States	2.26	8.53	4.8
France	1.01	17.4	24.8
Mexico	0.51	5.65	37.2
Brazil	0.41	2.54	18.8
Spain	0.35	9.00	17.5
Japan	0.32	2.63	5.1
United Kingdom	0.16	2.72	4.7

Table 2-2: Usage of Bitumen Emulsions (Le Coroller, 1999)

The world production of bitumen emulsions was estimated to be about 7 million tonnes at the end of the 20th century. The United States is the major producer and France utilises the highest amount of bitumen emulsions (Le Coroller, 1999). James (2006) estimated that 5-10% of the world paving grade bitumen is used in producing bitumen emulsions.

2.6.3 Composition of Bitumen Emulsions

Bitumen emulsions are two phase systems consisting of two immiscible liquids. Major components include bitumen, water and emulsifying agents. In some cases, the emulsions may contain additives such as stabilizers, coating improvers, anti-strips or break control agents (Asphalt Institute, 1989). The bitumen is dispersed in a continuous aqueous phase in the form of discrete globules of mostly 0.001 to 0.01 mm held in suspension by electrostatic charges stabilised by the use of an emulsifier (Wirtgen, 2004). Emulsifiers consist of polar (hydrophilic) and non-polar (hydrophobic) groups. This unique arrangement enables the emulsifier to orientate itself amongst the bitumen soluble group which is hydrophobic and the water soluble group which is hydrophilic (Austroads, 2008). The bitumen particles are held in suspension and do not coalesce readily due to the presence of the emulsifier that is concentrated on the surface of the bitumen particles as stated in Gibbs (1996) and BP (2014).

2.6.4 Classification and Breaking of Bitumen Emulsions

Bitumen emulsions must remain in a stable state so they can be transported, stored and handled. Ultimately, they must be made to separate or break so that the bitumen can coat aggregate particles or pavement surfaces (BP, 2014). The main effects of the bitumen emulsion take place when the water phase is separated from the bitumen phase and this is termed breaking of the emulsion and this occurs mainly when the emulsion comes in contact with the aggregate material.

The coating of the bitumen particles by the emulsifier gives them an electrostatic charge which acts as a major criterion for the classification of bitumen emulsions. The three major categories of emulsion are anionic emulsions in which the bitumen droplets have a negative charge, cationic emulsions in which the particles are positively charged and non-ionic emulsions that have no charge on the bitumen droplets. Emulsions are further classified based on how quickly they break as stated in Table 2-3. This is to further simplify and standardize its use.

Anionic	**Cationic**	**Setting Mode**
ARS	CRS	Rapid Setting
AMS	CMS	Medium Setting
ASS	CSS	Slow Setting

Table 2-3: Classification of Bitumen Emulsions (BP Bitumen, 2014)

Based on the basic law of electricity that states that like charges repel each other while opposite charges attract, if the electrostatic charges on the bitumen particles are opposite to that of the aggregate surface, electrical attraction will take place. The bitumen particles then start to migrate to the aggregate surface. This migration will cause the emulsion to 'break' and start to separate into its original components: water and bitumen. The breaking of emulsion depends highly on the type and concentration of the emulsifying agent as stated by Das (2008). The ability of the bitumen emulsion to break facilitates effective coating of the aggregate particles or pavement surfaces. The rate at which emulsions break depends on its use and mode of application.

To illustrate, emulsions used in surface dressing are required to break relatively quickly to prevent run-off of the emulsion and the possibility of rain damage, while emulsions used in stabilisation of soil must break relatively slowly to allow adequate mixing (BP, 2014). The breaking of bitumen emulsions is usually characterised by a change in colour from brown to black as observed by Oke (2010). The optimum balance between stability and breaking rate is principally obtained by careful selection of emulsifier type and concentration, emulsion pH and bitumen droplet size. The charge on the emulsion is another factor as anionic emulsions work most effectively with positively charged aggregates that include limestone and marble. They tend to rely more on evaporation of the water for the breaking and curing processes to occur which has a direct implication on the breaking and curing rates although water displacement can be fairly rapid under favourable conditions. Cationic bitumen emulsions break through an electrochemical process and therefore weather conditions play a lesser role in the breaking rate of these emulsion types. Full curing of a cationic emulsion still requires the water to be lost through evaporation, absorption or 'pushing out' by the action of rolling and traffic as stated in BP (2014). Cationic emulsions react effectively with predominantly negatively charged aggregates giving a more rapid break and better adhesive properties. The feasibility of using both negatively and positively charged aggregate particles has been exploited by companies resulting in the production of cationic emulsions that work well irrespective of the charge on the aggregate particles or weather conditions (Nynas, 2010). In both anionic and cationic emulsions, high humidity, low temperatures or rainfall soon after application can severely delay full curing. Figure 2-9 depicts a schematic diagram of a cationic emulsion. The bitumen droplet is suspended in the continuous water phase by the positively charged emulsifier.

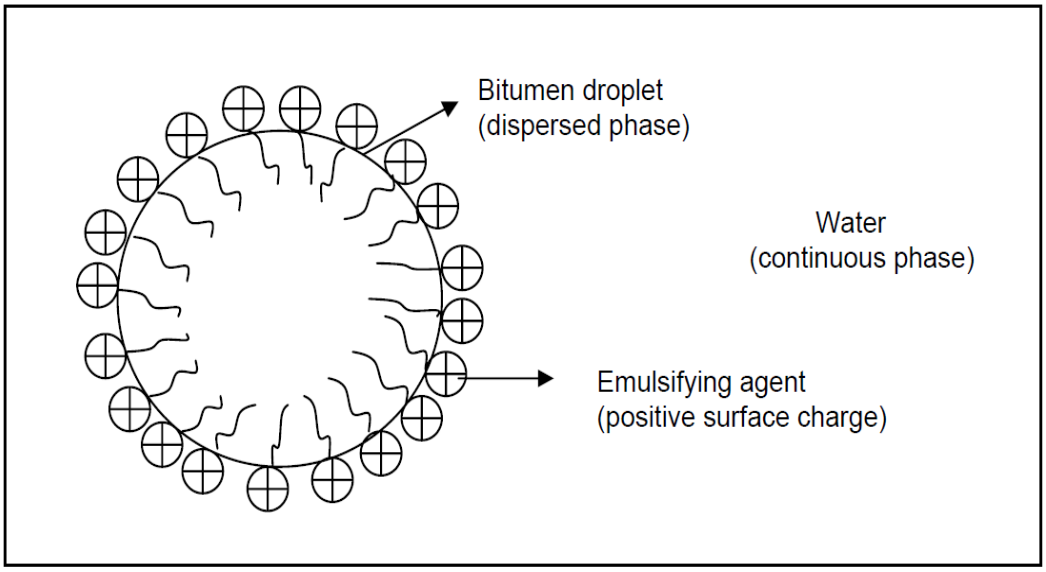


Figure 2-9: Schematic of a Cationic Bitumen Emulsion

2.6.5 Curing of Bitumen Emulsions

Bitumen emulsion is fully cured when the water and/or any volatile oils have evaporated and cohesive bond strength is established between the binder and aggregate. The water can be removed through evaporation, pressure (rolling) and by absorption into the aggregate (Austroads, 2008). Evaporation of the water is highly dependent on weather conditions and it is difficult for bitumen emulsions to properly cure when subjected to high humidity, low temperatures and rainfall soon after application (AEMA, 2009). Figure 2-10 shows the breaking, setting and curing stages of bitumen emulsions. Curing facilitates the development of mechanical and performance properties of the mixture. AEMA (2009) states that the major factors that affect breaking and curing of bitumen emulsions include the type and amount of emulsifier, weather conditions as warmer temperatures are favourable to breaking of bitumen emulsions which facilitates curing.

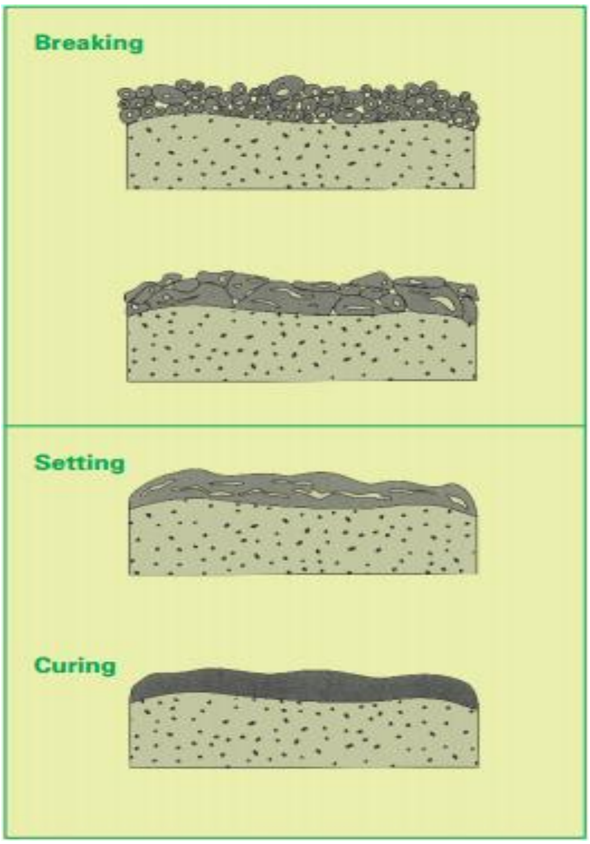


Figure 2-10: Major Stages for Bitumen Emulsions (BP, 2014)

Continuous research and development in bitumen emulsion technology has resulted in chemical formulations that ensure bitumen emulsions break rapidly at cool temperatures as shown in Nynas, (2010). Other factors that affect breaking and curing of bitumen emulsions include water absorption rates of the aggregate as a rough textured porous aggregate absorbs water faster which helps in speeding up the setting time. Mechanical forces such as slow moving traffic help to force water out of the emulsion which facilitates mix cohesion and stability. Surface chemistry that takes into account the intensity of the aggregate surface charge and the emulsifier charge has an impact on the setting rate of the bitumen emulsion. Thom (2009) stated that the aggregates need to be pre-wetted in order to produce reasonable binder coating. The pre-wet water content should be within appropriate limits as high moisture content of the aggregate material will slow the curing process.

2.6.6 Tests on Bitumen Emulsions

Tests on bitumen emulsions are carried out in order to ensure that specifications and requirements are met. Typical tests on bitumen emulsions are stated in Table 2-4.

Test	Description
Particle Charge Test	The test aims to determine the charge on the bitumen droplets of the emulsion. Positive and negatively charged electrodes are left in a sample of emulsion for half an hour. If bitumen is deposited on the negative electrode, the emulsion is cationic. If bitumen is deposited on the positive electrode, the emulsion is anionic.
Setting Time Test	This test indicates the time taken for a sample of bitumen emulsion to break under controlled set of conditions when mixed with a standard aggregate material.
Water Content Test	This test determines the percentage mass of water in an emulsion.
Viscosity	This test measures the rate of flow of bitumen emulsion at 25°C. The emulsion is heated to 25°C and poured into a standard flow cup. The time taken by 200mL of emulsion to pass through a standard orifice at the bottom of the container is measured.
Coating and Water Resistance	The test measures the ability of the bitumen emulsion to withstand the action of being mixed and coated with aggregates as completely as possible without been washed off by any water that may fall on it once mixing is complete.

Table 2-4: Typical Tests for Bitumen Emulsions

2.6.7 Factors Affecting Quality and Performance of Bitumen Emulsions

There are many factors that can affect the production, storage, handling, use and performance of bitumen emulsions. AEMA (1997) stated that these factors could significantly affect the performance and quality of the bitumen emulsions either in a positive or negative manner and they include: The type and concentration of the emulsifying agent, manufacturing conditions that include temperature, shear and pressure, charge on the emulsion particles, order of ingredient inclusion, chemical properties of the asphalt cement including hardness and quantity, type of equipment used in manufacturing the emulsion, chemical properties of the emulsifying agent, the addition of chemical modifiers or polymers, quality of the water and quantity of emulsion in the mix.

2.6.8 Benefits and Issues with Bitumen Emulsion

Advantages of using bitumen emulsions include:

- They can be used with damp aggregates.
- Elevated temperatures are not required for proper use and application.
- They provide economic, environmental and sustainable benefits.

The major problems with the use of bitumen emulsions is that the produced mixtures have slow curing rates which means that time is required for stiffness and strength to develop. Prime bitumen emulsion products could be expensive.

2.7 Mix Design and Structural Design Considerations

2.7.1 Introduction

The principal objective of a mix design is to produce mixtures that fulfil their structural and functional requirements. FHWA (1997) stipulated that guidelines have been developed by several agencies, based on empirical formulae, laboratory tests or past experiences but there is still no widely accepted mix design standard or structural design methodology currently available for cold recycled mixtures. Table 2-5 summarises the major mix design procedures. The basic underlying principles and design procedures these mix design methods entail as stated by Epps (1990) and (Thanaya, 2007) include:

- Determine suitable aggregate gradations.
- Evaluate properties of the aggregate materials.
- Select type and amount of stabilising agents.
- Aggregate - binder affinity/suitability (binder coating tests).
- Determine the optimum water content at compaction.
- Determine the optimum bitumen content.
- Mixture, compaction and testing of trial mixture.
- Establish job mix formula.
- Testing for mechanical and performance properties.
- Field trials.

In recent times, the trend for mix design is progressing towards performance related and performance based approaches as defined in Table 2-5. Table 2-6 highlights the mix design approaches adopted in various specifications and organisations.

Mix Design Procedure	Description
Empirical Mix Design	Empirical mix design involves optimization of variables by conducting mechanical tests that take into account previous experience or results of experiments. It usually requires a number of observations to be made in order to ascertain the relationships between input variables and outcomes.
Recipe Based Design	Recipe design is based on experience of traditional mixes of known composition. This is an experience based approach, which has shown good performance over a long period of time and under given site, traffic and weather conditions.
Analytical Based Design	The analytical method does not consider preparation of any physical specimen. Composition is determined exclusively through analytical computations.
Volumetric Method	In the volumetric method, proportional volume of air voids, binder and aggregates are analysed in a mixture, which is compacted in the laboratory by some procedure close to the field compaction process.
Performance Related Approach	In performance related approach, the specimens that meet volumetric criteria are compacted and tested with simulation and/or fundamental tests to estimate properties that are related to pavement performance.
Performance Based Approach	This approach is based on the performance of the complete system. Laboratory instrumentation tends to simplify the situation, yet it is indeed difficult to simulate field conditions. The Superpave mix design recommends use of the Superpave shear tester, and the indirect tensile tester for evaluation in the laboratory of the bituminous mix. These tests are basically accelerated performance tests of bituminous mixes.

Table 2-5: Summary of Mix Design Methods (RILEM, 1998)

Specification/Organisation	Country	Category
NARC 96-I-III	Australia	Recipe/Volumetric/Performance related
ASTO/PANK"95	Finland	Recipe/Volumetric/Performance related
AFNOR	France	Recipe/Volumetric/Performance related
DIN	Germany	Recipe/Empirical
CROW	The Netherlands	Volumetric/Performance related
BS 594/598	UK	Recipe/Empirical
Asphalt Institute	USA	Empirical/Volumetric
SHRP Superpave	USA	Volumetric/Performance related/Performance Based

Table 2-6: Mix Design Approach Adopted in Various Specifications/Organisations (RILEM, 1998)

It is common practice for a selection of performance tests including stiffness, fatigue, permanent deformation, temperature susceptibility, low temperature cracking, moisture susceptibility and permeability tests to be incorporated in order to obtain a design that meets stated structural and functional requirements. Considerations that factor economic advantages, environmental benefits and sustainability play a crucial role in mix designs in recent times (Das, 2000). In recent times, other factors including noise, skid resistance, spray are been incorporated into performance requirements.

2.7.2 Evaluation of Materials

The major materials used in producing CAEMs include:

- Reclaimed Asphalt Pavements (RAP).
- Virgin aggregate materials.
- Stabilising agents.

RAP aggregate materials can be obtained from pulverised/crushed field samples, extracted pavement cores or samples produced and crushed in the laboratory. Ideally, it is advised that pulverised/crushed samples should be obtained from field where possible (Jitareekul, 2009). All materials used should be representative for both grading and shape with their properties properly characterised and evaluated (Milton and Earland, 1999). In characterising and evaluating the properties of RAP aggregate materials, Emery (1993) stated that these properties are particularly important which include aggregate gradation, particle density and water absorption, moisture content, RAP binder composition (binder content, gradation after extraction, softening point, penetration index) and the physical properties of the aggregates (shape, elongation and flakiness index). This information is valuable as it is the first step in characterising the aggregates and understanding the material properties.

Stabilisers used for cold in–place recycling projects should be properly researched and evaluated to ensure that they are the right fit for the project. FHWA (1997) opined that RAP aggregate materials be treated as black rock however, Ojum and Thom (2014) found that the residual binder in RAP aggregate material that has not severely aged with penetration $\geq$ 20dmm positively influences the mechanical and performance properties of CAEMs. FHWA (1997) stated that the most commonly used recycling agent for complete cold recycling processes are bitumen emulsions as they are liquid at ambient temperatures and they are easily dispersed throughout the mix. Finally, in selecting the appropriate stabilising agents for a project, cost, ease of use, availability, location and compatibility should be taken into consideration.

2.7.3 Job Mix Formula

Needham (1996), AEMA (1997) and Montepara and Giuliani (2002) remarked that the widely used Marshall or Hveem design methods or their modified versions are useful design methods for cold recycled mixtures. The Marshall method seeks to select the asphalt binder content at a desired density that satisfies minimum stability and range of flow values as stated by White (1985) while the Hveem design method introduced the Hveem Stabilometer. The Hveem design method initiated the concept of measuring various mix parameters to obtain the optimum quantity of bitumen. This mix design method was updated subsequently to account for moisture susceptibility and sand equivalent tests. Montepara and Giuliani (2002) stated that these design methods can help in ascertaining the best dosing of the bituminous emulsion and the total content of liquids (optimum fluid content). If cement is to be used, it is imperative to consider the water/cement ratio and evaluate the time to obtain certain mechanical and performance properties as suggested by Oke (2010). Needham (1996) and Oke (2010) stated that the following factors including the following empirical formulae as stated below can provide guidance in ascertaining mix parameters for the cold mixture design in cases where laboratory tests are not available or possible.

1. The Initial Residual Binder:

   Empirical formula for the addition level of emulsion can be estimated as shown below:

Equation 2-1:

$$P = (0.05A + 0.1B + 0.5C) \times (0.7)$$

Where:

P = Percent by weight of initial residual bitumen content by mass of total mixture

A = Percent of mineral aggregate > 2.36mm

B = Percent of mineral aggregate <2.36 and >0.75mm

C = Percent of mineral aggregate < 0.75mm

2. Determination of Aggregate Gradation:

   Cooper et al., (1985) proposed the following equation to aid in determining the aggregate gradation. Cooper's equation provides adequate interlocking properties for the mixture and helps in regulating the fines content as stated by Kamal et al., (2003) and Thanaya (2007).

   Equation 2-2:

$$P = \frac{(100 - F)(d^n - 0.075^n)}{D^n - 0.075^n} + F$$

   Where:

   P = Percentage material passing sieve size (d) in mm

   D = Maximum aggregate size (mm)

   F = Percentage filler

   n = Exponential value that dictates the concavity of the gradation line

3. Estimation of the Initial Residual Bitumen Content and Initial Emulsion Content in line with Equation 2-1 as stated above.

4. Coating tests or binder tests to evaluate aggregate-binder compatibility.
5. Determination of compaction level compaction level to meet porosity target (Oke, 2010);

- Compaction by applying an initially judged compaction effort.
- Curing for dry density determination.
- Determination of specific gravity of the mix and porosity value after obtaining dry density data.
- Adjustment of compaction effort and determination of compaction effort to meet the porosity target. Suggested porosity target is about 5-10%. The following useful formulae were also suggested:

Equation 2-3:

$$SGmix = \frac{100}{\frac{\%CA}{SGCA} + \frac{\%FA}{SGFA} + \frac{\%F}{SGF} + \frac{\%Binder}{SGBinder}}$$

Where:

SG_{mix} = Specific Gravity of the mix

CA = Percentage of Coarse Aggregates

FA = Percentage of Fine Aggregates

F = Percentage of Filler

SGCA = Specific Gravity of Coarse Aggregates

SGFA = Specific Gravity of Fine Aggregates

SGF = Specific Gravity of Filler

Equation 2-4:

$$Bulk\ density = \frac{\%\ Weight\ in\ air}{Weight\ SSD - Weight\ in\ water}$$

Bulk density by weight of total mix where the weight saturated surface dry (SSD) is obtained by towel drying the samples after weighing in water, until no air bubble occurs.

Equation 2-5:

$$Porosity\ (P - \%) = \left(1 - \frac{Bulk\ density}{SGmix}\right) \times 100\%$$

6. Variation of residual bitumen content.
7. Determination of optimum residual bitumen content.
8. Calculation of bitumen film thickness at optimum residual bitumen content.

   The bitumen film thickness (BFT) can be calculated using the formula:

Equation 2-6:

$$BFT = \frac{\%Binder}{100 - \%Binder} \times \frac{1}{SGBinder} \times \frac{\%F}{ASA}$$

Where ASA is the aggregate surface area and requires surface area factor as given in Table 2-7, the calculation of the aggregate surface area (ASA) is calculated by multiplying the total percent passing each sieve size by the appropriate SAF and then adding together.

Figure 2-11 presents a general overview of the mix design procedure for Cold Mix Asphalts (CMA).

Particle/Sieve Size	Surface Area Factor (m^2/kg)
Maximum size (all sizes greater than 4.75mm)	0.41
4.75mm (No. 4)	0.41
2.36mm (No. 8)	0.82
1.18mm (No. 16)	1.64
600µm (No. 30)	2.87
300 µm (No. 50)	6.14
150 µm (No. 100)	12.29
75 µm (No. 200)	32.77

Table 2-7: Surface Area Factor Values (SAF) (Thanaya, 2007)

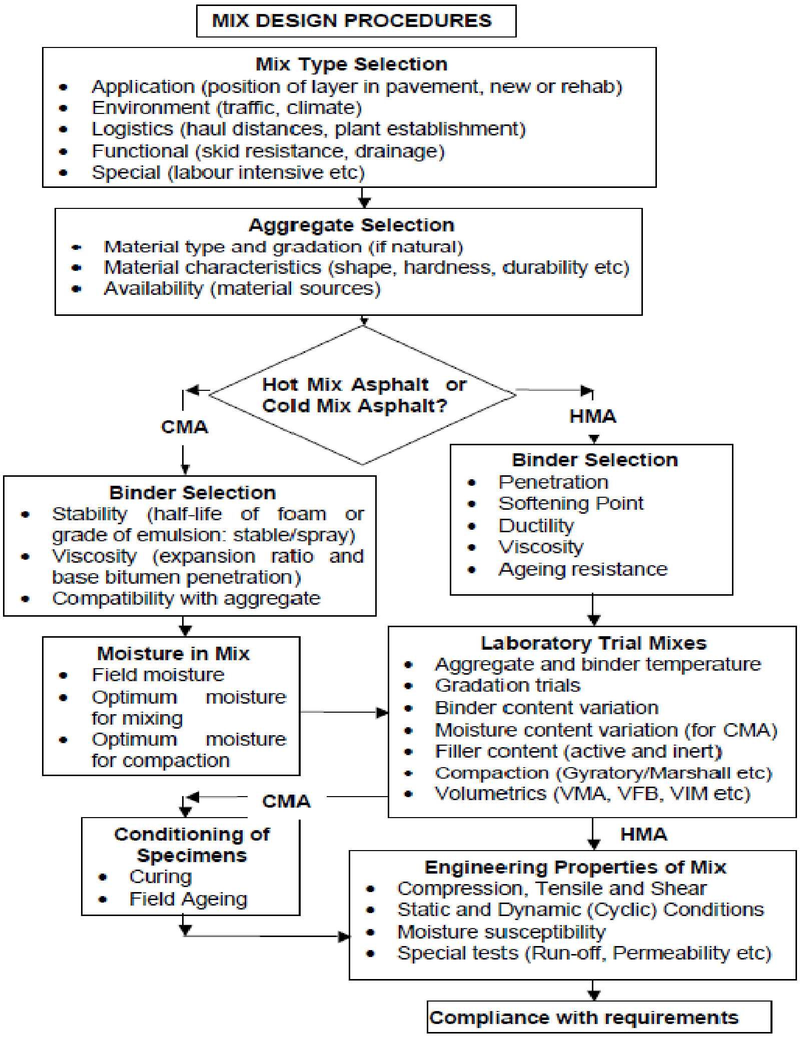


Figure 2-11: General Mix Design Procedure (Jenkins, 2000).

2.7.4 Compaction

Compaction is an important factor that directly affects the mechanical and performance characteristics of the mixtures produced. Adequate compaction ensures that the mixtures perform optimally. Harun and Morosiuk (1995) opined that to produce a mechanically stable material, compaction is best at refusal density although Thanaya (2007) suggested that compaction be done to target porosity within 5-10%. PIARC (2003) stated that the total fluid content needs to be ascertained to ensure adequate compaction of the mix. AEMA (1997) argued that for gradations containing significant amount of fines, drying of the aggregates prior to compaction may be required. Major compaction methods available and commonly used include the Marshall hammer, the roller compactor and the gyratory compactor. The Marshall hammer is useful in preparing compacted specimens for testing not only for stability and flow tests in the Marshall apparatus but also for indirect tensile strength, unconfined compressive strength and indirect tensile modulus tests as stated by Sunarjono (2008). In recent years the gyratory compactor has gained popularity for its use in the preparation of samples. Kim and Lee (2006) in their research stated that 75 blows from the Marshall hammer did not provide adequate compaction to simulate conditions after construction of roads. Kim and Lee (2006) further stated that the gyratory compaction method produced a higher density value than the Marshall compaction method but the samples made by the gyratory compactor exhibited lower resilient modulus values in comparison to samples prepared using the Marshall compactor. Thanaya (2007) points out that to achieve a target air void content of between 5-10%, compaction of at least 240 gyrations is required and this is classified as extra heavy compaction.

Thanaya (2007) further stated that the application of a heavy compaction level is inevitable in cold mixes to ensure that the emulsion sets and the mixes stiffen properly. An excessive quantity of fluids in the mix reduces the efficiency of the compaction process preventing the mixtures from reaching their optimal density resulting in reduced stiffness and strength properties. Serfass et al., (2004) depicts the relationship between the degree of compaction and stiffness modulus on emulsion cold mixes in Figure 2-12. This shows that an increase in the degree of compaction resulted in an increase in the stiffness of the mixtures. All these point to the fact that compaction technique employed in research studies is an important aspect of mix design that should be properly reviewed and considered. It is imperative that the compaction technique selected for research should reflect conditions experienced and obtainable in the field as closely as possible.

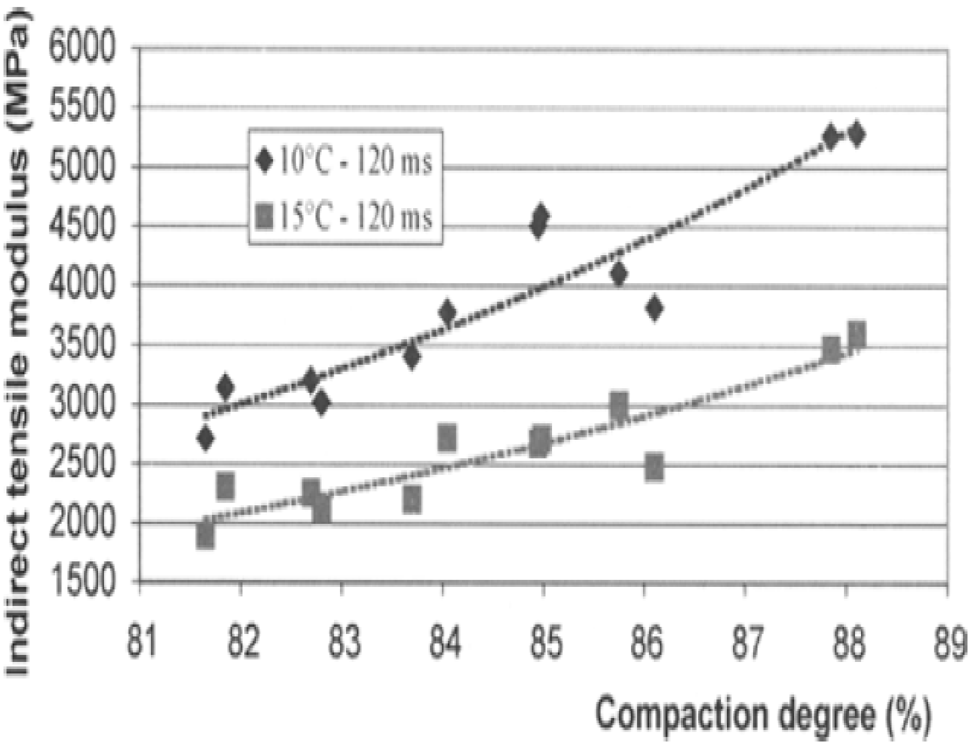


Figure 2-12: The Influence of Compaction Level on Stiffness

2.7.5 Curing

Curing is the process in which mixtures lose their moisture content and the bitumen returns to a continuous phase at elevated temperatures. Jenkins (2000) defined curing of cold bituminous mixes as the process where the mixture discharges water through evaporation, particle charge repulsion or pore-pressure induced flow paths as stated by Oke (2010). Curing and conditioning of specimens is an important requirement for cold mixes. Mixtures gain full strength as water evaporates (Thom, 2009). Lee et al., (1983) showed that moisture loss is directly proportional to strength gain and this is evident at higher temperatures over a period of time. Cold emulsion mixes behave in a peculiar way especially during their early life. This peculiar behaviour results from the combination of several factors including the presence of water, aggregate-emulsion reactivity, binder film coalescence and cohesion build-up (Serfass et al., 2004). Montepara and Giuliani (2002) stated that pavement layers treated exhibited premature distress quickly after construction when trafficked which adversely affected the performance of the pavement. The research showed that cold emulsion mixes display slow early life strength and construction is better in dry conditions. This is to prevent premature damage, susceptibility to stripping and ravelling. Bocci et al., (2002) stated that as curing temperature and time increased, the performance of the mixtures improved significantly. Santucci (1977), Leech (1994) and Thanaya (2007) reported that full curing can occur between 2-24 months in the field. Serfass et al., (2004) indicated that cold mixes experience a fully cured and mature state after a period of time and in most cases a complete cycle of seasons which could be prolonged in cooler and significantly humid conditions. Evaluating cured cold mixes in the laboratory is evidently necessary.

Accelerated curing plays a major role but the major issue to contend with is the ability to reproduce field curing conditions. Serfass et al., (2004) proffered some guidelines with respect to accelerated curing which include: the curing procedure must produce materials as close to their in-place maturity state and must not significantly age the bituminous binder. The curing regime should be as short as possible to prevent ageing of the binder and the laboratory equipment should not be too sophisticated.

It should be noted that cold mixes seldom become totally dry in field conditions. The moisture content has often been found to be between 0.5 and 1.5% in road bases in temperate climates (Oke, 2010). Ruckel et al., (1982) recommended that to obtain cold mixtures in a mature state that represent field conditions, samples should be cured for 1 day at 40°C simulating intermediate curing and at 40°C for 3 days simulating long term curing. This regime was used in research conducted by Marquis (2003) and Jenkins et al., (2004). Serfass et al., (2004) proposed accelerated curing of 14 days at 35°C. The UK Highways Agency's Manual of Contract Documents for Highway Works (MCHW) Volume 1 Series 900 Clause 948 proposed curing at either 40±2°C or 20+2°C for 28 days which depends on if the mix is bound with cementitious, hydraulic or bituminous binders separately or in combination as defined in Figure 2-6. The standard advised that the specimens should be sealed to keep moisture in. The manual further states that these curing regimes simulate field conditions over the first year of the road. Ruckel et al., (1982) highlighted the fact that curing temperatures exceeding 60°C may significantly stiffen the binder thereby altering properties of the mixtures.

Serfass et al., (2004) supports this view and stated that curing at temperatures ≥ 50°C dried the specimens too quickly causing premature failure. Serfass et al., (2004) stated that the moisture content of small specimens evaporated faster in comparison to larger specimens which take a longer time. For this study, a detailed investigation into the effects of accelerated curing on Cold Asphalt Emulsion Mixtures (CAEMs) was conducted of which a major aim was to simulate field conditions as closely as possible.

SABITA (1993) summarised the factors that could affect curing in CAEMs and they are stated below:

- The rate at which the aggregate absorbs water (rough textured, porous aggregates reduce the time it takes for the emulsions to break.
- The moisture content of the mix after compaction.
- The grading of the aggregate and the voids content of the mix.
- The type and quality of the bitumen used in producing the bitumen emulsion.
- The mechanical forces caused by compaction and traffic.
- Mineral composition of the aggregate (physical-chemical interactions between surface of the aggregates and the emulsion).
- Magnitude of the electrical charges of the aggregate in relation to that of the emulsion.

2.8 Structural Design Considerations

The structural design of a flexible pavement is required in order to estimate its design life (Oke, 2010). Structural design is important in order to determine by optimisation the layer thicknesses that should give the structural support required for the design life of a pavement. Thom, (2008) suggested two alternative structural design methods for cold mixtures: the mixtures should be treated as a hot-mix or a superior granular material. Various structural design methods that include the AASHTO Method, the Asphalt Institute Method, the Catalogue Method, SHELL design methods, the CBR Design Method and a UK method developed by Milton and Earland (1999) are available. Structural design and analysis is a complex process as stated by (Ebels, 2008 and Twagira, 2010). Thom (2008) opined that the structural design of pavements should aim to protect the subgrade, guard against deformation and break up of pavement layers, protect from environmental attack, provide suitable surface and ensure maintainability in line with views of Brown, (2012) that in order to design any civil engineering structure, it is vital to understand how it can fail in service. Brown (2012) further stated that a pavement does not suddenly fail but its service to the user gradually deteriorates. The two principal modes of failure in asphalt pavements are fatigue cracking of the bound layers and rutting (cumulative deformation) in the wheel tracks. Brown (2012) highlighted the fact that for design purposes, the key locations to ascertain stresses, strains and deflections in the pavement structure due to traffic loadings are the bottom of the asphalt layer and the top of the subgrade where maximum key parameters develop of which the strains due to cracking and rutting have been considered most critical for the design of asphalt pavements as stated by Gedafa (2006).

Huang (2004) in agreement with Gedafa (2006) stated that the critical points are the horizontal tensile strains (ε_t) at the bottom of the asphalt layer which causes fatigue cracking and vertical compressive strains (ε_z) on the surface of the subgrade that impacts on permanent deformation or rutting in the pavement. Modelling tools involving multi-layer linear elastic analysis are widely available and used to ascertain stress, strain and deflections in the pavement structure under traffic loadings. KENLAYER is one of such tools used for structural design, modelling and analysis. Huang (2004) showed that distress models can be used to predict the life of new pavement structures assuming effective pavement structure configuration. If the reliability for a certain distress is less than the minimum level required, the assumed pavement configuration should be changed. KENLAYER was found useful in this research as the structural design and analysis could be used to obtain key parameters attributable to the non-linear behaviour of the CAEMs. KENLAYER has capabilities of ascertaining the stress, strains and deflections of multi-layer systems under stationary or moving wheel loads with each layer either as linear elastic, non-linear elastic or visco-elastic. Brown and Brunton (1986) proposed a structural design procedure which starts by specifying the traffic loading, estimating the components (layers) and sizes (thicknesses), due consideration for available materials which is then followed by conducting structural analysis using theoretical principles. The critical stresses, strains and deflections are then compared with allowable values to determine whether the design is satisfactory, adjustments to materials or geometry are made until satisfactory design is achieved followed by conducting an economic feasibility study. The use of finite elements in structural design is gaining attention. This is due to its accuracy in calculating critical parameters in various layers within the pavement under various traffic loadings.

2.9 Current Practices for Cold Mixtures

Cold bituminous mixtures are used widely in a number of countries in Europe, South Africa, Australia, New Zealand and USA. This section briefly discusses the various approaches in France, Sweden, Spain, South Africa and the USA.

2.9.1 France

The process known as grave emulsion has been extensively used in France as a base course material to re-profile, overlay and strengthen deteriorated pavements. Grave emulsion has been used also to construct low trafficked roads and for overlaying cement bound base courses to prevent crack propagation. Aggregates used could be crushed rock or gravel that complies with the French standards on durability, angularity and cleanliness. The aggregate gradation is densely graded with high sand content to provide internal friction in the aggregate mixture and good surface texture. Also, the mixture has low filler content to minimise susceptibility to rutting. The maximum specified void content is 15%. The binder content for the grave emulsion is between 3-3.5% which provides partial coating of the aggregate which helps to promote high level of aggregate contact. The issue with the low binder content is that the mixtures are more susceptible to ingress of moisture hence in recent times, there has been a move towards higher binder contents of 4% or more. Usually, a medium setting emulsion is specified to coat the fine aggregates before partial coalescence takes place so that the fine coated mastic acts as discontinuous cement to bind the larger aggregate together. The best aggregate/bitumen proportion to satisfy the specification is ascertained using the Duriez method with details in Needham (1996).

2.9.2 Sweden

About 1 million tonnes of old pavements are recycled each year in Sweden. Cold recycled asphalt materials are the most common method for flexible pavements on roads with low traffic volumes where Average Daily Traffic (ADT) is less than 1500 vehicles per day. Cold recycled bituminous emulsion mixtures in Sweden can be used for wearing courses, base courses or road bases. This is an economical technique since the material does not need to be heated providing additional environmental benefits. There are cases where 100% recycled materials have been reused. Typically, in most cases, about 10-20% virgin aggregates are added. Maximum RAP aggregate size for base course is limited to a maximum of 22mm sieve size. Pre-wet water is usually added to aid with the mixability of the mixture however, if the RAP has a high moisture content (>5%), no pre-wet water is added. The bitumen emulsion content ranges between 2-4%. Mixing is done to achieve a homogenous and well coated mixture. A proper mix design is normally conducted to obtain the optimum parameters including the optimum liquid ratio (water + emulsion). This is obtained by using heavy (Proctor or Marshall) compaction. Typical mix design procedure is shown in Figure 2-13. The specifications for cold recycling are stated in the National Swedish Road Administration guidelines (VTI NOTAT 1-2001) which cover sampling and evaluation of the RAP, guidelines for selecting new binder, pre-wet water and virgin aggregate, preparing and conditioning of the test specimens, mix design and quality assurance by testing the mechanical and durability properties. The design is a performance related design. Expected performance requirements on roads with ADT_{total} > 500 vehicles are shown in Table 2-8.

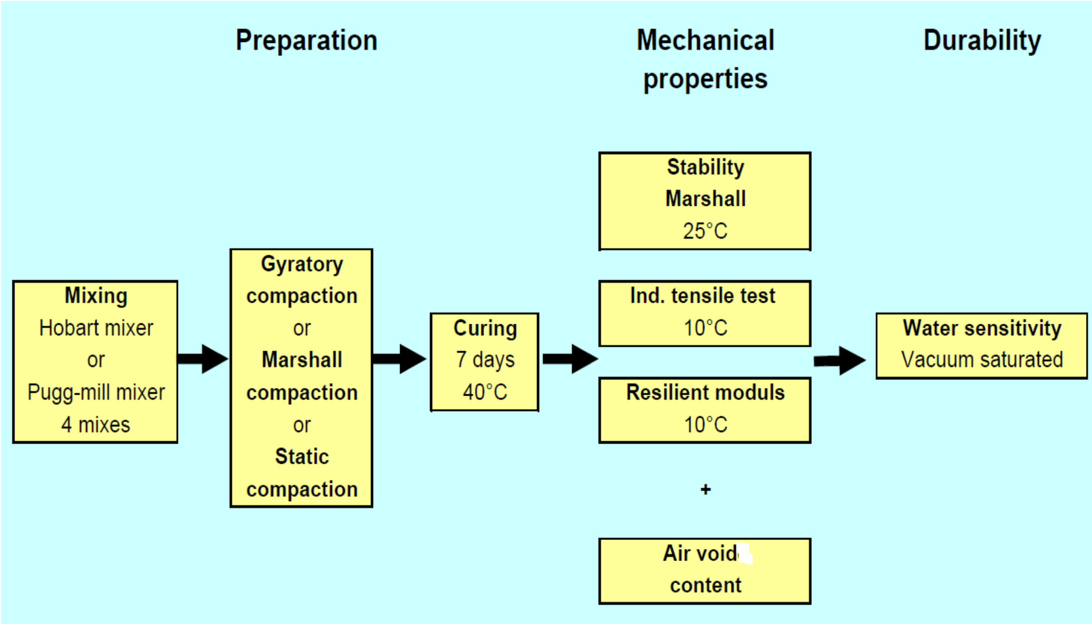


Figure 2-13: Mix Design Procedure for Cold Mix in Sweden (Jacobson, 2002)

Method	**Road-base**	**Wearing course**
Void content, % by vol.	6-14	4-12
Stability according to Marshall at 25°C, kN	>7	>5
Stiffness modulus at 10°C, MPa	>2000	-
Indirect tensile strength at 10°C, dry samples 7 days, kPa	-	>300
Water sensitivity, %, three samples	>50	>60

Table 2-8: Performance requirements for Cold Mixtures in Sweden (Jacobson, 2002)

2.9.3 Spain

Cold recycled bituminous emulsion mixtures are used in Spain comprising of either gravel or open graded mixtures. The open graded mixtures used typically comprise of 20% voids which provide mixtures that are resistant to fatigue cracking, deformation with improved skid resistance due to the interlocking large aggregate skeleton, surface friction and sufficient drainage of the mixture.

High viscosity, medium setting, cationic or anionic emulsions are used in open graded mixtures to provide a thick bitumen coating of the aggregate. They are used for wearing or binder courses, base courses and for repair of potholes on low volume roads without modified binders. The use of polymer modified binders enhances the performance properties of cold recycled bituminous mixtures and can be used for higher volume roads as stated by Needham (1996).

2.9.4 South Africa

Sabita (South African Bitumen Association) is driving the use of bitumen emulsion for road base construction in South Africa having recognised the potential of this approach publishing various design guidelines and standards as shown in Sabita (1993) and (1999) with Asphalt Academy (2002) providing further details. Evaluation work using a Heavy Vehicle Simulator (HVS) monitored and tested the performance of emulsion based mixtures. The results showed that the material was a viable option for use and the addition of cement or lime improved the strength, durability and resistance to cracking of the mixtures. The mixture design is similar to other design methods and aims to ascertain the optimum moisture and bitumen emulsion contents. Laboratory curing methods for bitumen emulsion treated materials suggest curing in the mould for 24 hours at ambient temperature followed by 48 hours curing at 40°C (if optimum moisture content is less than 8%) or 45 hours at 60°C (if optimum moisture content is greater than 8%). An alternative curing is proposed where the specimens are cured at ambient temperatures for 7 days when cement is added or for 28 days at ambient temperature when no cement has been added although concerns with simulating field conditions have been raised.

2.9.5 USA

The design for cold recycled bituminous emulsion mixtures are stated in Asphalt Institute MS-14, Asphalt Cold Mix Manual and MS-19, A Basic Asphalt Emulsion Manual.

Three approaches exist for designing cold recycled bituminous mixtures that include: (1) Assume that the RAP acts as 'black rock'. (2) Assume the complete softening of the residual binder. It is then imperative to evaluate the physical and chemical characteristics of the RAP taking into account the physical and rheological properties of the residual binder. Based on the results, recycling agents are added in order to restore the residual binder to its original consistency. (3) The third approach is a combination of (1) and (2) where it is assumed that some softening of the residual binder occurs. The degree of softening is related to the properties of the residual binder, bitumen emulsion and environmental conditions. Mechanical testing of the cold recycled bituminous mixtures form part of all mix designs to meet set performance criteria.

Most Highway Agencies that use cold recycled bituminous mixtures have their own mix design procedures which range from using empirical formulas based on the amount and consistency of the recovered binder to predict the initial bitumen emulsion content to sophisticated testing such as resilient modulus, moisture susceptibility and stability tests. The different testing methods usually consist of all or a portion of the following steps stated in Figure 2-14.

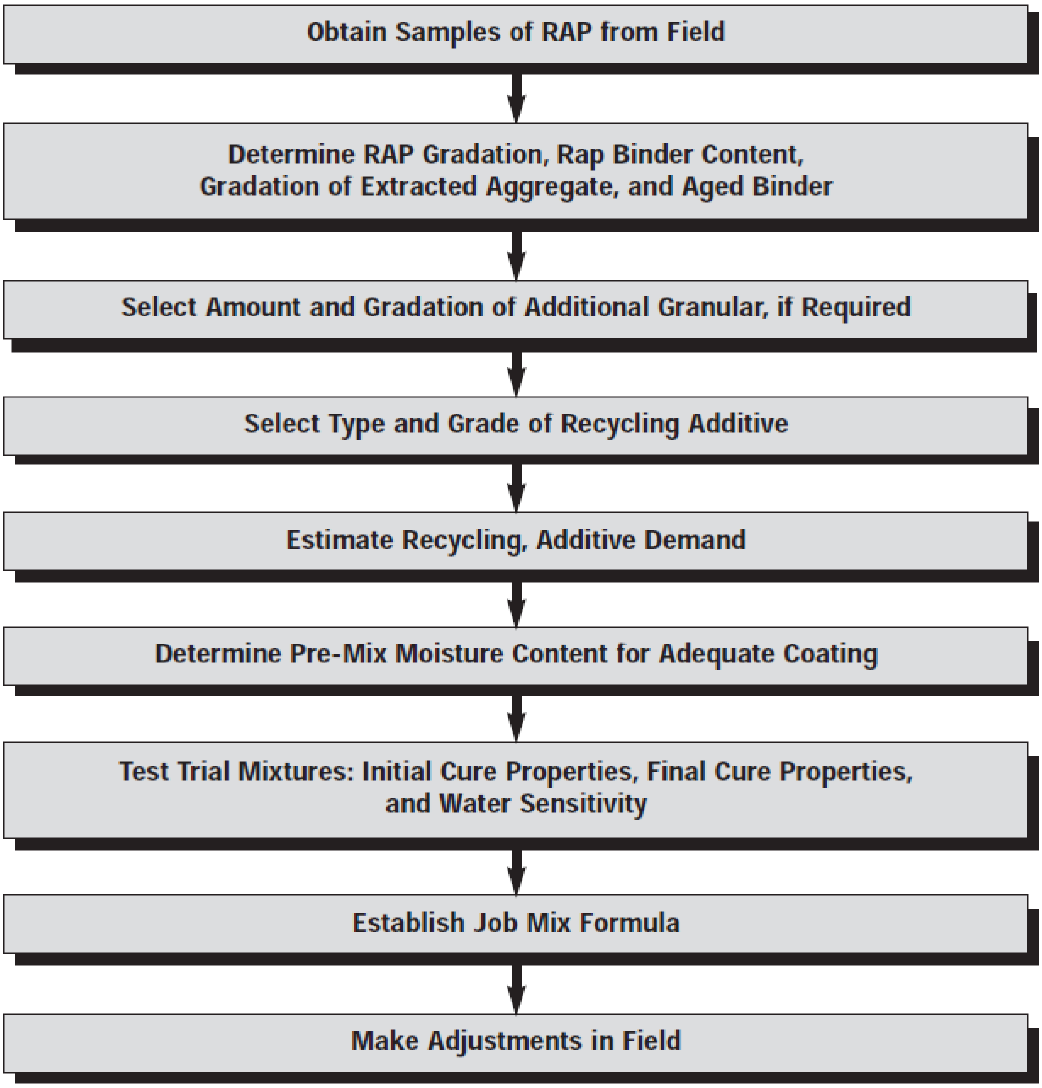


Figure 2-14: Design Flow Chart

2.10 Summary

Cold Asphalt Emulsion Mixtures (CAEMs) are economical, environmentally friendly and sustainable alternatives to hot mix asphalts. One advantage of using bitumen emulsions is that it is liquid at ambient temperatures and can be mixed with aggregates without the need to heat the stone and the bitumen as is the case with hot mix asphalts.

There is no universally accepted method for cold mix designs. Based on observations from previous research works, about 5% (by mass) of bitumen emulsion is usually added to the mix. If the existing pavement consists of a thick layer of asphalt, the amount of bitumen emulsion is between 3% - 4% depending on the proportion of asphalt in the mix (Lewis and Collings, 1999). It is advisable to conduct an appropriate mix design in order to determine the optimum bitumen emulsion content and pre-wet water content for the CAEMs. An adequate mixing and compaction procedure is essential for cold mixes. The laboratory curing protocol used should be simulative and obtainable in the field. Stiffness and strength of CAEMs develop as the emulsion sets. This results in low early life stiffness and strength in the mixtures. It is common practice to include cement as it is effective in improving the strength and adhesive properties of the layer with CAEMs. The percentage of cement that is added depends on the mix design and varies from one road to another and for different countries/regions. 1% cement is widely used. The major drawbacks of using bitumen emulsion include the fact that bitumen emulsion is not normally produced on site, emulsifying agents can be expensive, production requires strict quality control, curing can take a long time as strength development is dictated by moisture loss. Stiffness, fatigue characterisation, resistance to permanent deformation properties, resilient modulus and water susceptibility are good means of assessing the performance of cold mixes. Structural design and analysis of pavements is vital in order to ascertain the stresses, strains and deflections in the pavement structure in response to traffic loadings so as to estimate design life of the pavement.

3. MIX DESIGN OF COLD ASPHALT EMULSION MIXTURES

3.1 Overview

The use of Cold Asphalt Emulsion Mixtures (CAEMs) incorporating Reclaimed Asphalt Pavements (RAP) in the construction and rehabilitation of roads has seen an increase globally. This has resulted in the need for sound standards and regulations to be established for the laboratory mix design procedures to closely reflect conditions as experienced in the field. The mix designs currently available for CAEMs are in some cases not fully developed and not universally accepted. This is as a result of the complex nature of CAEMs acting as unbound granular materials in early life due to the presence of moisture in the mix in the uncured stage and as bound materials after the evaporation of moisture following curing.

This chapter examines and evaluates some of the distinctive attributes and qualities of CAEMs with an analysis of key features that should be investigated and taken into account in the laboratory mix design of CAEMs. This mix design chapter reports on the results of an initial study to investigate the properties of the materials used in the research. The procedures and outcomes of the laboratory trial works and pilot scale experiments conducted are presented with the view of optimising the volumetric characteristics and also, the mechanical and performance properties of the various categories of CAEMs used in the research. The overall objective is to produce CAEMs that are durable with suitably good mechanical and performance properties.

The major aims of the chapter are to:

(1) Investigate the process involved in producing CAEMs: material characterisation, evaluation of mix design procedures including the volumetric properties, compaction and curing characteristics of the mixtures.

(2) Develop a consistent mix design procedure.

(3) Conduct mechanical and performance tests to understand the behaviour and assess the properties of CAEMs in order to produce mixtures with optimum performance.

3.2 Materials Used for the Investigation

The materials used for the research are typical of materials used in the actual recycling of asphalt pavements for road applications. The materials were appropriately sourced taking into account structural requirements, durability, economics, workability and previous experience.

The materials used for work in the research include:

- Reclaimed Asphalt Pavement (RAP)
- Virgin Aggregate comprising Limestone Aggregate and Sharp Sand
- Bitumen Emulsion
- Cement
- Water

3.2.1 Reclaimed Asphalt Pavement (RAP)

The advantage of using RAP in mixtures include considerable cost savings, energy savings and environmental benefits that arise as a result of a reduction in the disposal of waste road planings from road maintenance and rehabilitation projects. The RAP aggregate materials used for the research were supplied by Lafarge Aggregate Limited and obtained from their Elstow Asphalt Plant in Bedfordshire. The physical properties of the RAP were assessed in terms of aggregate gradation, flakiness index, particle density and water absorption. The RAP as obtained from the Elstow Asphalt Plant is shown in Figure 3-1

Figure 3-1: RAP as obtained from the Quarry

The RAP aggregate material as obtained from the Elstow Asphalt Plant was air dried at room temperature in the laboratory at about 20±5°C for 24 hours and then placed in a thermostatically controlled oven at a temperature of 40°C for 24 hours in order to have a consistent RAP aggregate material that is dry, uniform and homogenous.

The moisture content of the RAP after drying was obtained as 2.01%. This was measured by successive weighing of the aggregate material before and after drying of the aggregate material. To further characterise the RAP aggregate materials, aggregate gradation was carried out. The result of the gradation analysis of the RAP aggregate material as obtained from the quarry in accordance with BS EN 933-2:2012 is stated in Table 3-1.

Aggregate Gradation of the RAP	
Sieve Size (mm)	Percent Passing (%)
31.5	100
20.0	99.47
14.0	81.92
10	59.25
6.3	34.27
4.0	21.54
2.8	17.00
2.0	13.49
1.0	9.01
0.500	5.16
0.250	1.31
0.125	0.42
0.063	0.17
Particle Density (Mg/m³)	
Particle size 31.5mm – 4mm	
▪ Oven dried	2.42
▪ Saturated and surface dried	2.47
▪ Apparent particle density	2.54
Particle size 4mm – 0.063mm	
▪ Oven dried	2.57
▪ Saturated and surface dried	2.67
▪ Apparent particle density	2.86
Water Absorption (%)	
Particle size 31.5mm – 4mm	1.86
Particle size 4mm – 0.063mm	3.99
Flakiness Index	11
Elongation Index	13
Shape Index	9

Table 3-1: Aggregate Gradation and Physical Properties of RAP

In order to further ascertain the properties of the RAP, a composition analysis was conducted. The binder in the RAP was separated using a fractionating column in accordance with BS 598-102:2003 and BS EN 933-1:2012. The properties of the extracted binder including the viscosity, softening point and penetration index were obtained. The results of the composition analysis and the gradation of the RAP prior to and after binder extraction are stated in Table 3-2 and Figure 3-2 respectively. The residual binder content of the RAP was obtained as 5.5%. The penetration and softening point values of the residual binder after extraction were 20dmm and 64.2°C respectively, in accordance with BS EN 1426:2007 and BS EN 1427:2007 respectively. The properties of the residual binder in the RAP fall within acceptable limits as it has been experimentally proven that only RAP with residual binder less than 5dmm acts as "black rock" providing no valuable mechanical and performance properties to the CAEMs as shown by (Ojum and Thom, 2009).

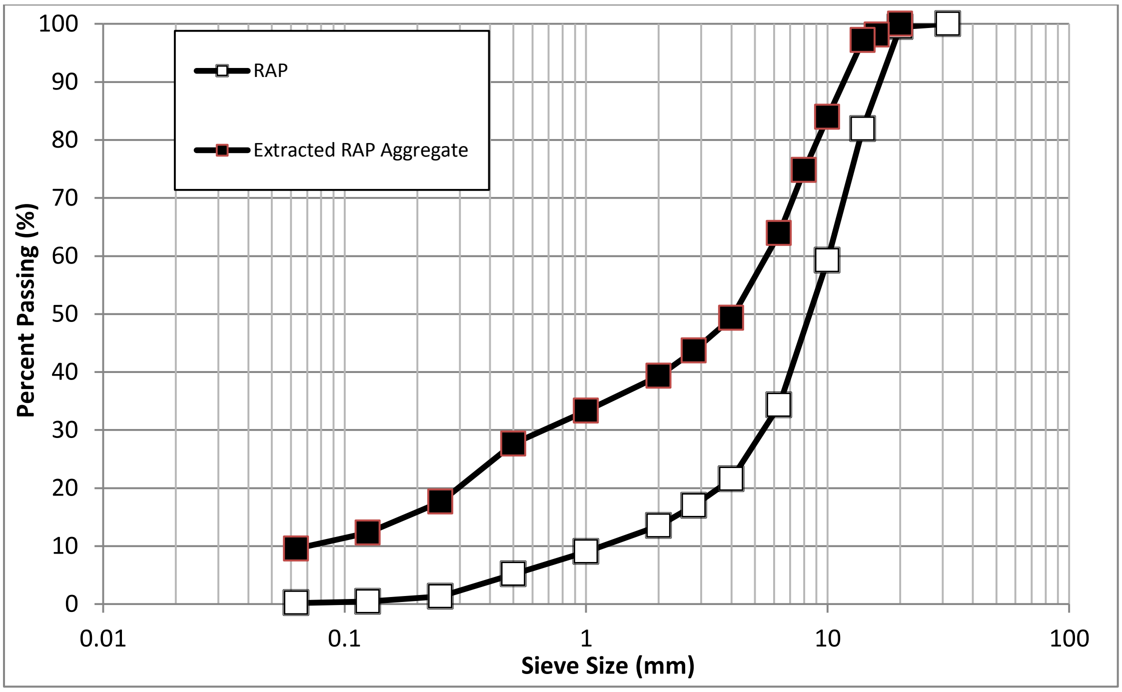


Figure 3-2: Aggregate gradations of RAP and residual RAP after binder extraction

Properties of Recovered Binder from RAP Aggregate Material	
Binder Content in RAP (%)	5.5
Penetration (25°C) (0.1mm)	20
Softening Point (Ring and Ball) (°C)	64.2
Viscosity at 135°C (mPa.s)	1077
Sieve Analysis	
Sieve (mm)	Percent Passing (%)
20	100
16	98.13
14	97.17
10	83.92
8	74.78
6.3	63.90
4	49.29
2.80	43.68
2.00	39.30
1.00	33.28
0.500	27.62
0.250	17.61
0.125	12.31
0.063	9.53

Table 3-2: Composition Analysis of RAP

Further to this, to ascertain if the bitumen in the RAP could be classified as “active” or “inactive”, an indicative cohesion test was conducted currently under investigation by the International Union of Laboratories and Experts in Construction Materials, Systems and Structures (RILEM), which involved conditioning a sample of RAP for 4 hours at 70°C followed by the manufacture of three 100mm diameter by 63.5mm high specimens using Marshall Compaction with 50 blows per face.

After compaction, Indirect Tensile Strength (ITS) tests in accordance with BS EN 12697-23:2003 were carried out at 20°C and then in wet conditions, soaked at 20°C for 24 hours. If the soaked ITS ≤ 100kPa or the specimens do not hold together at 70°C, the residual binder in the RAP is considered to be inactive.

For comparison, the tests were also conducted in dry conditions and with RAP conditioned at 140°C. In all cases, the values exceeded 100kPa indicating that the binder in the RAP used in this study can be classified as "active". The results are presented in Figure 3-3.

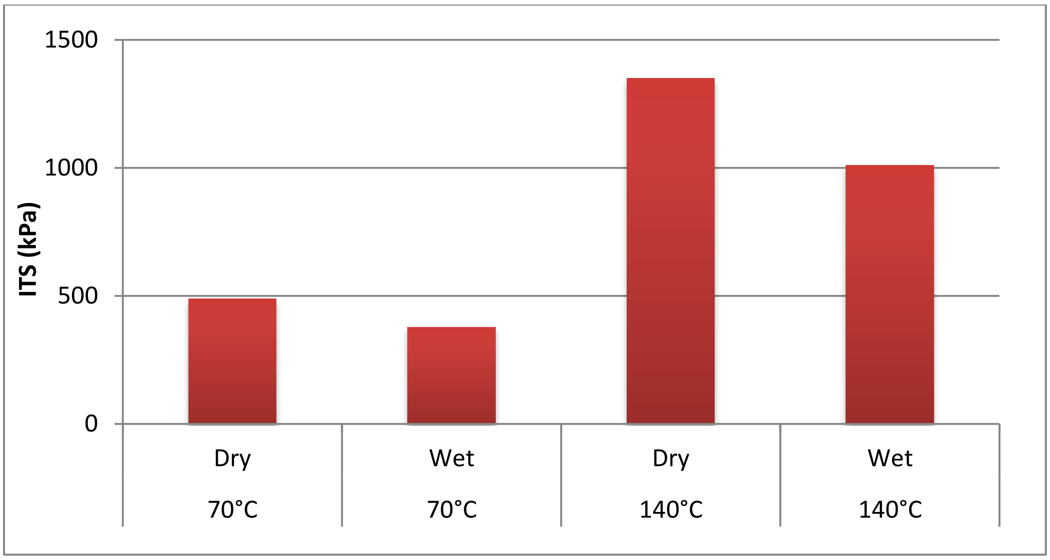


Figure 3-3: Test to ascertain the "active" or "inactive" state of binder in RAP

3.2.2 Virgin Aggregates

The limestone aggregate was obtained from Dene quarry in Derbyshire while the sharp sand used as fines for the mixture was supplied from the Elstow Asphalt Plant, both operated by Lafarge Aggregates Limited.

The limestone aggregate material as supplied consisted of the following nominal sizes: 20mm, 14mm, 10mm, 6mm, dust and filler. Figure 3-4 and Table 3-3 show the gradation and the physical properties of the virgin limestone aggregate materials and sharp sand used in the research project.

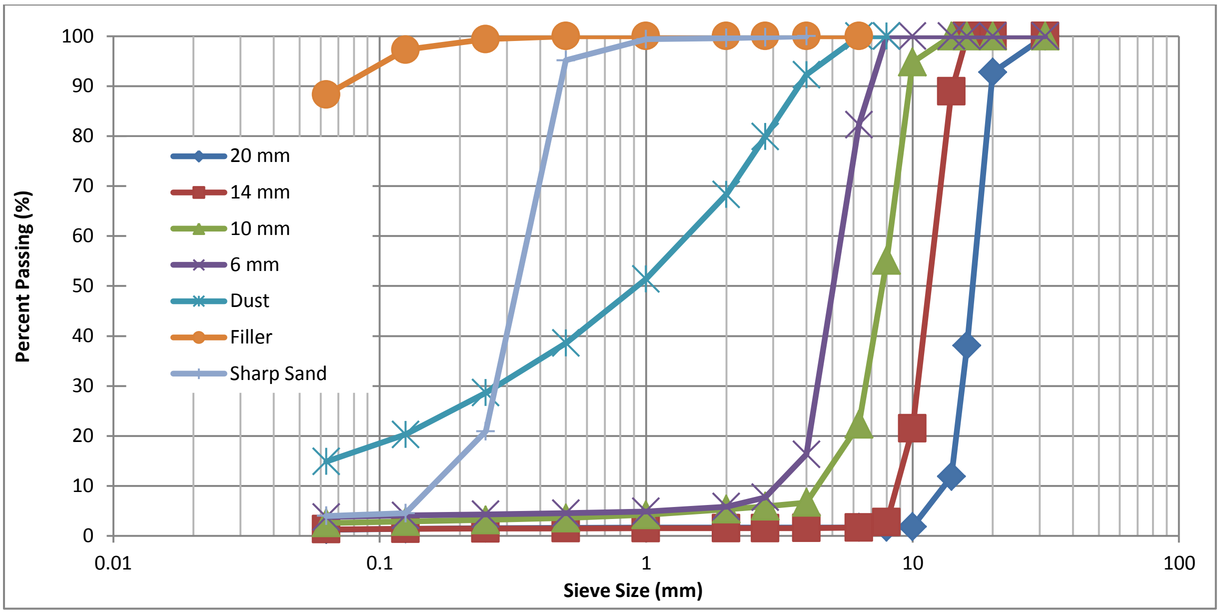


Figure 3-4: Aggregate Gradation of the Nominal Sizes of the Virgin Aggregate Materials

The addition of filler is imperative as initial evaluation of the aggregate showed that the RAP had minimal fines passing the 0.063mm sieve. The sharp sand was selected for use as it had suitable multi-sized grains that provided interlocking properties.

The particle size distribution was determined in accordance with BS EN 933-1:2012. The particle density, saturated surface dry density, apparent particle density and water absorption were determined in accordance with standards as stated in BS EN 1097-6:2000 and BS EN 1097-7:2008 for the filler particles. The particle density of the virgin aggregate was much higher in comparison to the RAP.

The RAP on saturation could absorb more water due to its structure and composition. The coating provided by the residual binder retains more of the absorbed water in comparison to the virgin aggregate.

Sieve Size (mm)	20mm	14mm	10mm	6.3mm	Dust	Filler
31.5	100	100	100	100	100	100
20	92.76	100	100	100	100	100
16	38.05	100	100	100	100	100
14	11.86	88.94	100	100	100	100
10	1.83	21.47	94.81	100	100	100
8	1.69	2.72	55.13	100	100	100
6.3	1.68	1.66	22.31	82.30	100	100
4	1.67	1.55	6.67	16.32	92.3	100
2.8	1.67	1.53	5.92	7.7	79.9	100
2.0	1.65	1.53	5.34	5.84	68.3	100
1.0	1.66	1.52	4.22	4.87	51.4	100
0.500	1.61	1.50	3.57	4.53	38.6	100
0.250	1.53	1.46	3.19	4.3	28.6	99.40
0.125	1.40	1.40	2.86	4.09	20.3	97.30
0.063	1.23	1.29	2.57	3.75	14.9	88.30
Particle density (Mg/m³)						
Oven dried	2.63	2.60	2.61	2.43	2.67	2.62
Saturated and Surface dried	2.65	2.63	2.64	2.53	2.67	N/A
Apparent	2.69	2.68	2.70	2.70	2.69	N/A
Water Absorption (%)	0.74	1.03	1.20	1.97	0.20	N/A

Table 3-3: Aggregate Gradation and Physical Properties of Virgin Limestone Aggregate Material

3.2.3 Bitumen Emulsion

The bitumen emulsion used for this study was a cationic bitumen emulsion that has affinity to a wide range of mineral aggregates and promotes adhesion of bitumen to a wide range of aggregate materials which is effective for use in all weather conditions Nynas (2010). It is called Nymuls CP 50 supplied by Nynas Bitumen.

The properties of the bitumen emulsion were characterised to ensure compatibility with the aggregate materials used in the research. Initial coating and water resistance tests using the combination of aggregate materials for this project showed that the emulsion mix coated the aggregate completely, withstood shear forces during mixing, created a homogenous mixture and was able to withstand the washing action of water after initial trial sample mixes were produced. The bitumen emulsion was found to be very suitable for use in this research study.

The bitumen-water ratio in the emulsion was 60:40. The penetration of the binder in the bitumen emulsion was found to be 47dmm and the softening point was 52°C, in line with procedures as stated in BS EN1426:2007 and BS EN1427:2007 respectively. The density of the bitumen emulsion was obtained as 1.016g/cm^3 in accordance with BS EN 15326:2007.

The bitumen emulsion mixture was kept in air-tight sealed containers stored at room temperature. The bitumen emulsions were stirred into a homogenous state before application in making the CAEMs.

3.2.4 Cement

The cement was manufactured by CEMEX UK and conforms to BS EN 197-1:2000 for CEM II/ A-L 32.5 R. The addition of 1% cement is common practice for cold mix asphalt (Wirtgen, 2010). Cement improves the mechanical properties of CAEMs. Needham and Brown (2000) showed that OPC acts as a secondary binder in the cold mix which was corroborated by Thanaya et al., (2009) presenting results showing that cement contributes to the reduction in water susceptibility of CAEMs and aids in binding the particles together leading to an increase in strength gain which is vital especially at the early life stages of CAEMs.

3.3 Experimental Plan

The Manual of Contract Documents for Highway Works (MCHW) Volume 1 Series 900 Clause 948 (Highways Agency, 2008) gives general guidance on preparing the mixtures for this research. In achieving the proposed targets, an emulsion design based on MCHW Volume 1 Series 900 Clause 948 was used. The clause stipulates the process involved in producing mixtures from graded aggregates processed using RAP and similar sources blended with other aggregates if necessary and bound with cementitious, hydraulic or bituminous binders separately or in combination. The clause covers four generic material categories as defined below in Table 3-4. The document gives information on how aggregate materials are to be sourced, processed, the type of binder and other constituent materials to be used, quality conformation, the minimum binder contents, the gradation limits, mixture design validation, compaction, conditioning and testing of the produced mixtures.

Material Category	Symbol	Description
1. Quick Hydraulic	QH	Portland Cement is the main component and bituminous binders are excluded in this category.
2. Slow Hydraulic	SH	Hydraulic binders such as Pulverised Fly Ash (PFA), Lime and Granulated Blast Furnace Slag (GBS) are used while bituminous binders and Portland Cement are excluded from this category.
3. Quick Visco – Elastic	QVE	Bituminous binder acts as the main component but also includes Portland Cement in the mix.
4. Slow Visco – Elastic	SVE	Bituminous binder acts as the main binder but Portland Cement is excluded from the mix.

Table 3-4: Material Classification as detailed in MCHW Series 900 Clause 948

Particle size distribution of the aggregates is one of the most important aggregate qualities. It impacts on the volumetric properties thereby affecting the stiffness, stability, durability, fatigue, permeability, workability and moisture susceptibility of produced mixtures (Roberts et al., 1996).

The aggregate gradation used in this research was based on Table 9/12 Zone C of MCHW Volume 1 Series 900 Clause 948. To further optimise the aggregate gradation, a Fuller curve was used in order to obtain the gradation that best gives the maximum density and ensures smaller particles are adequately packed together with larger particles, thereby reducing voids and creating better particle to particle contact, stability and durability for the produced mixtures. Fuller curves are based on Equation 3-1.

Equation 3-1 $$P = \left(\frac{d}{D}\right)^n \times 100$$

Where:

P = Total percentage passing the given size

d = Aggregate size being considered

D = Maximum aggregate size

n = Parameter that adjusts the curve for fineness or coarseness (for maximum particle density n ≈ 0.45)

Table 3-5 depicts the aggregate gradation limits used in the research as stated in Table 9/12 Zone C of MCHW Volume 1 Series 900 Clause 948 and targeting the 20mm Fuller curve.

Sieve Size (mm)	Lower Limits (mm)	Midpoint (mm)	Upper Limits (mm)	20mm Fuller Curve (mm)
40	100	100	100	100
31.5	86	93	100	100
20	65	82.5	100	100
14	52	76	100	85
10	44	72	100	73
4	26	50	74	48
2	18	38	58	35
0.5	8	23	38	19
0.25	5	16.5	28	14
0.063	3	12	21	7

Table 3-5: Aggregate Gradation

This research studied the effect of RAP addition to CAEMs with RAP content as high as 95% in order to understand the effect on the volumetric, mechanical and performance properties and how they realistically affect pavement mechanical behaviour with the view of developing techniques to obtain long lasting and sustainable CAEMs.

To achieve the objectives of the research, four proportions of RAP aggregate materials were used as categorised below:

- Category 1: 0% RAP (no RAP, 100% VA)
- Category 2: 50% RAP (50% RAP, 50% VA)
- Category 3: 85% RAP (85% RAP, 15% VA)
- Category 4: 95% RAP (95% RAP, 5% VA)

The influence of temperature when mixing, placing and compacting on the performance of CAEMs was investigated in this study to simulate typical winter, moderate and tropical/summer temperature conditions. How this affects the material properties including the ease of mixing, coating and compaction has been examined. Further to this, detailed mechanical and performance tests were conducted in order to understand best practices for mixing, placing and compaction of CAEMs. The mixing temperatures investigated were:

- Temperature 1 at 5°C simulating typical winter conditions.
- Temperature 2 at 20°C simulating moderate temperate conditions.
- Temperature 3 at 32°C simulating tropical conditions.

Combinations of hydraulic and bituminous binders are often used as stabilizing agents. Cement or hydrated lime can be added as active filler as they improve the properties of the CAEMs especially at early life, providing strength at this critical stage. This study examined and investigated the influence of various dosage levels of cement and how it affects the mechanical and performance properties of CAEMs especially with increasing RAP contents. The study also evaluated the influence of cement dosage on the mixing and compaction of CAEMs at different temperatures. This is important as it can have an impact on traffic opening time and long term performance. Some of the CAEMs investigated in this research are Slow Visco-Elastic (SVE) with no cement while others are Quick Visco-Elastic (QVE) as defined in Table 3-4. It should be noted that only Ordinary Portland Cement (OPC) was used in this project.

Cement addition levels used for this research were:

- 0% Cement
- 1% Cement
- 3% Cement

3.4 Aggregate Gradation for the Mixtures

The aggregate gradations for Categories 1-4 as defined above closely targeted the 20mm Fuller Curve and are stated below in Table 3-6. Figure 3-5 depicts the gradations graphically while Table 3-7 shows the individual aggregate particle size contributions for the various categories used in achieving the target gradation.

Sieve Size (mm)	Percent Passing (%)			
	Category 1	Category 2	Category 3	Category 4
31.5	100	100	100	100
20	98.9	100	99.9	100
16	90.7	100	99.4	100
14	84.9	84.8	89.1	84.4
10	71.9	73.0	73.0	71.5
8	68.7	69.6	72.6	71.5
6.3	64.5	61.9	58.5	52.9
4	46.7	46.8	47.9	46.6
2.8	40.6	42.4	43.1	42.4
2	36.4	37.5	37.3	35.8
1	30.5	30.0	28.2	25.4
0.5	25.7	23.1	20.0	16.4
0.25	14.2	11.2	10.8	10.0
0.125	9.6	8.1	8.1	7.9
0.063	7.6	7.5	7.5	7.3

Table 3-6: Particle Size Distribution for Categories 1 – 4

Recycled bituminous bound materials are easily affected by the aggregate gradation especially the fines component of the mix as the bitumen tends to encapsulate the fines while partially coating the coarse aggregates as stated by (Jenkins, 2000). Ruckel et al., (1982) suggested that the amount passing the 0.075mm sieve should be between 5 and 20%. This was achieved in this study with all categories within this limit.

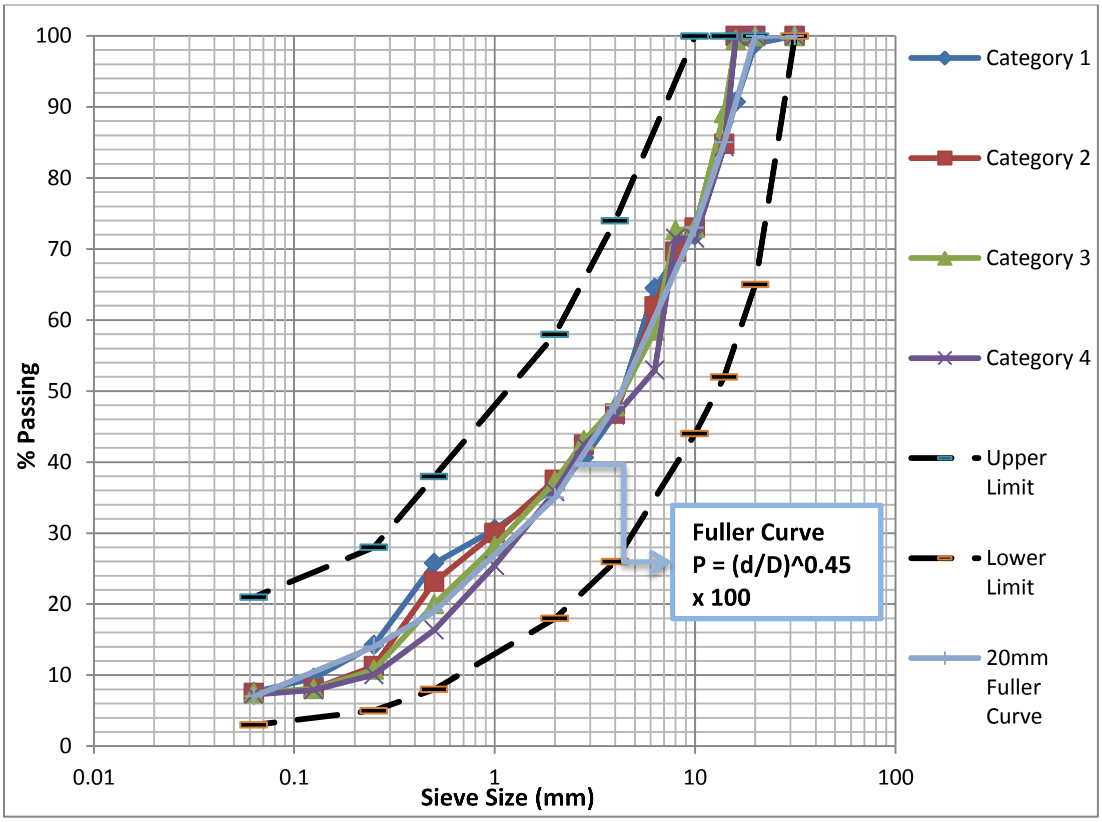


Figure 3-5: Aggregate Gradations for Categories 1 – 4

Aggregate Size (mm)	Percent Weight (%)			
	Category 1	Category 2	Category 3	Category 4
20mm	15	-	1	-
14mm	17	14	2	-
14mm RAP	-	14	10	16
10mm	-	2	-	-
10mm RAP	-	2	15	13
6.3mm	23	18	-	-
6.3mm RAP	-	4	15	20
4mm RAP	-	2	10	4
Sharp Sand	11	10	5	-
Dust	33	-	-	-
Dust (RAP)	-	28	35	40
Filler	1	6	7	7

Table 3-7: Aggregate Particle Size Contribution for Categories 1 – 4

3.5 Estimating the Initial Bitumen Emulsion Content

Currently there is no universally accepted mix design method for CAEMs although there are standards, principles and procedures available and used by various researchers for mix design of cold mix asphalts. The Asphalt Institute (MS-14, 1989) provides a method to calculate an estimate for the initial binder demand for CAEMs incorporating RAP. The equation is presented below as Equation 3-2.

Equation 3-2 $$P = (0.05A + 0.1B + 0.5C) \times (0.7)$$

Where:

P = Percent by weight of initial residual bitumen content by mass of mixture

A = Percent of mineral aggregate > 2.36mm

B = Percent of mineral aggregate <2.36 and > 0.075mm

C = Percent of mineral aggregate < 0.075mm

Based on the equation and the aggregate gradations as stated in Table 3-6, the estimated binder content, P for Categories 2 – 4 are stated below in Table 3-8.

Symbols	Category 2	Category 3	Category 4
A	61.5	62.2	61.7
B	30.7	30.0	30.7
C	7.8	7.8	7.6
P	7.03	7.01	6.97

Table 3-8: Estimated Bitumen Content (MS-14, 1989)

As stated in Table 3-2, the compositional analysis conducted on the RAP highlighted the fact that the initial residual binder content in the RAP is 5.5%. Taking this into account, the RAP binder contributions for the various categories are stated in Table 3-9.

Categories	Residual Binder in RAP (%)	Binder Contribution from RAP (%)	Estimated Initial Binder Demand (%)
Category 2: 50% RAP, 50% VA	5.5	5.5×0.5=2.75	7.03 – 2.75 = 4.28
Category 3: 85% RAP, 15%VA	5.5	5.5×0.85=4.68	7.01 – 4.68 = 2.33
Category 4: 95%RAP, 5%VA	5.5	5.5×0.95=5.23	6.97 – 5.225 = 1.75

Table 3-9: Binder Contribution from RAP (MS-14, 1989)

In an emulsion mixture, the bitumen is suspended as tiny droplets in water. The estimated bitumen emulsion content needs to be ascertained. Considering the fact that the bitumen-water ratio for the Nymuls CP-50 bitumen emulsion used was obtained as 60% bitumen and 40% water, the estimated Bitumen Emulsion Content (BEC) for the various categories are stated below in Table 3-10.

Categories	Estimated Bitumen Emulsion Content (%)	Approximated Figures
Category 2	4.28 ÷ 0.6 = 7.13	7%
Category 3	2.33 ÷ 0.6 = 3.88	4%
Category 4	1.75 ÷ 0.6 = 2.92	3%

Table 3-10: Estimated Bitumen Emulsion Content

3.6 Compaction Characteristics

The modified Proctor test in accordance with BS EN 13286-2:2004 was performed to obtain the Maximum Dry Density (MDD) and Optimum Moisture Content (OMC). This gives an indication of the water content at which the mixtures can be compacted in order to achieve maximum dry density. The test gives an estimate of the mixture density that can be achieved and provides a reference for assessing the density of a compacted layer of the mixture. The modified Proctor test specifies a test method for the determination of the relationship between the water content and the dry density of hydraulically bound or unbound mixtures after compaction using proctor compaction. The water content of the materials was increased in 1% increments ranging between 3%-8%. Table 3-11 states details of the parameters used for the modified Proctor Test.

The test involves placing a set quantity of aggregate material in the mould at each water content level such that on compaction, the material occupies about one fifth of the height of the mould.

Test	Characteristics of Test	Value
Modified Proctor Test	Diameter of mould	150mm
	Mass of Rammer	4.5kg
	Diameter of Rammer	50mm
	Height of Fall	457mm
	Number of Layers	5
	Number of blows per face	56

Table 3-11: Modified Proctor Test Parameters

Results of the dry densities are detailed in Table 3-12. These details were useful in ascertaining the Optimum Moisture Content (OMC) and Maximum Dry Densities (MDD) for Categories 1 – 4 as depicted in Figure 3-6. The OMC gives an indication of the total fluid content for the mixtures but it should be noted that the mechanism of compaction involving water and bitumen emulsion is slightly different hence the need to determine the Optimum Total Fluid Content (OTFC) for the CAEMs as detailed in the next section.

Water Content (%)	Dry Densities (kg/m^3)			
	Category1	Category 2	Category 3	Category 4
3	2118	2105	2058	2039
4	2132	2120	2072	2054
5	2169	2147	2109	2081
6	2250	2180	2148	2128
7	2239	2144	2089	2040
8	2200	2109	2068	2008

Table 3-12: Dry Densities Obtained from Modified Proctor Test

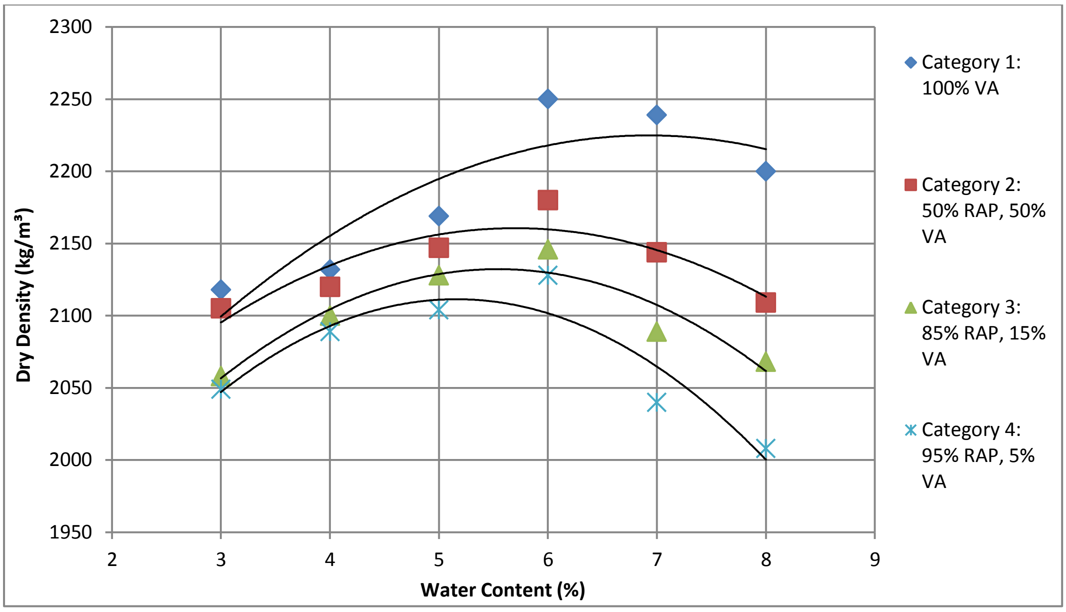


Figure 3-6: Modified Proctor Test results

Figure 3-6 shows the OMC and MDD for Categories 1 – 4 while Table 3-13 states the OMC and MDD values.

Mixture Type	Optimum Moisture Content (%)	Maximum Dry Density (kg/m^3)
Category 1	6.8	2225
Category 2	6.0	2152
Category 3	5.8	2130
Category 4	5.2	2108

Table 3-13: Optimum Moisture Contents and Maximum Dry Densities for Categories 1 – 4

3.7 Procedure for Manufacturing the Cold Asphalt Emulsion Mixtures

The procedure for preparing and manufacturing CAEMs is very important in the mix design process as this ensures that mixtures produced are consistent and homogenous. Mixing for this research study was carried out using a Sun and Planet mixer depicted in Figure 3-7. This mixer was advantageous as it was temperature controlled, which allowed investigation of the effects of mixing and compaction temperatures on the different categories of CAEMs produced.

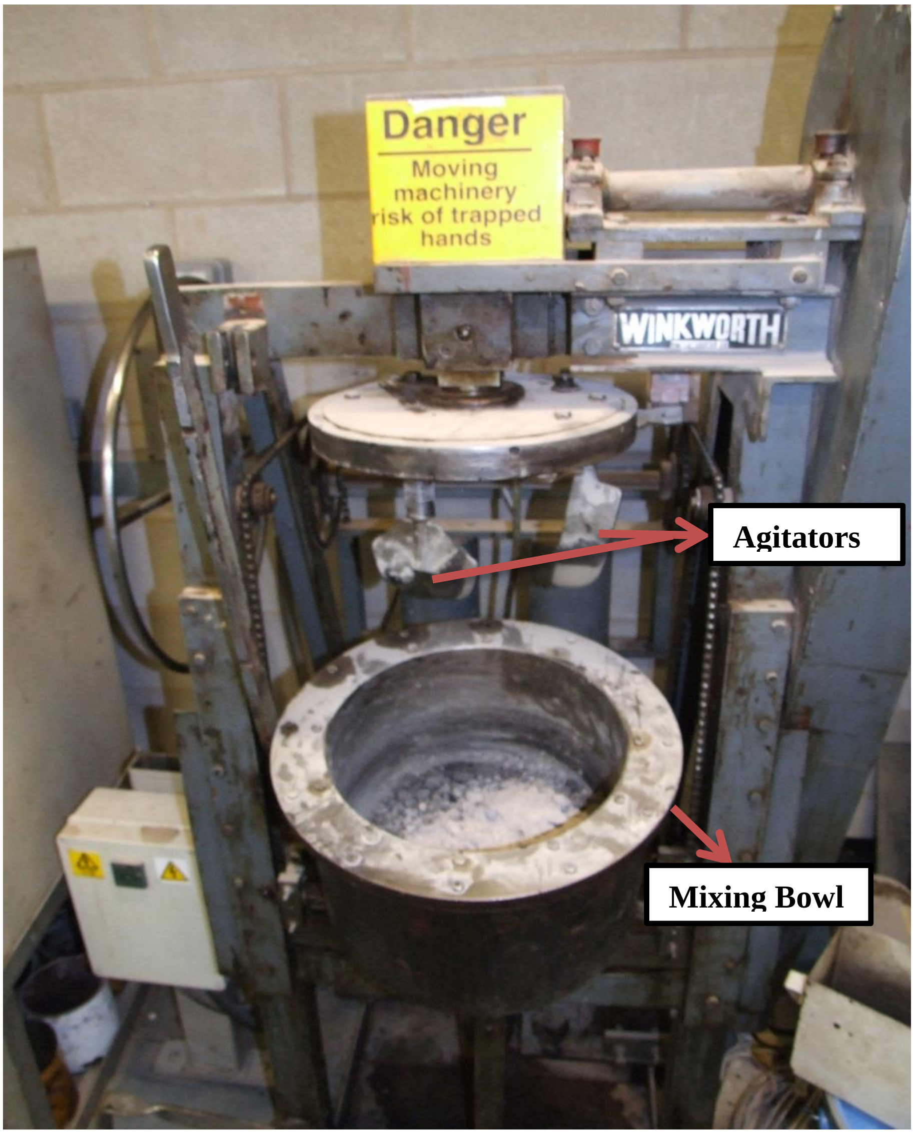


Figure 3-7: The Sun and Planet Mixer

This section details results of a study undertaken to obtain the best sequence for mixing the pre-wetting water content (PWC) and the bitumen emulsion with the aggregate materials. The pre-wetting water lubricates the aggregate materials and activates the surface charges on the aggregate materials prior to the addition of the bitumen emulsion. The pre-wetting water also helps in preventing premature breaking of the bitumen emulsion. Optimal pre-wetting water content facilitates good coating, bonding and improved properties of the CAEMs. An inadequate mixing protocol can adversely influence the compatibility and workability of the mixtures. 100mm cylindrical moulds were utilised as this provided an avenue for producing cylindrical specimens in an easy, convenient manner. Compaction was done using a Cooper gyratory compactor as seen below in Figure 3-8.

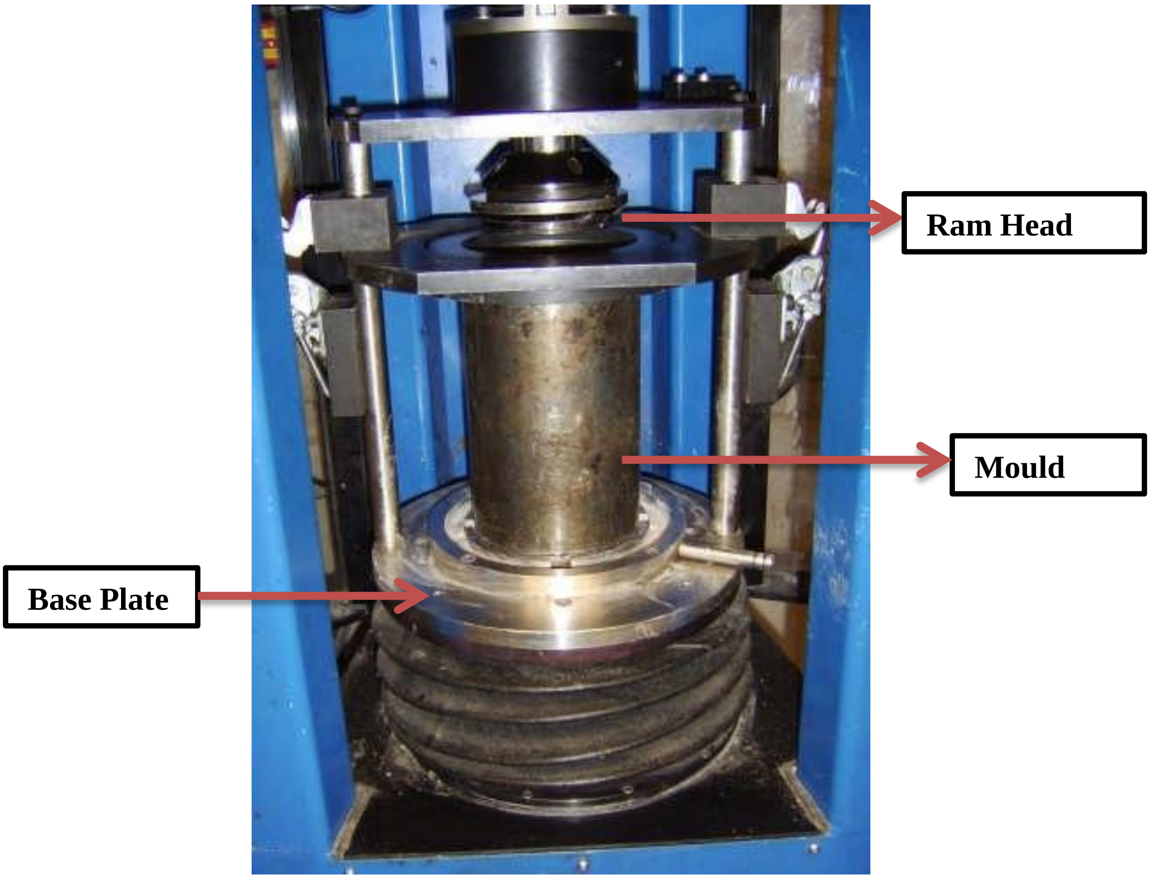


Figure 3-8: Cooper Gyratory Compactor

This compactor was well suited for producing the CAEMs used in this study. Density and compaction shear stress values are generated automatically by the compactor, providing a good basis for comparison during use. Previous studies on various modes of compaction and degree/level of compaction to simulate field conditions have positively recommended the use of the Cooper gyratory compactor. (Oke, 2010), (Sunarjono, 2008) and (Jenkins, 2000) and many other researchers recommend 50 gyrations at an angle of gyration of 1.25° and a compaction pressure of 600kPa at 30 revolutions per minute when using the gyratory compactor. After compacting, the specimens were left in the mould for 24 hours before extracting.

Curing at this stage of the study was done at 40°C for 72 hours in an unwrapped condition to accelerate the curing process. The volumetric properties of the produced CAEMs were ascertained and testing of the mixtures carried out. Mechanical and performance tests were conducted in order to understand the behaviour and properties of the CAEMs produced.

3.8 Optimising the Mixing Protocol

Three mix protocols were investigated, analysed and evaluated in order to select the best mixing protocol to give a homogenous, well coated and consistent mixture with appropriate mechanical and performance properties to be adopted for the research. This study to evaluate the mixing protocol was done prior to obtaining the Optimum Total Fluid Content (OTFC). It should be noted that Category 2: 50% RAP, 50% VA was produced for this section of the study using 7% BEC and 1% PWC.

The compaction and curing protocols highlighted in the above section was used in producing the mixtures to investigate, analyse and evaluate the mixing protocol to be adopted. Table 3-14 details the mixing protocols and results obtained from the study.

MPWC (s)	MBEC (s)	ITSM (MPa)	Air Void Content (%)	Average ITSM (MPa)	Average Air Void Content (%)
Method A					
60	60	1527	21.9	1498	21.3
60	60	1539	21.1		
60	60	1427	20.8		
Method B					
90	90	1949	19.5	1955	19.6
90	90	1971	19.6		
90	90	1946	19.8		
Method C					
60 + 30	60 + 30	2323	18.1	2221	18.2
60 + 30	60 + 30	2287	18.7		
60 + 30	60 + 30	2054	17.8		

MPWC	Mixing Time after adding the Pre – Wetting Water Content
MBEC	Mixing Time after adding the Bitumen Emulsion Content

Table 3-14: Optimising the Mixing Protocol

Method A involved mixing the PWC with the aggregates for 60s followed by adding the bitumen emulsion content and then mixing for a further 60s. Method B followed exactly the same procedure as Method A although with an increased mixing time of 90s.

The PWC was added and mixed for 90s followed by the bitumen emulsion which was then mixed for a further 90s. It was observed that specimens produced for both Method A and B were not properly coated with the bitumen emulsion after the mixing process. Mixture segregation between the aggregate and the bitumen emulsion was evident after compaction with visible patches of coated and uncoated aggregate. Although, there was a slight improvement in stiffness and void content between Method A and B, a new mixing regime was needed to ensure adequate coating of the materials.

Method C involved first placing the aggregate materials into the mixing bowl of the Sun and Planet mixer and immediately mixing for 60s before adding the PWC to ensure all aggregate materials are thoroughly mixed together. The PWC was added in a thin stream and the mixture mixed for 60s using the mixer and this was immediately followed by hand mixing using a spatula for 30s ensuring that the aggregate materials were thoroughly mixed and wetted ready for the addition of the bitumen emulsion. The bitumen emulsion was added and then mixed with the Sun and Planet mixer for 60s. To ensure homogeneity and consistency in the mix, the materials were then mixed by hand using a spatula for 30s making a total mixing time of 240s (4 minutes).

Visual inspection of the CAEMs produced using Method C in comparison to Method A and B showed that the materials were thoroughly mixed together and well coated. The issue of patches in the produced specimens was totally eliminated and this was evident in the physical appearance of the produced specimens.

Method C proved the best mix protocol as it gave the best workability, had the best stiffness results and the lowest void content as indicated by the results obtained. Table 3-14 and Figure 3-9 show the results of this analysis. Method C was subsequently adopted as the mix protocol used for these research studies.

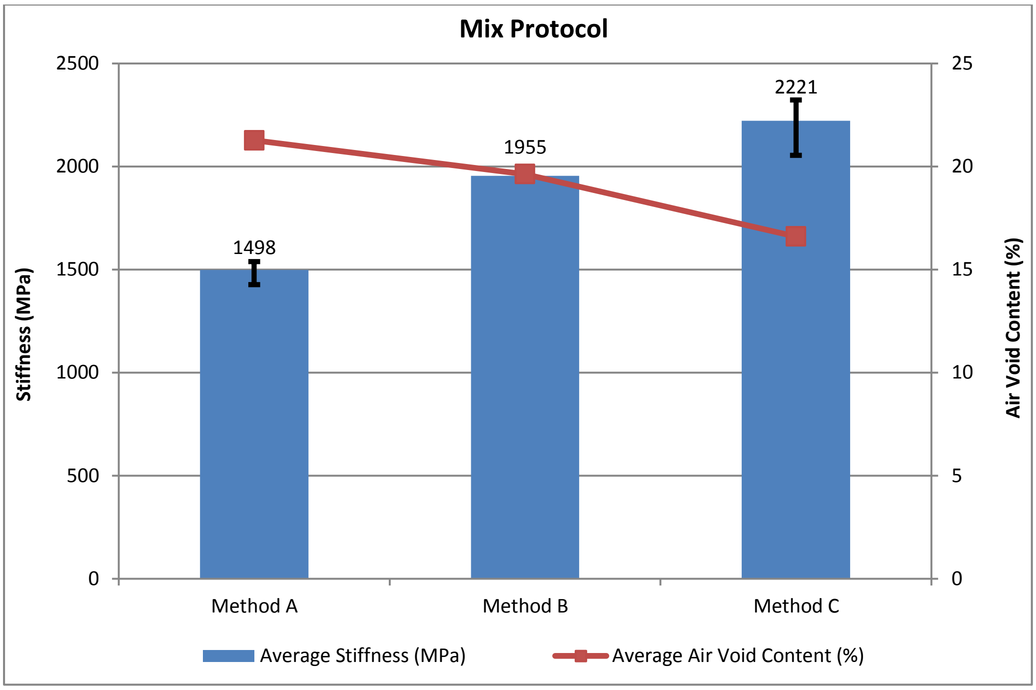


Figure 3-9: The Relationship between Mixing Protocol, Stiffness and Void Content

3.9 Obtaining the Optimum Total Fluid Content (OTFC)

The Optimum Total Fluid Content (OTFC) plays a significant role as it influences the workability and volumetric properties of the mixtures. The major purpose of the pre-wetting water content is to facilitate even distribution of the bitumen emulsion onto the surface of the aggregate for best binder coating (Thanaya, 2003). The bitumen emulsion coats and binds the aggregate materials together giving the mixture its mechanical and performance properties.

To obtain the OTFC, the estimated bitumen emulsion content obtained given in Table 3-10 is kept constant while the pre – wet water content (PWC) was varied as shown below in Table 3-15.

Category 2: 50% RAP, 50% VA	**Category 3: 85% RAP, 15% VA**
7% BEC + 1% PWC = 8% TFC	4% BEC + 1% PWC = 5% TFC
7% BEC +1.5% PWC = 8.5% TFC	4% BEC + 2% PWC = 6% TFC
7% BEC + 2% PWC = 9% TFC	4% BEC + 3% PWC = 7% TFC
7% BEC + 3% PWC = 10% TFC	4% BEC + 4% PWC = 8% TFC

Table 3-15: Bitumen Emulsion and Pre-Wetting Water Contents for OTFC

After mixing, using the optimised mixing procedure as stated above, the mix was placed in 100mm cylindrical diameter moulds and compacted immediately in the Cooper Gyratory Compactor. Compaction was done at ambient room temperature of 20°C, using a pressure of 600kPa, 50 gyrations and a rate of gyration of 30revs/minute at an angle of gyration of 1.25°.

After compaction in the gyratory compactor, the specimens were left in the mould for 24hrs at room temperature. This is to account for the delicate nature of the CAEM during its early life and to prevent damage to the specimens. After this, the specimens were extracted from the moulds and placed in a thermostatically controlled oven for accelerated curing at 40°C for 72hrs in an unsealed state.

Mechanical and performance tests namely the Indirect Tensile Stiffness Modulus (ITSM) and Indirect Tensile Strength (ITS) tests were conducted in dry and wet conditions to assess and evaluate the behaviour and properties of the CAEMs produced.

Eight cylindrical specimens were produced at each pre – wetting water content; 4 specimens were used to conduct the ITSM and ITS tests in dry conditions while the remaining specimens were tested after soaking at 40°C for 72hrs in accordance with BS EN 12697-12:2008. The ITSM and ITS tests were conducted in accordance with BS EN12697-26:2012 and BS EN 12697-23:2003 respectively. The stiffness and strength results including the ratios are depicted below in Figure 3-10 and Figure 3-11 respectively:

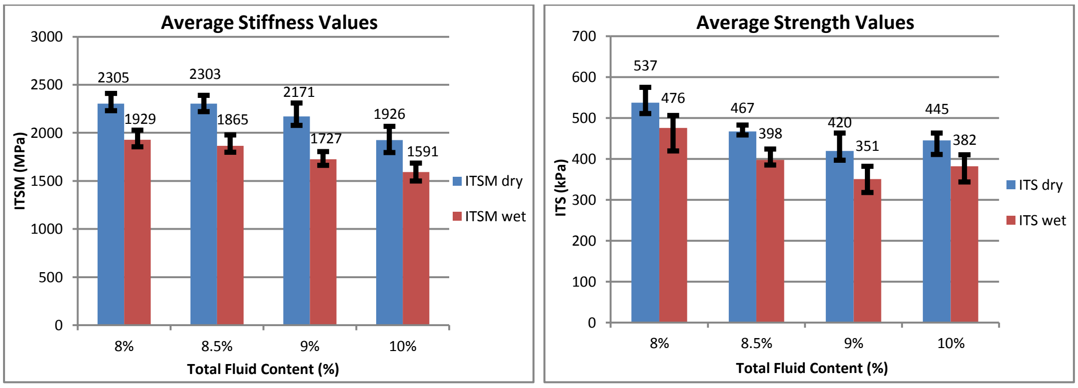


Figure 3-10: Average Strength and Stiffness Values for Category 2

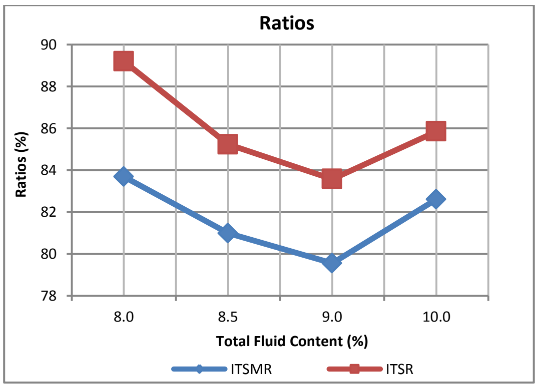


Figure 3-11: ITSMR and ITSR values for Category 2

The results show the stiffness and strength characteristics in dry and wet conditions of the CAEMs with respect to the Total Fluid Content (TFC) and the ratios of ITSM and ITS tests in dry and wet conditions, ITSMR and ITSR values are also depicted in Figure 3-11 and Figure 3-13 for Categories 2 and 3 respectively.

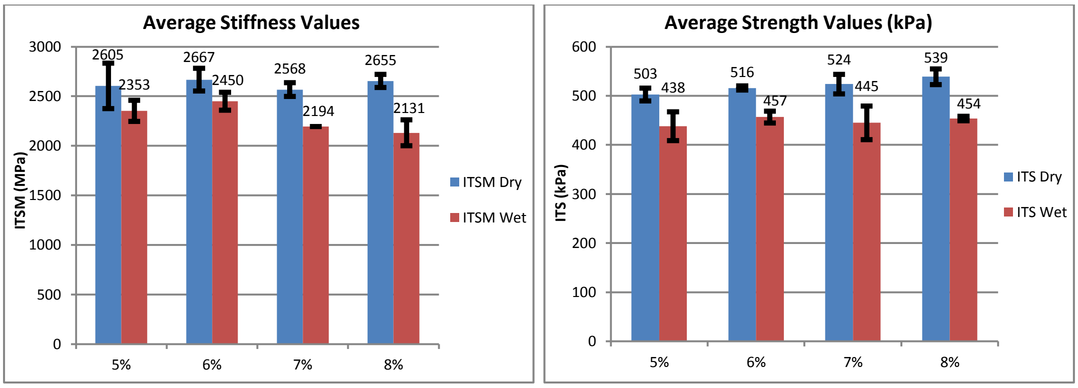


Figure 3-12: Average Strength and Stiffness Values for Category 3

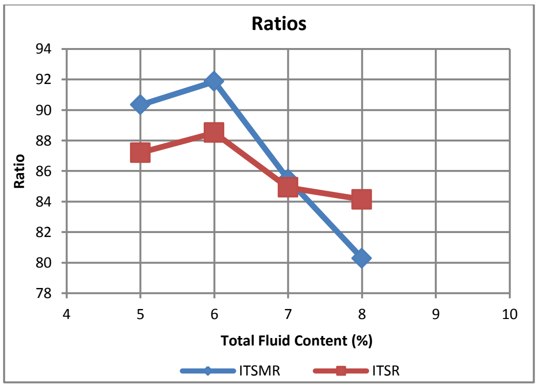


Figure 3-13: ITSMR and ITSR values for Category 3

Based on the ITSMR and ITSR values as shown in Figure 3-11 and Figure 3-13, 8% was selected as the OTFC for Category 2 and 6% for Category 3.

3.10 Obtaining the Optimum Bitumen Emulsion Content (OBEC)

In order to obtain the Optimum Bitumen Emulsion Content (OBEC), the Optimum Total Fluid Content (OTFC) obtained in the above section for Categories 2 and 3 were kept constant while the Pre-wetting Water Content (PWC) and Bitumen Emulsion Content (BEC) were varied.

This is necessary in order to optimise the binder content in the OTFC. The procedure as stated in the above section for obtaining the OTFC was followed in batching, preparation, mixing, compaction and testing of the CAEMs. Table 3-16 details the proportions of PWC and BEC evaluated.

Category 2: 50% RAP, 50% VA	**Category 3: 85% RAP, 15% VA**
4.25% BEC + 3.75% PWC = 8% TFC	2.25% BEC + 3.75% PWC = 6% TFC
5.25% BEC + 2.75% PWC = 8% TFC	3.25% BEC + 2.75% PWC = 6% TFC
6.25% BEC + 1.75% PWC = 8% TFC	4.25% BEC + 1.75% PWC = 6% TFC
7.25% BEC + 0.75% PWC = 8% TFC	5.25% BEC + 0.75% PWC = 6% TFC

Table 3-16: Bitumen Emulsion and Pre-Wetting Water Contents for OBEC

The stiffness results obtained for Category 2 as depicted in Figure 3-14 show that as the BEC increased, the stiffness in dry conditions increased slightly and peaked at 5.25% followed by a decrease in stiffness values at both 6.25% and 7.25% BEC.

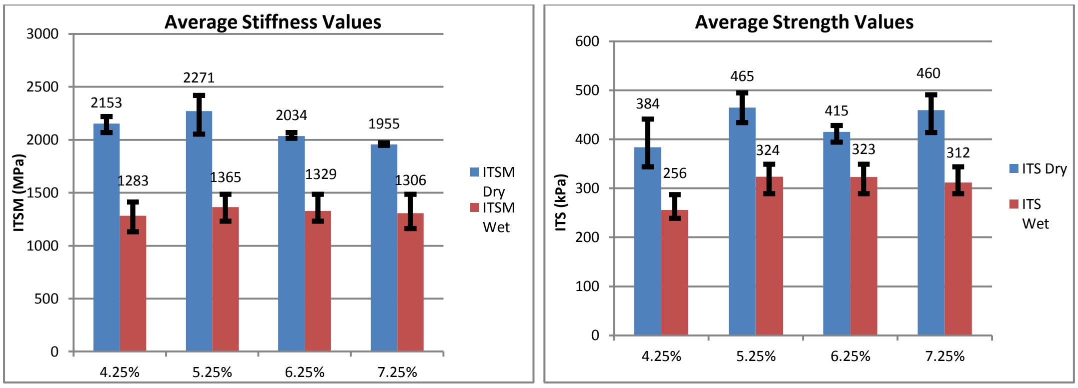


Figure 3-14: Average Strength and Stiffness Values for Category 2

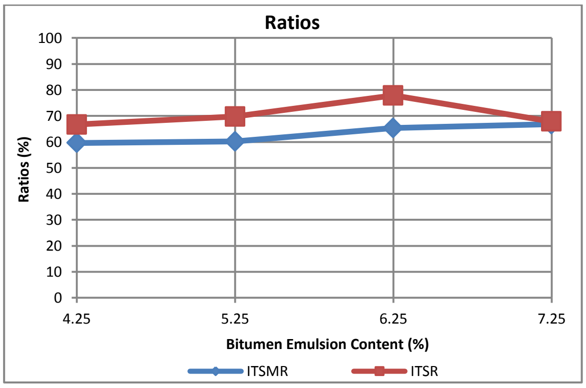


Figure 3-15: ITSMR and ITSR values for Category 2

In wet conditions, the stiffness values were generally similar and this was also observed when the ITS tests were conducted in dry conditions. The strength values in wet conditions peaked at 5.25% BEC and thereafter showed steady values. Evaluating the ratios, ITSMR and ITSR as displayed in Figure 3-15, it was evident that the ratios peaked at 6.25% BEC although the average stiffness and strength values for 5.25% BEC were better in comparison to the other BEC. After due consideration and evaluation of the results obtained, 5.25% BEC (and 2.75% PWC) was adopted as the OBEC and PWC for Category 2.

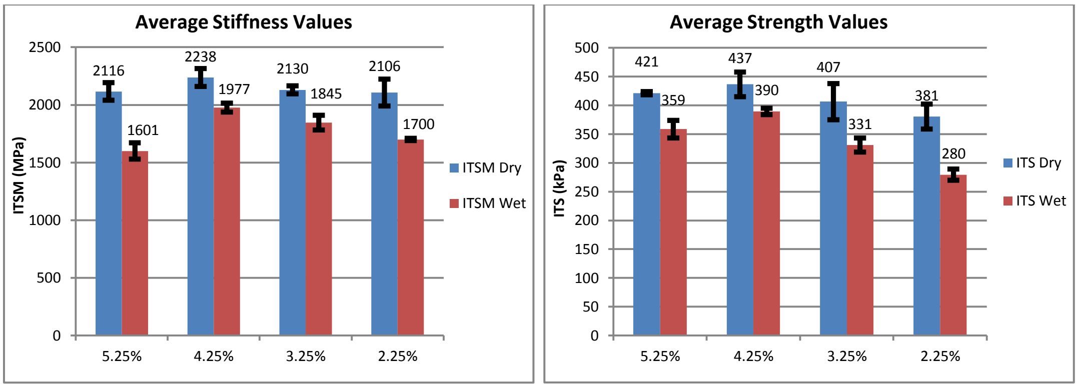


Figure 3-16: Average Strength and Stiffness Values for Category 3

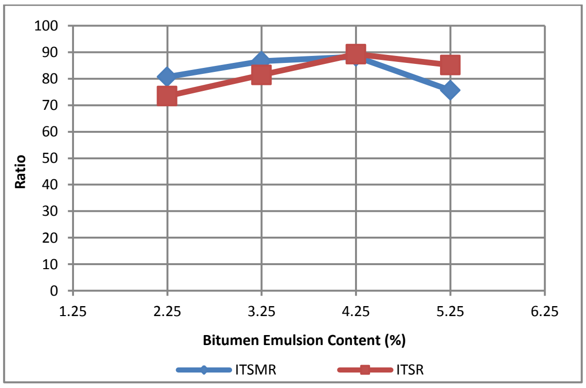


Figure 3-17: ITSMR and ITSR values for Category 3

Figure 3-17 and Figure 3-16 presents the results obtained for Category 3. As the BEC increased, the stiffness and strength values in both dry and wet conditions increased slightly and peaked at 4.25% BEC followed by a decrease in stiffness and strength values at 5.25% BEC. Evaluating the ratios, as shown in Figure 3-17, it was evident that the ratios also peaked at 4.25% BE. With respect to this, 4.25% BEC (and 1.75% PWC) was adopted as the OBEC and PWC for Category 3.

It should be noted that in conducting the mix design for this study, detailed investigations was carried out for Categories 2 and 3. The OBEC and PWC for Category 4 were extrapolated from results and trends obtained from the mix design studies conducted for Categories 2 and 3.

The existing trend shows that since the residual binder content in the RAP is active and positively contributing to the mechanical and performance properties of the CAEMs, an increase in the RAP content will result in a decrease in the OBEC. This resulted in estimating that a sufficient level of OBEC and PWC for Category 4 will be 3.75% BEC and (1.75% PWC). This was adopted as the OBEC and PWC for Category 4.

The OBEC and PWC for Category 1 were taken as 6.5% BEC and 1.5% PWC. This was adequate based on extensive investigations and previous research carried out by (Ojum, 2009) and (Oke, 2010). The study focused on using CAEMs to investigate the effects of residual binder in RAP. An extensive mix design was conducted on CAEMs produced using virgin aggregates in these studies and it was found that OBEC and PWC of 6.5% BEC and 1.5% PWC will produce optimally performing virgin aggregate CAEMs. This was subsequently adopted and used for Category 1.

3.11 Optimising the Compaction Characteristics of the CAEMs

After obtaining the Maximum Dry Density (MDD) as shown in Figure 3-6 and Table 3-13, CAEMs for Categories 2 and 3 were produced at their Optimum Bitumen Emulsion Contents (OBEC) and Pre-Wet Water Contents (PWC).

Following this, compaction was conducted using the Cooper gyratory compactor at a pressure of 600kPa, rate of gyration of 30revs/minute and an angle of gyration of 1.25°.

The gyratory compactor was set to 200 gyrations and results evaluated to ascertain the number of gyrations to achieve the MDD, otherwise termed the target density, as obtained from the Modified Proctor test results for both Categories 2 and 3. The results are shown below in Figure 3-18.

Evaluating the results, the number of gyrations required to reach the target density for Category 2 was slightly lower than that for Category 3. This is probably as a result of the higher OTFC for Category 2 influencing the compaction characteristics of the CAEMs making Category 2 easier to compact in comparison to Category 3.

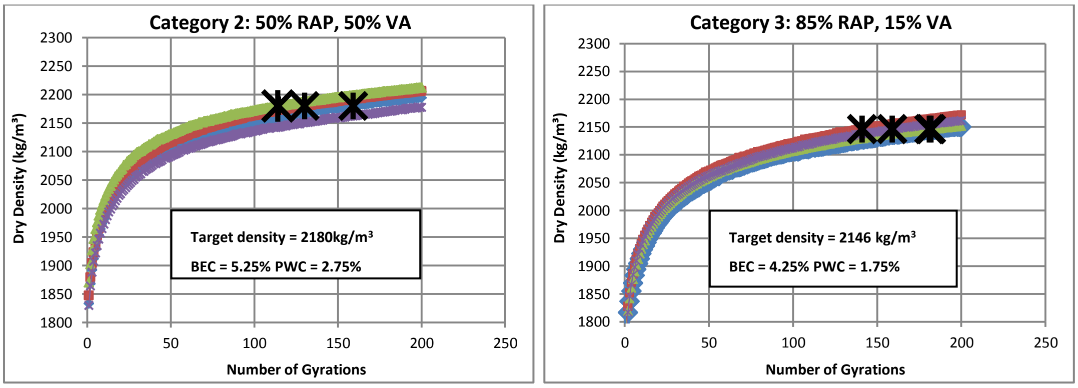


Figure 3-18: Obtaining the number of gyrations for Category 2 and Category 3

Overall, it can be seen that the number of gyrations to achieve the target densities for Category 2 and 3 lie within the range of 114 – 182 gyrations. After careful consideration, a compaction effort of 150 gyrations was adopted as the appropriate compaction effort to be used in producing the CAEMs for all categories for the research project. This compaction effort and limit have been found to be suitable for mixtures incorporating CAEMs as stated by (Oke, 2010).

3.12 Summary

This chapter has investigated and examined the process in ascertaining the mix design for the CAEMs studied for this research. The overall objectives of the study was to produce durable, efficient and consistent CAEMs made with high contents of RAP with suitably good mechanical and performance properties and in the process develop a consistent mix design procedure for the CAEMs. A guideline for CAEMs with high RAP contents is presented in Figure 3-19.

Figure 3-19: Mix Design Guideline for CAEMs with High RAP Content

The following conclusions can be deduced from this mix design study:

- The residual binder content of the RAP was obtained as 5.5%.
- The penetration and softening point values of the residual binder after extraction were 20dmm and 64.2°C respectively.
- Based on the results of the cohesion tests conducted, the RAP used in this study can be classified as "active" positively influencing the mechanical and performance properties of the produced CAEMs. This needs to be taken into account in ascertaining the OBEC of the CAEMs as the RAP used in the study cannot be treated as "black rock".
- It was assumed that there was 100% residual binder contribution.
- The OBEC and PWC for the various categories are stated below:

    - Category 1: 6.5% BEC and 1.5% PWC
    - Category 2: 5.25% BEC and 2.75% PWC
    - Category 3: 4.25% BEC and 1.75% PWC
    - Category 4: 3.75% BEC and 1.75% PWC

- Table 3-17 presents the total bitumen and water content for the various categories as stated above taking into account bitumen proportion from the emulsion and residual binder contribution from the RAP aggregate material. The total water is a function of the PWC added and the water contribution from the bitumen emulsion. It should be noted that the bitumen emulsion used had 60% water and 40% water. The residual binder content as stated above was 5.5%.

Category	Total Bitumen (%)	Total Water (%)
Category 1	3.9	4.1
Category 2	5.9	4.9
Category 3	7.2	3.5
Category 4	7.5	3.3

Table 3-17: Total Bitumen and Water Contribution

- The procedure for manufacturing the CAEMs was optimised.
- The sequence adopted for mixing the specimens comprised of a combination of mixing by hand and using the Sun and Planet mixer for a total of 4 minutes.
- This procedure produced a homogenous and consistent mix.
- The stiffness response was superior in comparison to the other options investigated.
- The Cooper gyratory compactor was used in producing 100mm cylindrical cores used in the research study.
- The parameters for compaction are: Pressure of 600kPa, rate of gyration of 30revs/minute and an angle of gyration of 1.25°.

4. STUDY ON ACCELERATED CURING

4.1 Introduction

This part of the study investigated the curing mechanism of CAEMs. Curing is the process in which a material develops stiffness and strength with time as the material discharges water through evaporation, particle charge repulsion, bitumen charge coalescence or pore induced flow paths (Jenkins, 2000).

In comparison to Hot Mix Asphalts (HMA), the behaviour of CAEMs is more complex due to the presence of water in the mix resulting in a multiphase material with low strength and stiffness at the early life which then increases as water is dissipated through the curing process. Studies on accelerated curing and conditioning regimes for CAEMs currently exist and these are used in a variety of ways by researchers although the underlying question remains: how realistic are these curing and conditioning regimes in simulating pavement conditions as experienced in the field? Typical curing regimes used include:

- 60°C for 3 days (Maccarone et al., 1994)
- 40°C for 3 days (Widely used, TG2: Asphalt Academy)
- 20°C for 28 days (Bocci et al., 2011)
- 25°C for 2 days (Brovelli and Crispino, 2012)
- 40°C for 18-21 days (Thanaya et al., 2009)

Most accelerated curing regimes fail to reproduce the actual condition of the mix that is expected in the pavement and tests are therefore conducted at superior conditions. This results in overstating the mix performance especially at the early life. This is especially true when the fundamental properties of the CAEMs are compared with other mixes such as HMA and Warm Mix Asphalt (WMA) (Ojum et al., 2014). This chapter investigates the need for accelerated curing and the degree of acceleration that is realistic by studying different curing regimes and their effect on mechanical and performance behaviour of CAEMs. The experimental design was selected in such a way that the curing conditions include regimes that are followed by different highway agencies and also conditions that simulate different climatic zones.

4.2 Experimental Plan

Curing has a significant effect on the mechanical and performance properties of cold mix asphalts (Jenkins and Moloto, 2008). This study aimed to provide a valid investigation into the behavior of CAEMs taking into account:

1. The effect of temperature, curing conditioning and curing duration.
2. The influence of cement with different curing regimes.
3. The mechanical and performance properties of CAEMs with respect to the different curing regimes.

In order to achieve the objectives of the study, a comprehensive experimental curing design program as stated in Table 4-1 was developed.

Phase	Curing Temperature (°C)	Curing duration (days)	Curing Condition	Cement Content (OPC)
1	5	28	Fully Wrapped (FW)	0%
	20			
	40			
2	5	28	Unwrapped (UW)	0%
	20			
	40			
3	5	28	Fully Wrapped (FW)	1%
	20			
	40			
4	5	28	Unwrapped (UW)	1%
	20			
	40			
5	20	21	Combination* (COMBO)	0%
6	20	21	Combination* (COMBO)	1%
7	40	3	Fully Wrapped (FW)	0%
8	40	3	Fully Wrapped (FW)	1%

*COMBO - Partially wrapped for 7 days + fully wrapped for 14 days

Table 4-1: Summary of the Experimental Design Program

4.3 Methodology

As previously mentioned, a key objective of curing in the laboratory is to simulate field conditions in order not to overestimate the mechanical and performance properties of the laboratory produced mixtures. Three curing temperatures were investigated in this study as detailed in the experimental plan as stated in Table 4-1. The experimental plan was designed to include regimes in use by various agencies and conditions that simulate different climatic zones. Ruckel et al., (1983) suggested a curing regime of 40°C for 24 hours to simulate between 7 – 14 days in the field and duration of 72 hours to simulate longer term curing of about 30 days in the field.

This is widely used in literature. This regime is specified in the South African Technical Guide TG2 (Asphalt Academy of South Africa, 2002). This temperature was investigated for this study albeit at curing durations of 3 days and 28 days to simulate short and long term accelerated curing of CAEMs.

Ambient temperatures of about 20°C have also been used by researchers for curing. Ruckel et al., (1983) states that cold mix asphalts should be left in the compaction mould at ambient temperatures for 24 hours. This is due to the fragile nature of the specimens at the early life. This was adopted for use in this study. Long and Thyse, (2002), Long and Ventura, (2004), Saleh, (2004) have used ambient temperatures of about 20°C for longer durations of between 7 – 28 days. 20°C was investigated in this study at curing durations of 21 and 28 days.

The final temperature investigated was 5°C to simulate cold climatic conditions. This is to provide an in-depth analysis on purported mechanical and performance properties of CAEMs at such reduced temperatures. The specimens were cured for 28 days at this temperature state.

Curing at 60°C was deemed inappropriate for this study as it was above the softening point of the bitumen used in the research. It was avoided in order not to adversely affect the inherent properties of the binder in the emulsion resulting in ageing. (Jones et al., 2008) discovered that at such elevated temperatures, there was apparent binder flow resulting in altered binder properties.

To achieve the objectives, a set of samples were fully wrapped (FW), unwrapped (UW) and a third mode of conditioning was partial wrapping for 7 days followed by full wrapping for a further 14 days termed “COMBO”. This was to simulate field conditions in which the binder or surface course is laid over the CAEM layer after 7 days. 1% cement was included in conducting this investigation. The detailed experimental plan is as stated in Table 4-1.

Category 2: 50% RAP, 50% VA was used in this investigation. The physical properties and aggregate gradation of both the RAP and VA, the optimum mix design properties including the mixing sequence and compaction methods are as presented in the mix design chapter of this thesis. The specimens were wrapped using a special aluminum foil tape.

Figure 4-1 shows the cores in their respective curing conditions.

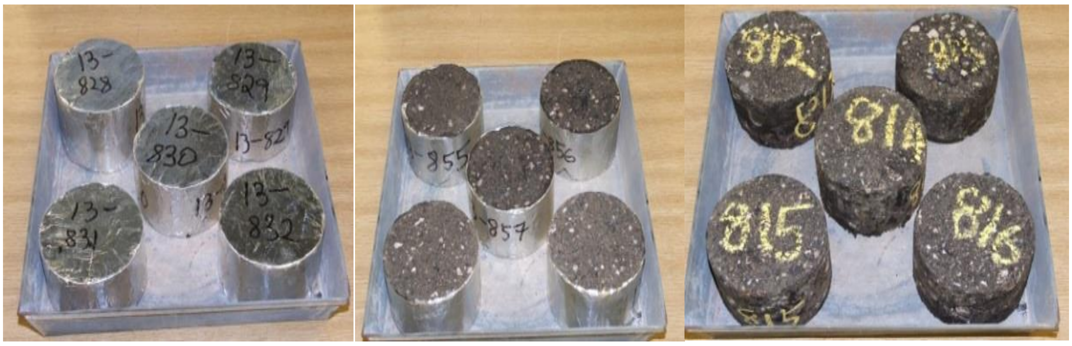


Figure 4-1: (L-R) – Fully Wrapped, Partially Wrapped and Unwrapped

4.4 Test Methods

The stiffness modulus was determined using a servo-pneumatic testing machine (NU-14) by applying indirect tension to the cylindrical specimens. The Indirect Tensile Stiffness Modulus (ITSM) test was conducted at 20°C in accordance with BS EN 12697-26 specifications.

The Indirect Tensile Strength (ITS) test was conducted to determine the strength characteristics of the CAEMs. The test involved applying compressive loads to the samples between two loading strips which creates tensile stresses along the vertical diametric plane causing a splitting failure (Kavussi and Modarres, 2010). The test was conducted at 20°C in accordance with BS EN 12697-23. At each condition, the average of 3 CAEMs was used in ascertaining the stiffness and strength characteristics.

The Repeated Load Axial Test (RLAT) was conducted in order to ascertain the resistance to permanent deformation of the CAEMs under the various curing regimes. The test method was used in ranking the CAEMs on the basis of resistance to permanent deformation which acted as a guide to the relative performance of the mixtures. The test was conducted at 30°C in accordance with standards as stated in BS DD 226. The average of 2 CAEMs was used in ascertaining the permanent deformation characteristics at each conditioning level. The charts graphically display the variability in the data using error bars that show the standard deviation. The standard deviation gives an indication of how closely clustered the data points are around the mean values calculated.

4.5 Test Results

The average air void contents for all mixtures produced to investigate the effects of accelerated curing on CAEMs in this study are presented in Table 4-2.

Phase	Curing Temperature (°C)	Air Void Content (%)	Curing duration (days)	Curing Condition	Cement Content (OPC)
1	5	12.7	28	Fully Wrapped (FW)	0%
	20	12.7			
	40	13.1			
2	5	15.6	28	Unwrapped (UW)	0%
	20	15.6			
	40	16.1			
3	5	12.3	28	Fully Wrapped (FW)	1%
	20	13.1			
	40	13.6			
4	5	15.0	28	Unwrapped (UW)	1%
	20	15.6			
	40	15.6			
5	20	14.4	21	Combination* (COMBO)	0%
6	20	14.4	21	Combination* (COMBO)	1%
7	40	12.1	3	Fully Wrapped (FW)	0%
8	40	12.6	3	Fully Wrapped (FW)	1%

Table 4-2: Air Void Contents for Mixtures Produced

4.5.1 Effect of curing conditioning

Figure 4-2 presents stiffness modulus results for the CAEMs for 28 days at three different temperatures at 0% and 1% OPC. Figure 4-2 show that the fully wrapped (FW) specimens had the lowest stiffness values notably those cured at 5°C with an average value of 929MPa for the CAEMs with 0% OPC.

The highest stiffness values for the FW mixtures with 0% OPC were observed on specimens cured at 40°C with an average value of 1791MPa. At all curing temperatures, the unwrapped (UW) specimens with 0% OPC had stiffness properties in the region of 2 to 2.5 times the stiffness of the FW specimens.

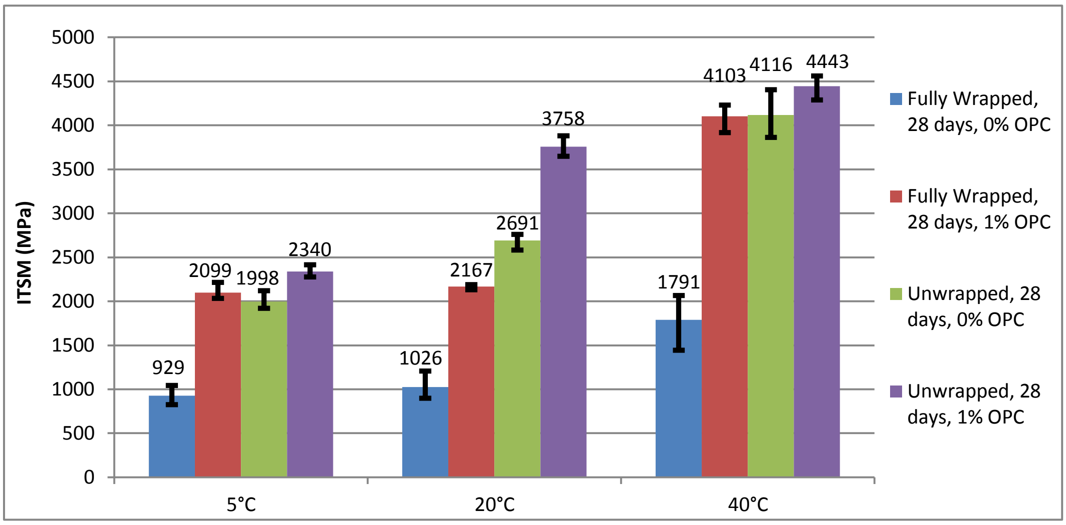


Figure 4-2: ITSM test results showing the effect of curing conditioning

The ITS test results are presented in Figure 4-3. The trends observed for the ITS test results are similar to those for stiffness as depicted in Figure 4-2. The lowest strength values were observed also for the FW specimens cured at 5°C. At 0% OPC, UW specimens achieved 1.5 to 2.5 times the strength of the FW specimens.

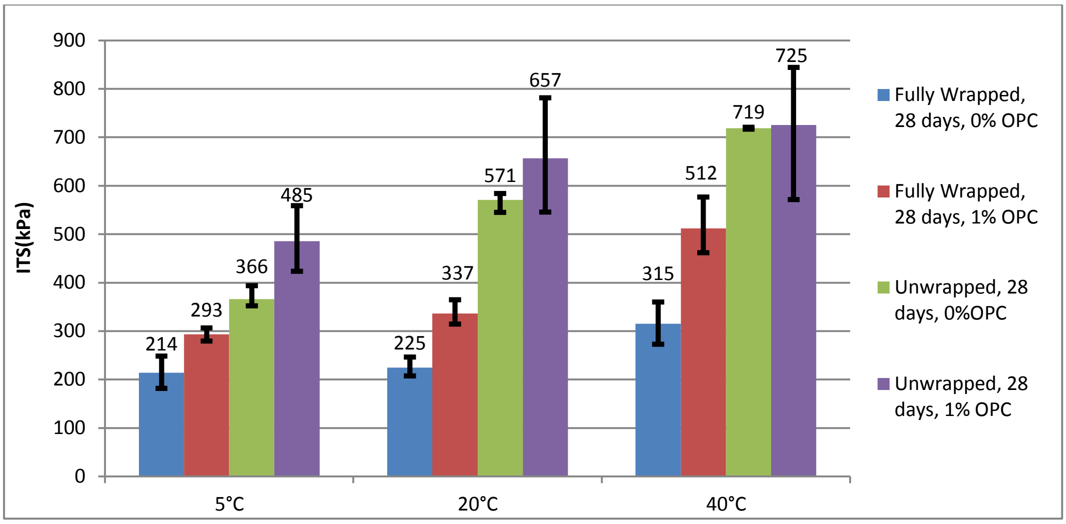


Figure 4-3: ITS test results showing the effect of curing conditioning

4.5.2 Effect of Temperature

The influence of curing temperature on the stiffness modulus and strength development of the mixtures is illustrated in the case of specimens with 0% OPC as presented in Figure 4-4. The stiffness modulus and strength development increases with increase in curing temperature which led to higher maximum values. This was the observed trend irrespective of the curing conditioning employed.

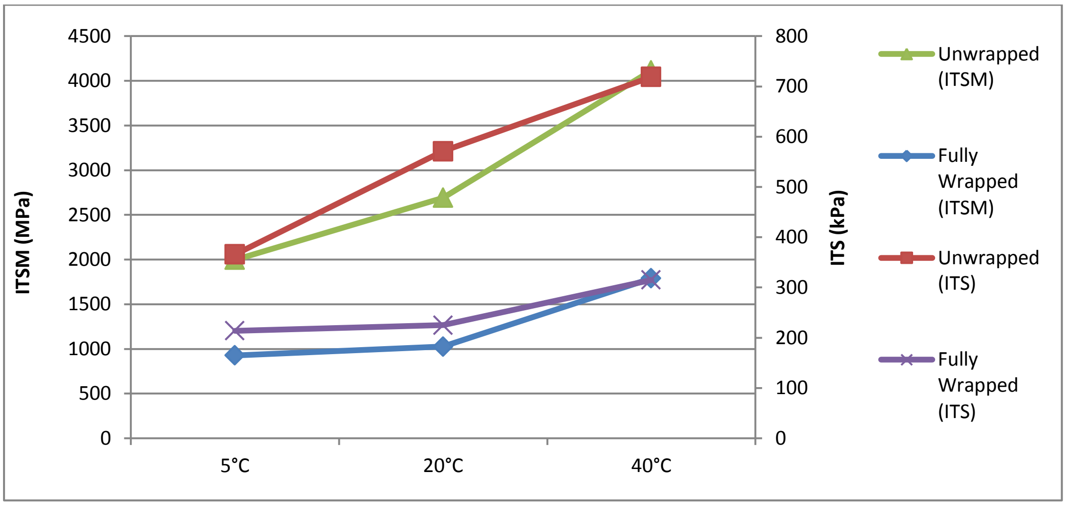


Figure 4-4: Stiffness and Strength Trends at 0% OPC

A major occurrence during the curing process is the evaporation of water from the CAEMs. Water loss in the CAEMs was monitored by successive weight measurements at intermediate times and the results are presented in Figure 4-5. The moisture loss was monitored using an average of 5 CAEMs and the error bars show the standard deviation around the mean values. There is a significant difference between the amount of moisture loss between the FW and UW specimens as depicted in Figure 4-5.

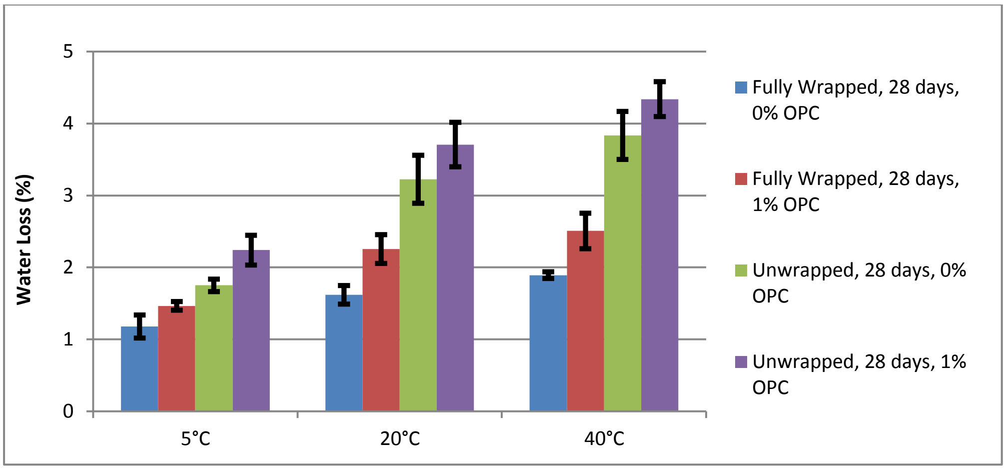


Figure 4-5: Water Loss in the CAEMs

After 28 days curing, the moisture in the UW specimens had evaporated more in comparison to the FW specimens. This trend links directly to the fact that as moisture evaporates from the CAEMs, there is an increase in the stiffness and strength of the mixtures as reflected in the stiffness and strength trend observed from the tests conducted as shown in Figure 4-2 to Figure 4-4. This trend was also observed by Needham and Brown (2000). In summary, observations from the results presented in this study show that the bonding provided by the bitumen emulsion develops as moisture evaporates and the mixtures attain optimal levels once most of this water is no longer present in the CAEMs. Increase in curing temperature further facilitates the evaporation of moisture in the CAEMs leading to improved mechanical and performance properties as depicted in Figure 4-2 to Figure 4-4.

4.5.3 Influence of Cement

The addition of cement increased the stiffness and strength at all curing temperatures for both FW and UW conditions as a result of the evaporation of water from the CAEMs and the formation of cementitious bonds.

In terms of stiffness, the addition of 1% OPC to FW specimens significantly improved the 28 day values of the FW specimens. Evaluating the ITS test results, there was an increase in the strength of the CAEMs when 1% OPC was added to the FW specimens although it was not at the same proportion as that seen in the stiffness results as presented in Figure 4-2 and Figure 4-3. To conclude, the addition of cement to the CAEMs improved the rate of increase and the obtained stiffness modulus and strength values acting as a secondary binder. Cement hydration creates heat which facilitates curing of the CAEMs leading to increased stiffness and strength. Overall, inclusion of cement to the mixtures had a positive effect on the mechanical and performance properties of the CAEMs which could be a means of improving CAEMs especially at early life and during adverse weather conditions.

4.5.4 Influence of Curing Time

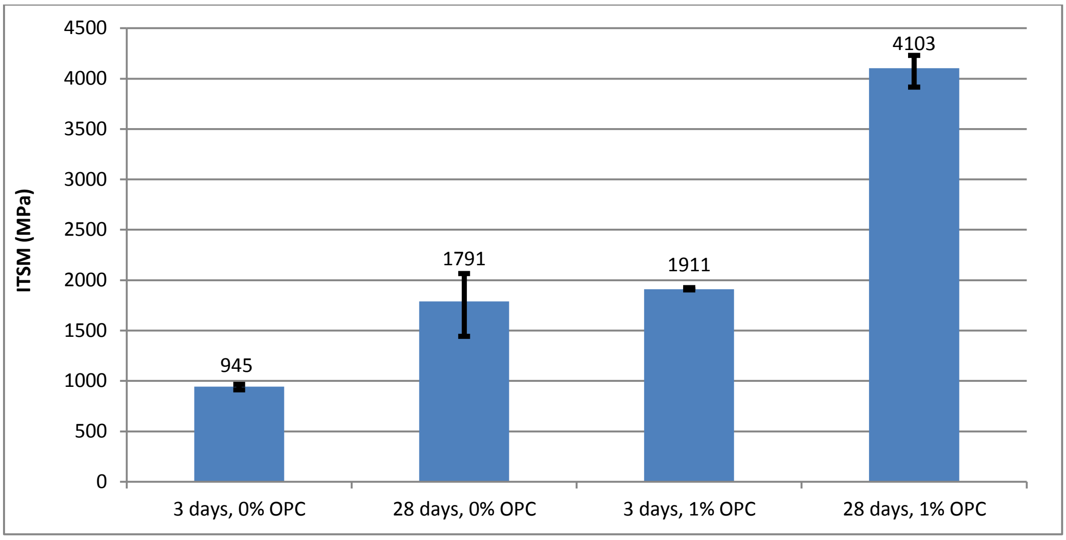


Figure 4-6: ITSM results for 3 and 28 days curing at 40°C at 0% and 1% OPC

This can be seen in Figure 4-6 and Figure 4-7 where the 3 day and 28 day FW specimens, cured at 40°C using 0% and 1% OPC are presented and compared.

Typically, it may be found that the stiffness values for the FW specimens, cured at 40°C for 3 days, were about double the values achieved at 28 days. The ITS results had a proportion of about 58% as seen below in Figure 4-7.

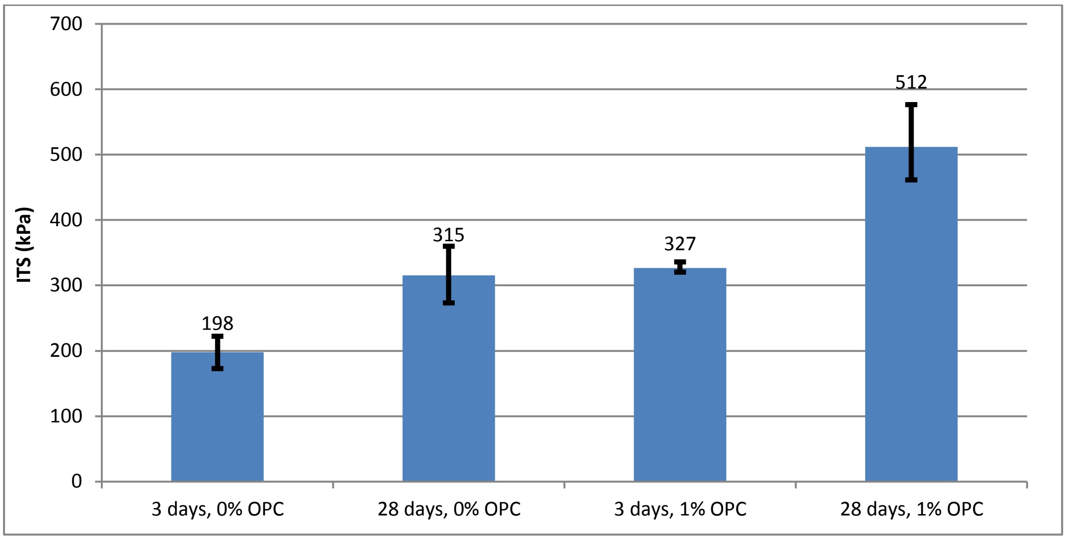


Figure 4-7: ITS results for 3 and 28 days curing at 40°C at 0% and 1% OPC

The combination of 7 days partially wrapped and 14 days fully wrapped (COMBO) was introduced to simulate typical field conditions with a surface or binder course applied 7 days after construction of the base layer and trafficking commencing 14 days later. Figure 4-8 and Figure 4-9 below compares this curing condition to the 28 day curing period of FW and UW curing conditions at 20°C. It is observed that in every case, the COMBO lies in between the FW and UW curing conditions. This suggests that 28 days UW especially at 40°C may result in an overestimate of actual performance and also suggests that 28 days FW may be conservative. This could also be an indication that CAEMs require duration of time to allow for curing before an overlay is placed or prior to trafficking.

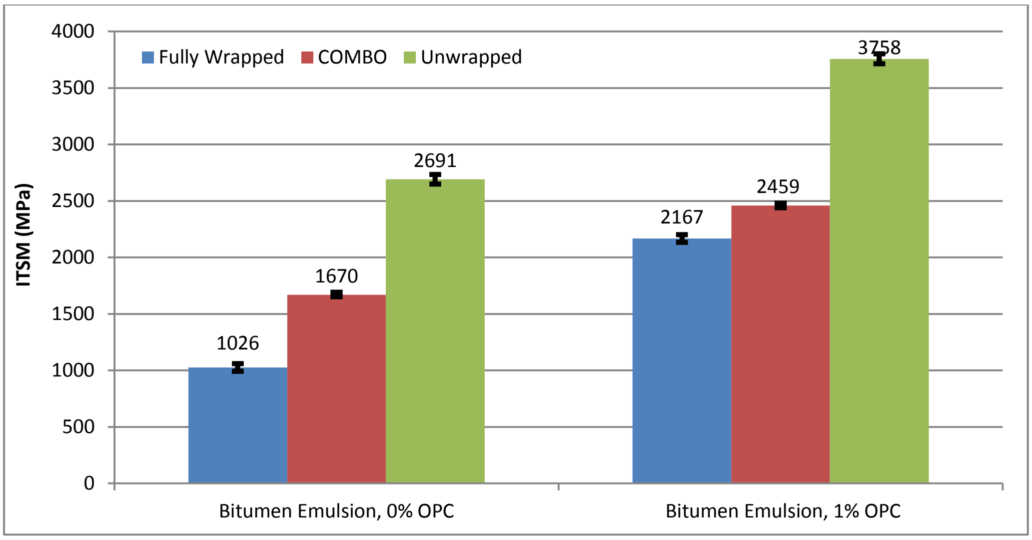


Figure 4-8: ITSM test results comparing the COMBO curing condition

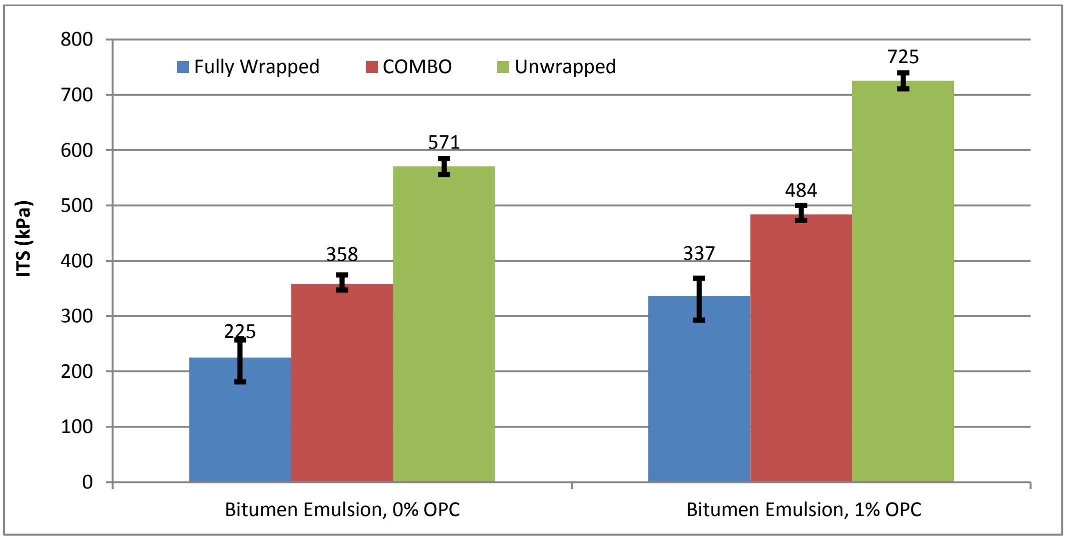


Figure 4-9: ITS test results comparing the COMBO curing condition

This study suggests that improvements in the mechanical and performance properties of the CAEMs are obtained after significant evaporation/loss of moisture from the mixture. The results also show the role of cement and its usefulness in positively improving the mechanical and performance properties especially at the early life of CAEMs.

4.5.5 Resistance to Repeated Loading

To complete this study the resistance to permanent deformation was measured using the Repeated Load Axial test (RLAT) in accordance to BS DD 226. Test specimens were subjected to repeated load axial pulses that aim to simulate loading conditions in road pavements. The parameters used are stated below:

- Conditioning Stress: 10kPa
- Conditioning Period: 600s
- Test Stress: 100kPa
- Pulse: 1s on, 1s off
- Test Duration: 1800 pulses
- Test Temperature: 30^{0}C

Figure 4-10 depicts the axial strain development of the FW and UW CAEMs at 20°C with 0% and 1% OPC. It reiterates the trends observed as seen in the results obtained from the stiffness and strength analysis. The average of two samples was used in ascertaining the permanent deformation characteristics of the CAEMs used in the research. The following observations were made. The FW specimens had significantly higher axial strains in comparison to the UW specimens. As seen in Figure 4-5, the rate of water loss in the FW CAEMs was significantly lower in comparison to the UW CAEMs. This was most evident in the FW CAEMs at 0% OPC. This could be a result of pre-existing water in the FW specimens acting as a lubricant and adversely reducing the CAEMs resistance to permanent deformation.

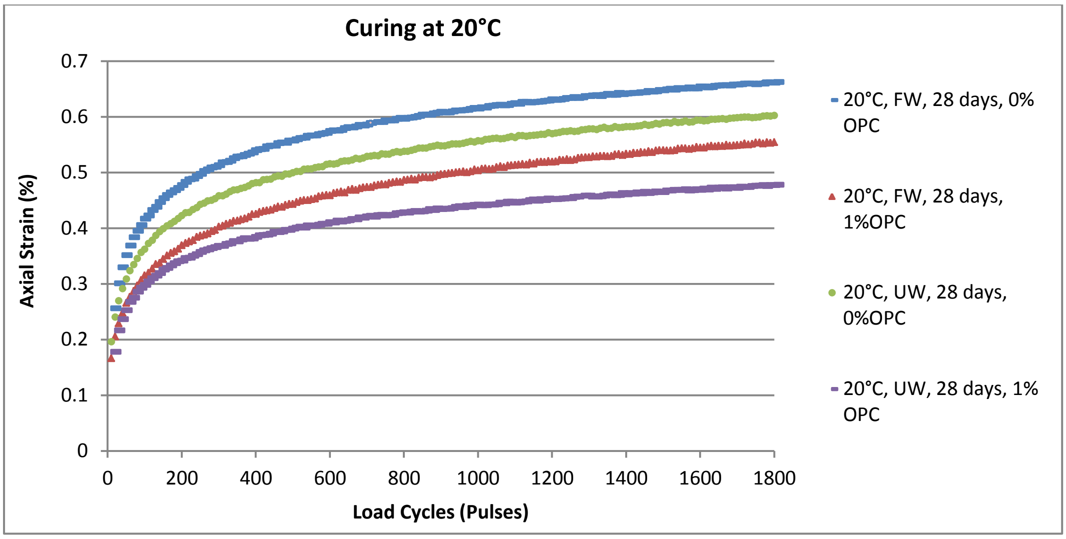


Figure 4-10: RLAT Test Results

The addition of 1% cement in the mixture positively influenced the ability of the CAEMs to resist permanent deformation and this was evident also for the FW CAEMs with 1% OPC. This reiterates the positive effect of adding 1% OPC to CAEMs. Figure 4-11 shows that at increased curing temperatures, there was a reduction in axial strains for the CAEMs indicating enhanced resistance to permanent deformation. This was evident at all curing conditions which were consistent with trends observed for the stiffness and strength analysis.

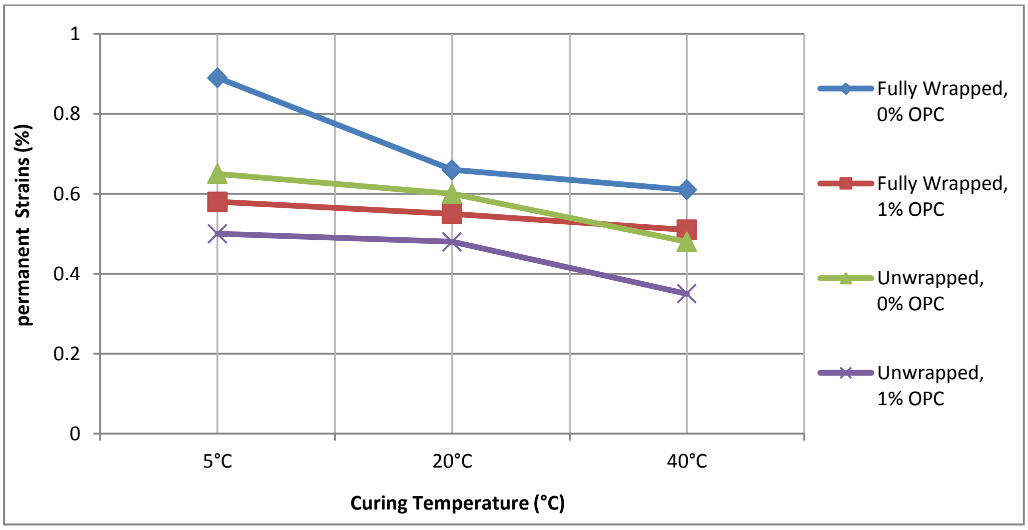


Figure 4-11: Permanent Strains after 1800 cycles

4.6 Summary

To summarize, the suite of tests carried out on the CAEMs has resulted in several interesting conclusions with practical repercussions, namely:

- Curing temperature significantly affects the rate of curing and the 28-day strength and stiffness values.
- The ratio of 28-day stiffness results of specimens cured unwrapped to those cured fully wrapped is typically 2-2.5 and this is independent of curing temperature. The equivalent indirect tensile strength ratio is 1.5-2.5.
- The inclusion of 1% cement in the fully wrapped case leads to stiffness results approximately equal to those of unwrapped specimens without cement, suggesting that 1% cement addition can be an appropriate means of improving the properties of the mixtures at early life and also countering the effects of adverse weather.
- Strengths and stiffness results at 3 days are approximately 50-70% of those at 28 days, illustrating the danger of damage due to early trafficking.
- A 2-stage curing regime intended to simulate site practice produced strength and stiffness data that lay between the wrapped and unwrapped 28-day results.
- The same trends appeared from tests for permanent deformation resistance as for strength and stiffness.

4.7 Conclusion

This chapter has investigated and evaluated various curing and conditioning regimes providing insight into how curing regimes influence the mechanical and performance properties of CAEMs. It should be noted that this study was also conducted on foamed bitumen bound cold asphalt mixtures as detailed in (Ojum et al., 2014).

The behaviors of emulsion-bound and foamed-bitumen bound materials had very similar trends in terms of the relative effects of curing time and temperature, specimen wrapping and cement addition. However, the emulsion bound materials had consistently better performance. This could be due to the fact that a harder grade of binder was used. Overall, the results showed that bonding develops as the mixing and compaction water evaporates from the CAEMs. Curing temperature and conditioning plays a crucial role. Incomplete evaporation of the water as in the FW and COMBO specimens will result in reduced performance for the CAEMs as they do not allow complete evaporation of moisture from the mixtures.

Based on findings and in line with objectives of the research, it was decided that the curing regime to be adopted for the study would be 20°C, UW for 28 days. This is to facilitate the evaporation of water and promote effective curing of the CAEMs produced in the laboratory without overstating the properties of the CAEMs as 40°C, UW for 28 days might be overstating the mechanical and performance properties of the mixtures.

5. DETAILED INVESTIGATION INTO THE MECHANICAL AND PERFORMANCE PROPERTIES OF THE COLD ASPHALT EMULSION MIXTURES

5.1 Introduction

This chapter discusses and analyses the results of investigations on the Cold Asphalt Emulsion Mixtures (CAEMs) designed in previous chapters. The investigation includes results of the properties of CAEMs produced using different contents of RAP (Categories 1 – 4) with and without the use of cement (0%, 1% and 3% OPC). The study also looked at the compaction characteristics of these mixture types focusing on the effects of the mixing and compaction temperatures. The study was needed in order to properly characterise the stiffness properties, fatigue characteristics, permanent deformation and durability of the mixtures which constitute the mechanical and performance properties of the CAEMs. Results obtained from these analyses were then used in the structural design segment of the research.

The chapter starts by summarising the procedures required in producing the CAEMs. This is followed by presenting the theory of the indirect tensile mode of testing as this forms a significant mode of testing used in the research. A description is given of the Indirect Tensile Stiffness Modulus (ITSM), Vacuum Repeated Load Axial Test (VRLAT) and the Indirect Tensile Fatigue Test (ITFT) followed by the detailed presentation of results, findings and analyses including moisture susceptibility of the CAEMs.

5.2 Specimen Preparation

5.2.1 Materials

The materials used are as described in Chapter 3 which includes:

- Reclaimed Asphalt Pavement (RAP)
- Virgin Aggregate (VA) comprising limestone and sharp sand
- Bitumen Emulsion
- Cement
- Water

To achieve the objectives of the research, four proportions of RAP and Virgin Aggregate (VA) materials were used as categorised below:

- Category 1: 0% RAP (no RAP, 100% VA)
- Category 2: 50% RAP (50% RAP, 50% VA)
- Category 3: 85% RAP (85% RAP, 15% VA)
- Category 4: 95% RAP (95% RAP, 5% VA)

The influence of temperature when mixing, placing and compacting on the performance of CAEMs was investigated in this study to simulate typical winter, moderate and tropical temperature conditions. How this affects the material properties including the ease of mixing, coating and compaction has been examined.

The mixing and compaction temperatures investigated include:

- Temperature 1: 5°C
- Temperature 2: 20°C
- Temperature 3: 32°C

The study also evaluated the influence of cement (OPC) dosage levels on the CAEMs which comprised:

- 0% Cement
- 1% Cement
- 3% Cement

Chapter 3 detailed and characterised the properties of the materials used in the research. Table 3-17 presents the total bitumen and water content for the various categories as stated above taking into account bitumen proportion from the emulsion and residual binder contribution from the RAP aggregate material. The residual binder content was 5.5%. Table 5-1 provides a summary of the procedures and parameters used in preparing the specimens. The Cooper Gyratory Compactor was used extensively in this study in the compaction of the CAEMs. Table 5-1 provides a summary of key parameters used for producing the CAEMs. The tests conducted on the CAEMs in order to ascertain the mechanical and performance characteristics of the mixtures for the study include: Indirect Tensile Stiffness Modulus (ITSM), Indirect Tensile Fatigue Test (ITFT), Vacuum Repeated Load Axial Test (VRLAT) and Water Susceptibility Tests.

<table>
<tr><td rowspan="5">1. Design Contents</td><td>Category</td><td>Mix Proportions</td></tr>
<tr><td>Category 1: 0% RAP (no RAP, 100% VA)</td><td>6.5% BEC + 1.5% PWC</td></tr>
<tr><td>Category 2: 50% RAP (50% RAP, 50% VA)</td><td>5.25% BEC + 2.75% PWC</td></tr>
<tr><td>Category 3: 85% RAP (85% RAP, 15% VA)</td><td>4.25% BEC + 1.75% PWC</td></tr>
<tr><td>Category 4: 95% RAP (95% RAP, 5% VA)</td><td>3.75% BEC + 1.75% PWC</td></tr>
<tr><td>2. Batching and Mixing</td><td colspan="2">• One batch makes 4 samples at 1000g per sample.
• The Sun and Planet mixer was used for this study.
• The aggregate materials are placed into the mixing bowl of the Sun and Planet mixer and mixed for 60s.
• The PWC was added in a thin stream and mixed for 60s using the mixer.
• This was followed by mixing using a spatula for 30s ensuring that the aggregate materials are thoroughly blended and wetted ready for the addition of the bitumen emulsion.
• The bitumen emulsion was added and mixed with the Sun and Planet mixer for 60s.
• To ensure homogeneity and consistency in the mix, the materials are then mixed by hand using a spatula for 30s.
• Total mixing time of 240s (4 minutes).</td></tr>
<tr><td>3. Compaction</td><td colspan="2">• 100mm diameter moulds are utilised.
• Angle of gyration – 1.25°
• Pressure – 600kPa
• Target number of gyrations - 150</td></tr>
<tr><td>4. Curing</td><td colspan="2">• Standard curing regime - 20°C, Unwrapped (UW) for 28 days unless stated otherwise.</td></tr>
<tr><td>5. Specimen Management</td><td colspan="2">• The specimens are left in the mould after compaction for 24 hours at room temperature to prevent damage to the specimens at their early life.
• Extraction of the specimens is done carefully with the use of a hydraulic de-moulding machine.
• Curing of the specimens follow.
• After curing, if testing is not immediately done, the specimens are stored in the cold store at 5°C.</td></tr>
<tr><td></td><td colspan="2">BEC - Bitumen Emulsion Content
PWC – Pre-wetting Water Content</td></tr>
</table>

Table 5-1: Summary of Parameters for Producing the CAEMs

The tests conducted were mostly done using the Nottingham Asphalt Tester (NAT) machines at the laboratory of the Nottingham Transportation Engineering Centre (NTEC) at the University of Nottingham. These machines were used extensively for testing bituminous materials due to their ease of use, user friendly interface, accuracy and adaptability. The NAT comprises four units that include a main test frame placed in a temperature control cabinet, an interface unit for data acquisition and test control, a pneumatic unit connected with the interface unit and an actuator mounted above the test frame used in controlling the applied load and a computer unit as shown in Figure 5-1.

Prior to analysing the results of the investigation into the mechanical and performance properties of the CAEMs, the indirect tensile mode is discussed in more detail.

5.3 Theory of the Indirect Tensile Mode

In the indirect tensile mode, a compression load is applied across the vertical diameter of the cylindrical specimen which results in a bi-axial stress distribution in the specimen as depicted in Figure 5-1.

Figure 5-1 shows that on loading a vertical compressive stress (σ_{vx}), a horizontal tensile stress (σ_{hx}) is induced on the horizontal diameter of the specimen. The magnitudes of these stresses differ along the diameter and reach a peak value at the centre of the specimen.

The stiffness modulus can be determined by then measuring the horizontal deformation (Δh), horizontal tensile stress (σ_{hx}) and the vertical compressive stress (σ_{vx}) across the horizontal diameter.

The calculations for the stiffness modulus rely on the following assumptions:

- The specimen is subjected to plane stress which occurs when ($\sigma_z = 0$).
- The material is linear elastic, homogenous and isotropic.
- Poisson's ratio (υ) is known.
- The vertical load (P) is applied as a line loading.

If these conditions are met, it can be assumed that the stress conditions in the specimen are in accordance with the theory of elasticity.

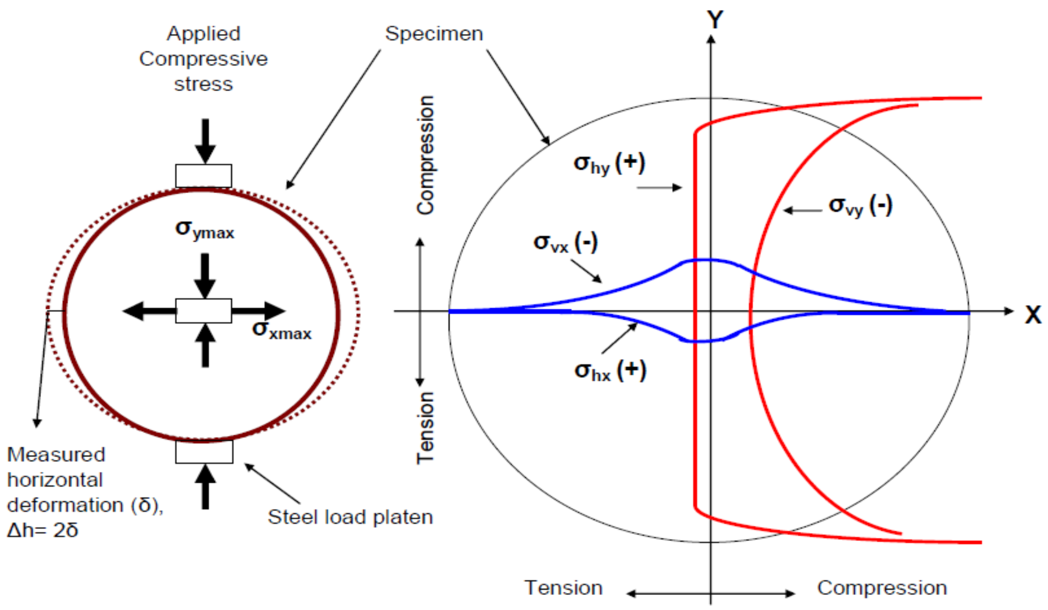


Figure 5-1: Indirect Tensile Test Mode showing Stress Distributions under Loading

Where: σ_{vx} = Vertical stress across x-axis (Compression)

σ_{hx} = Horizontal stress across x-axis (Tension)

σ_{vy} = Vertical stress across y-axis (Compression)

σ_{hy} = Horizontal stress across y-axis (Tension)

The theory of elasticity states that when the width of the loading strip is less than or equal to 10% of the diameter of the specimen and the distance of the element of material from the centre is very small then Equation 5-1 to Equation 5-4 can be applied as stated by (Read, 1996).

Equation 5-1

$$\sigma_{hx} = \frac{2P}{\pi . d . t}$$

Equation 5-2

$$\sigma_{vx} = \frac{-6P}{\pi . d . t}$$

The average horizontal and vertical stresses are determined as follows:

Equation 5-3

$$\overline{\sigma_{hx}} = \frac{0.273P}{d . t}$$

Equation 5-4

$$\overline{\sigma_{vx}} = \frac{-P}{d . t}$$

Where:

d = specimen diameter

t = specimen thickness

P = applied compression load

σ_{hx} = maximum horizontal tensile stress at the centre of the specimen

σ_{vx} = maximum vertical compressive stress at the centre of the specimen

$\overline{\sigma_{hx}}$ = average horizontal tensile stress

$\overline{\sigma_{vx}}$ = average vertical compressive stress

Taking into account the average principal stresses in a small element subjected to biaxial stress conditions, the horizontal tensile strain would be:

Equation 5-5

$$\overline{\varepsilon_{hx}} = \frac{\overline{\sigma_{hx}}}{S_m} - \upsilon \frac{\overline{\sigma_{vx}}}{S_m}$$

Equation 5-6

$$\overline{\varepsilon_{hx}} = \frac{0.273P}{S_m.d.t} + \frac{\upsilon.P}{S_m.d.t}$$

The horizontal deformation is determined by the following equations:

Equation 5-7

$$\Delta h = \overline{\varepsilon_{hx}}.d$$

Equation 5-8

$$\Delta h = \frac{0.273P}{S_m.t} + \frac{\upsilon.P}{S_m.t}$$

Therefore, the stiffness modulus can then be calculated as:

Equation 5-9

$$S_m = \frac{P\,(0.273 + \upsilon)}{\Delta h.t}$$

Where:

$\overline{\varepsilon_{hx}}$ = average horizontal tensile strain

υ = Poisson's ratio

S_m = Stiffness Modulus

Δh = horizontal deformation (measured)

The strain can be ascertained for analysis of indirect tensile fatigue. The strain is calculated at the centre of the specimen where the stresses are at a maximum taking into account Equation 5-1 and Equation 5-2: Equation 5-5 can then be expressed as:

Equation 5-10

$$\varepsilon_{\mathrm{hx\,(max)}} = \frac{\sigma_{\mathrm{hx(max)}}}{S_{\mathrm{m}}} - \upsilon \frac{\sigma_{vx(max)}}{S_m}$$

Substitute Equation 5-1 and Equation 5-2 into Equation 5-10:

Equation 5-11

$$\varepsilon_{hx(max)} = \frac{2.P}{S_m \pi.d.t} + \frac{\upsilon.6P}{S_m \pi.d.t}$$

Substitute Equation 5-1 into Equation 5-11:

Equation 5-12

$$\varepsilon_{hx\,(max)} = \frac{\sigma_{\mathrm{hx(max)}}.(1+3\upsilon)}{S_m}$$

Where:

$\varepsilon_{hx(max)}$ = maximum horizontal tensile strain at the centre of the specimen.

Taking into account the above equation, it can be used in calculating the maximum horizontal tensile strain at the centre of the specimen. This strain value is essential for the indirect tensile fatigue test. The stiffness modulus and the maximum tensile stress values are also crucial parameters required in obtaining the tensile strain for the fatigue test.

5.3.1 Indirect Tensile Stiffness Modulus (ITSM) test

The Indirect Tensile Stiffness Modulus (ITSM) test is carried out in accordance with BS EN 12697-26:2012. This test is non-destructive and measures the visco-elastic response of a material. The test makes use of cylindrical specimens either cored from field or prepared in the laboratory with a diameter of either 100mm or 150mm and thickness ranging between 30mm to 80mm. Table 5-2 provides a summary of the test parameters for the ITSM test while Figure 5-2 depicts the test configuration of the ITSM test in the NAT machine. The ITSM test is conducted for the measurement of small strains on bituminous mixtures by applying impulse loading on a vertical diameter of a cylindrical specimen.

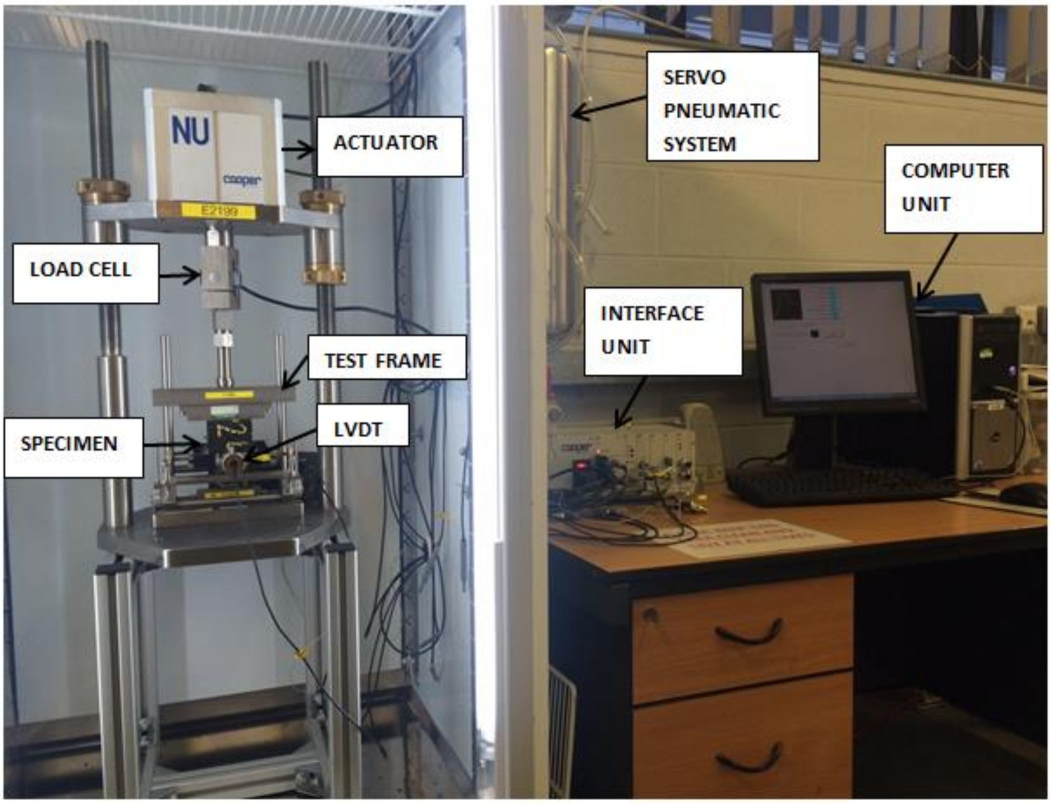


Figure 5-2: ITSM Test Configuration in the NAT

Target parameters for the ITSM test are stated below in Table 5-2.

Parameter	Protocol
Specimen thickness	30-75 mm
Rise time	124±4 ms
Target horizontal deformation	5±2 µm (100 mm diameter) 7±2 µm (150 mm diameter)
Pulse duration	3 seconds
Wave form	Haversine
Number of conditioning pulses	5 pulses
Test temperature	20±0.5°C
Poisson's ratio (assumed)	0.35

Table 5-2: Testing protocol for ITSM

5.3.2 Indirect Tensile Fatigue Test (ITFT)

The Indirect Tensile Fatigue Test (ITFT) was used in ascertaining the fatigue characteristics of the CAEMs as it is simple and fast to use in comparison to other fatigue tests that include two point beam flexure, centre point beam flexure (three point bending) and four point flexure tests. The ITFT is able to adequately differentiate between different mixture types. Rahman (2004) stated that ITFT simulates conditions in the field although the following limitations with the use of the test are highlighted:

- Inability to vary the ratio of vertical and horizontal components of the biaxial stress conditions resulting in the incapability of the test to replicate stress state conditions at critical points within the pavement.
- Stress reversal is not possible thereby resulting in undesirable accumulation of permanent deformation.
- When the principal stress is used as the damage determinant, the ITFT test tends to underestimate the actual fatigue life (Read, 1996).

The test was performed using the Nottingham Asphalt Tester (NAT) NU-14 machine in accordance with BS DD ABF: 2000 and BS DD ABF 2003. The test set up is depicted in Figure 5-3. Test temperature used for the test was 20±1°C using 100±2 mm diameter specimens and a rise time of 124±10 milliseconds. The specimens were loaded repeatedly at a rate of 40pulses/minute until failure occurred by cracking or a vertical deformation of 9 mm was obtained. In most cases the specimens failed before they reach a vertical deformation of 9 mm.

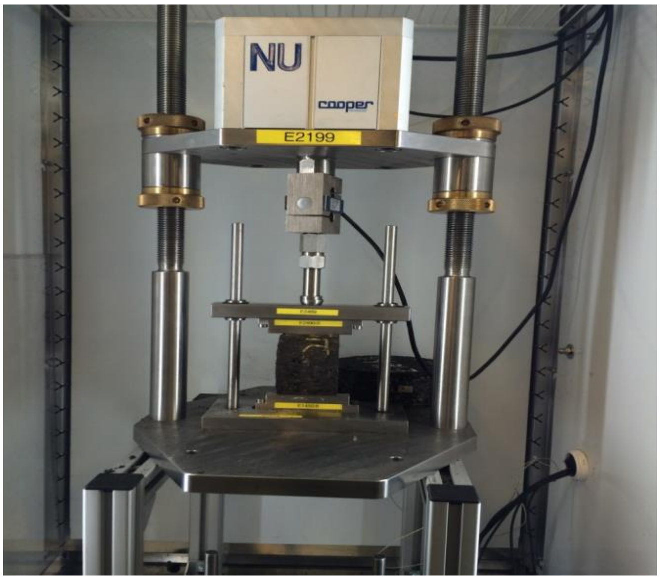


Figure 5-3: Test set up for Indirect Tensile Fatigue Test (ITFT)

The magnitude of the stresses varies along the diameter of the specimen but they are at a maximum at the centre (Rahman, 2004). Before the ITFT test was conducted, a stiffness test in stress controlled mode was conducted on the specimen at the required test stress level for the ITFT in order to ascertain the horizontal deformation and resultant stiffness values. Selection of stress levels for the ITFT was carefully done especially for the weaker specimens limiting the stress values to 300kPa and up to 500kPa for the stronger CAEMs.

Fatigue lines were obtained from plotting maximum tensile strain at the centre of the specimen against corresponding number of cycles to failure and linear regression analysis using the least square method was applied.

5.3.3 Vacuum Repeated Load Axial Test (VRLAT)

The Vacuum Repeated Load Axial Test (VRLAT) was conducted to assess the permanent deformation characteristics of the CAEMs. The test was conducted in accordance with BS DD 226. This test method is a modification to the Repeated Load Axial Test (RLAT) developed at the University of Nottingham (Nunn et al., 2000). Nunn et al., (2000) stated that although the RLAT can differentiate between asphalt mixtures of the same composition comprising of different binders, it cannot differentiate between mixtures with different aggregate gradations. Oliver et al., (1996) stated that the main advantage of using the VRLAT test method over the RLAT is that the VRLAT can ascertain the influence of aggregate, gradation, size and shape in permanent deformation as well as the influence of different binders. Oke, (2010) stated that the test was found to better simulate permanent deformation.

The test involves applying a load pulse vertically to the specimen by a rolling diaphragm pneumatic actuator using compressed air which is controlled by a solenoid valve. The operation of this is controlled by a computer through a digital to analogue converter. The specimens are sealed in a rubber membrane and they are secured at both ends by O-rings that rest in purpose cut grooves around the perimeter of two specially designed platens. The upper plate is fitted with an outlet pipe in the base which connects via a pressure regulator and gauge to a vacuum pump. The test was conducted by applying a confining pressure of 50kPa. Figure 5-4 shows the VRLAT test set up.

Figure 5-4: Test set up for Vacuum Repeated Load Axial Test (VRLAT)

5.4 Stiffness and Strength Characteristics of the CAEMs

5.4.1 The Effect of Increasing RAP content

To achieve the objectives of the research, four proportions of RAP aggregate materials were used as categorised below:

- Category 1: 0% RAP (no RAP, 100% VA)
- Category 2: 50% RAP (50% RAP, 50% VA)
- Category 3: 85% RAP (85% RAP, 15% VA)
- Category 4: 95% RAP (95% RAP, 5% VA)

This section shows results obtained from conducting stiffness and strength tests. The average of four samples was used in obtaining a data point for the stiffness and strength values. The variability in the data is represented using error bars that show the standard deviation. The standard deviation gives an indication of how closely clustered the data points are around the mean values calculated. Figure 5-5 shows the average air void content of Categories 1-4 at 0%, 1% and 3% OPC.

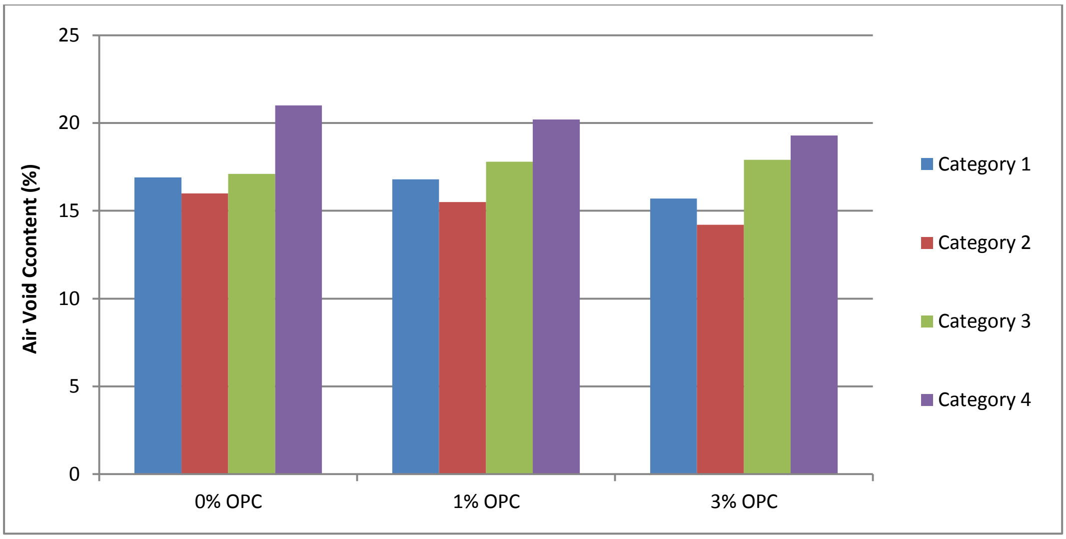


Figure 5-5: Average Air Void Content for Categories 1-4

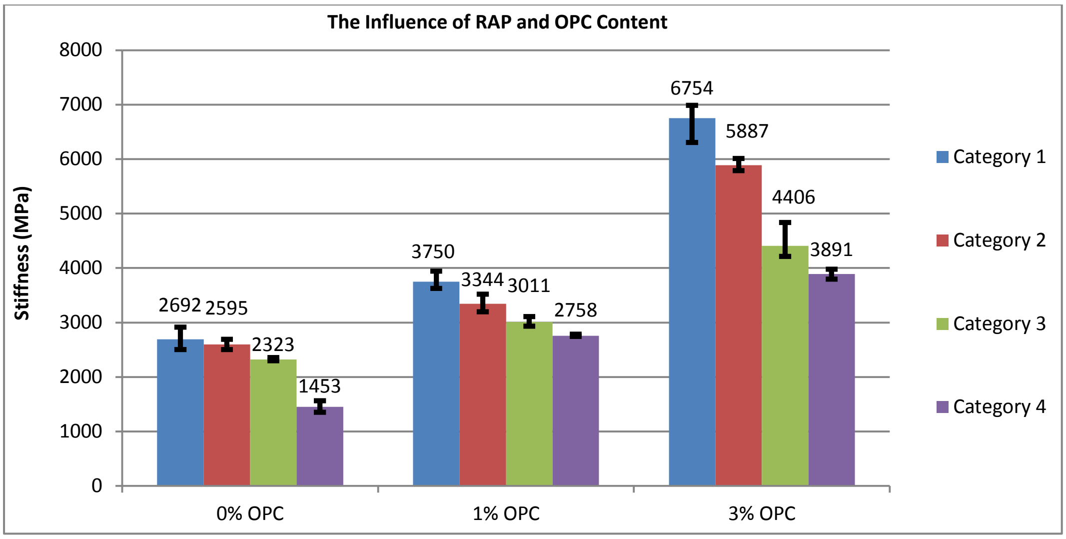


Figure 5-6: The Influence of Increasing RAP Content on Stiffness

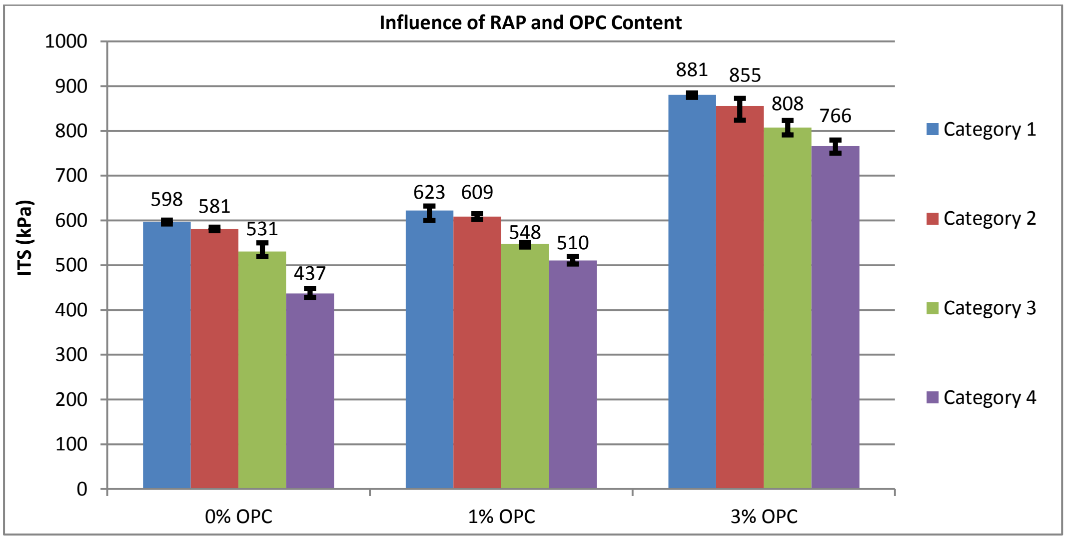


Figure 5-7: The Influence of Increasing RAP Content on Strength

Figure 5-6 and Figure 5-7 shows the stiffness and strength trend observed. The results show that at 0% OPC, the stiffness results for Categories 1 – 3 had very similar average values ranging between 2323 MPa and 2692 MPa. This was also observed for the strength tests with average values ranging between 531MPa and 598 MPa. Category 4 had the lowest average stiffness and strength values of 1453MPa and 437kPa respectively.

The air void content as depicted in Figure 5-5 has an influence on the stiffness and strength characteristics of the CAEMs. An increase in air voids results in a decrease in the stiffness modulus of the CAEMs and vice versa.

5.4.2 The Effect of Increasing OPC Content on the CAEMs

The trend observed was that the addition of 1% and 3% OPC significantly improved the stiffness and strength properties of the CAEMs. This was observed for Categories 1-4 as shown in Figure 5-6 and Figure 5-7. The addition of 1% OPC resulted in an average increase in stiffness of 32% for Categories 1-3 with Category 4 having the highest increase at 89%. The addition of 3% OPC more than doubled the stiffness properties of all the categories investigated. Substantial increase in strength was also observed. Average values ranged between 3891 – 6754MPa for stiffness and 766 – 881kPa for strength with the addition of 3% OPC. This increase in stiffness and strength could be attributed to the fact that the OPC is acting as a secondary binder. The rate of moisture loss is faster with the inclusion of OPC as seen in Figure 4-5 further facilitating the coalescence of binder in the CAEMs. Overall, Category 1: 100% VA had the best stiffness and strength properties as seen above followed by Categories 2 and 3. Category 4 had the lowest average values in stiffness and strength in comparison to the other CAEMs. This could be as a result of weaker aggregates in Category 4 due to its high RAP content in comparison to the other categories. This highlights the need for a suitable blend between RAP and VA in order to obtain optimal mechanical and performance properties when utilising CAEMs. The air voids also play an integral role as seen in Figure 5-5; Category 4 had the highest voids in comparison to Categories 1-3 at all OPC contents.

5.4.3 Moisture Susceptibility of the CAEMs

The study aimed to investigate the effects of water saturation and accelerated water conditioning on the CAEMs. Knowledge of the moisture susceptibility of CAEMs plays a fundamental role in the design and specifications for CAEMs as it provides information with respect to the durability of these mixture types. The study was limited to Category 2: 50%RAP, 50%VA and Category 3: 85%RAP, 15%VA at 0% OPC due to time constraints on the project. The test followed a system in which each set of the CAEMs was produced and divided into two equally sized subsets and conditioned. One subset was maintained dry after the standard curing regime of 20^{0}C for 28 days, unwrapped (UW) in the conditioning cabinet while the other subset was saturated and stored in water at elevated conditioning temperature as follows:

I. 40°C for 72 hours without vacuum saturation.
II. 40°C for 72hours with vacuum saturation.

Two wet conditioning procedures were followed. The first procedure was to account for the fragile nature of the CAEMs. It was initially thought that the application of a vacuum pressure of 6.7 ± 3 kPa applied to the CAEMs for 30 minutes would result in the collapse or disintegration of the specimens in the process while the second procedure applied the vacuum pressure of 6.7 ± 3 kPa as specified in the standard BS EN12697-12:2008. After conditioning, the Indirect Tensile Stiffness Modulus (ITSM) and Indirect Tensile Strength (ITS) for the dry and wet specimens were determined in accordance with BS EN 12697-26 and BS EN 12697-23 at the specified test temperature of 20°C. The ratio of the water conditioned subsets compared to the dry subsets was then determined and expressed as a percentage.

Figure 5-8 and Figure 5-9 detail the stiffness and strength responses for water susceptibility of Categories 2 and 3 CAEMs.

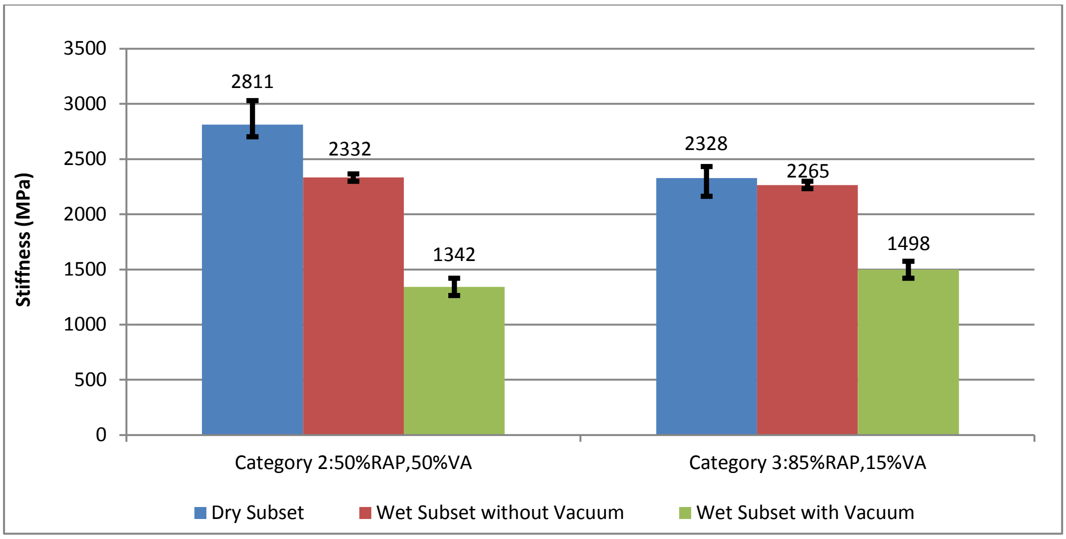


Figure 5-8: Stiffness Values for the Water Susceptibility Analysis

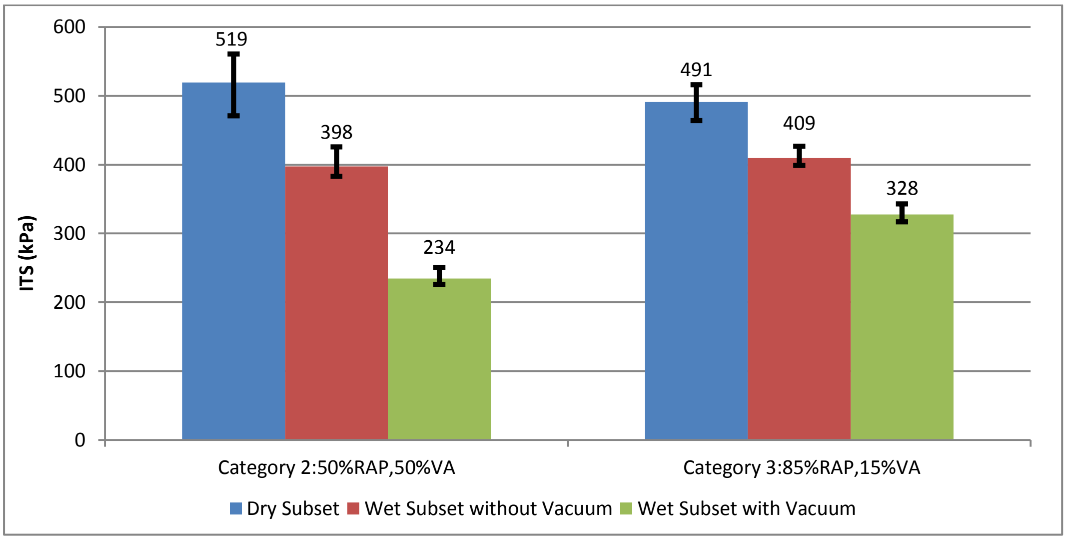


Figure 5-9: Strength Values for the Water Susceptibility Analysis

The results clearly show the effect of water saturation and accelerated water conditioning on the CAEMs. It was observed that there was a reduction in the stiffness and strength of the CAEMs as a result of water saturation at 40°C.

This reduction in stiffness and strength was further facilitated with the application of a vacuum pressure of 6.7 ± 3 kPa applied to the CAEMs for 30 minutes. This resulted in an average decrease in stiffness and strength of 54% for Category 2 and 34% for Category 3.

The ratios of the stiffness and strength of the water conditioned specimens compared to those of the dry specimens are expressed as a percentage and the results are depicted below in Figure 5-10 and Figure 5-11.

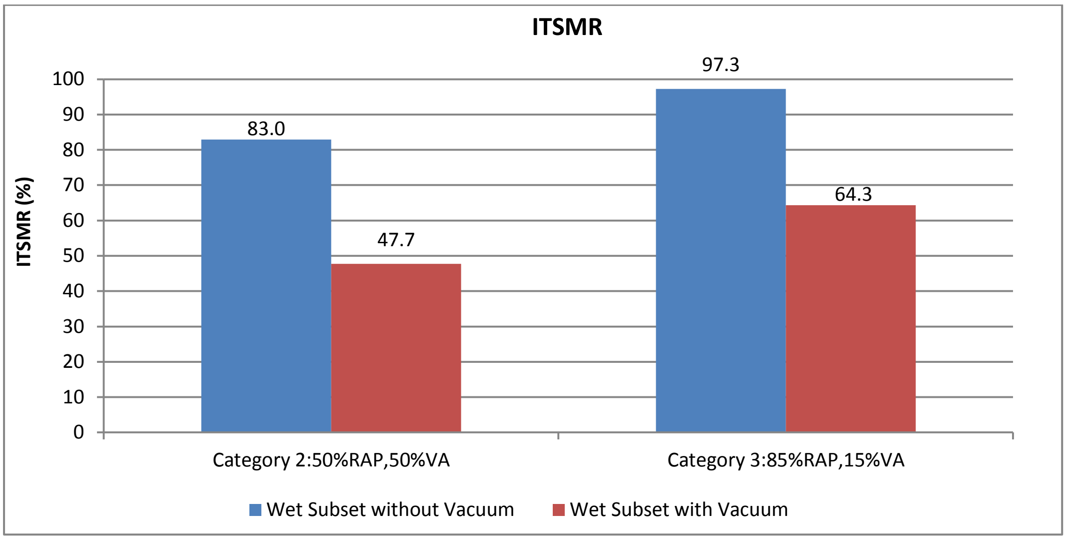


Figure 5-10: Stiffness Ratio for Water Susceptibility Analysis

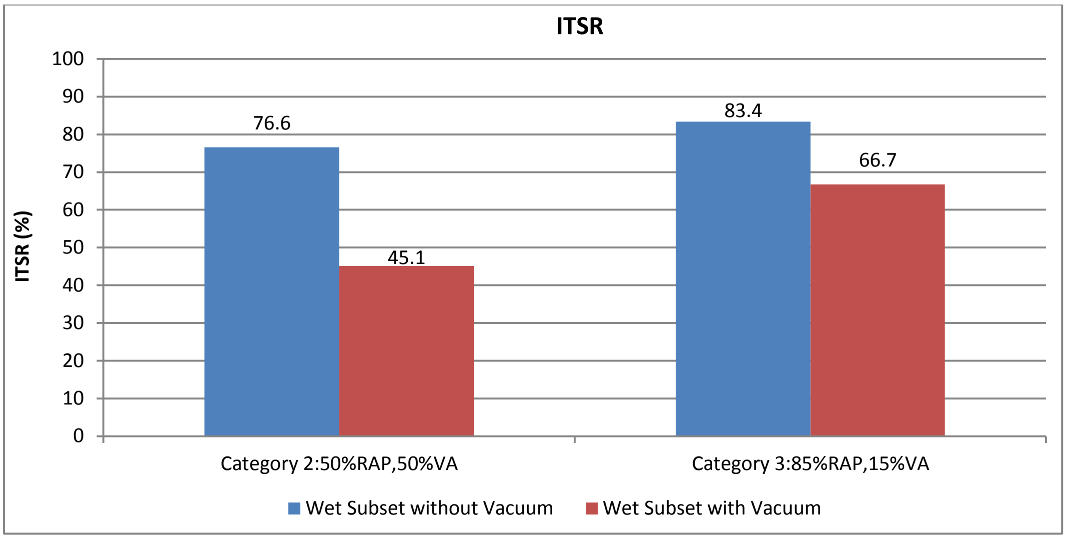


Figure 5-11: Strength Ratio for Water Susceptibility Analysis

Figure 5-10 and Figure 5-11 show that without the application of the vacuum pressure of 6.7 ± 3 kPa for 30 minutes on the CAEMs, the ITSMR and ITSR of the CAEMs were above 70% for Categories 2 and 3. The application of the vacuum pressure which resulted in saturating the CAEMs with water at 40°C resulted in a significant deterioration in the stiffness and strength of the CAEMs having an average ITSMR and ITSR of about 46% for Category 2 and 66% for Category 3.

Overall, Category 3 had better ratios in comparison to Category 2 although, it must be highlighted that the average stiffness and strength characteristics of Category 2 CAEMs was slightly better. Finally, it must be noted that the CAEMs used were produced at 0% OPC.

5.4.4 Effect of Mixing and Compaction Temperature

The effect of mixing and compaction temperatures was investigated in this study to gain more knowledge on how mixing and compaction temperatures of 5°C, 20°C and 32°C to simulate cold/wintery, temperate and tropical climatic temperatures could affect the mechanical and performance properties of the CAEMs.

This study focused on Categories 2 and 3 CAEMs at 0% and 1% OPC. The study was divided into two segments as stated below to account for the curing and conditioning of the CAEMs:

I. The CAEMs are conditioned, mixed and compacted at respective temperatures of 5°C, 20°C and 32°C. Curing of the CAEMs was at the standard curing regime of 20°C, UW for 28 days.
II. The CAEMs are mixed and compacted at temperatures of 5°C, 20°C and 32°C and then cured at the same temperature of 5°C, 20°C and 32°C, UW for 28 days.

Figure 5-12 and Figure 5-13 presents the average stiffness and strength results of the CAEMs mixed and compacted at 5°C, 20°C and 32°C to simulate various climatic conditions and cured at 20°C, UW for 28 days. The results show that an increase in the mixing and compaction temperature results in an increase in the stiffness and strength properties of the CAEMs. This was evident for both Categories 2 and 3.

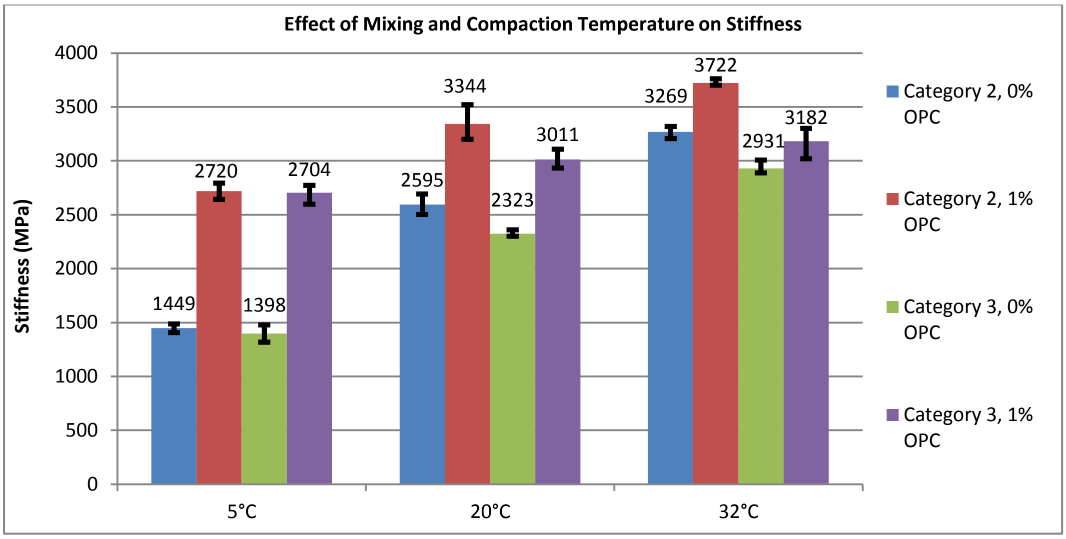


Figure 5-12: Effect of Mixing and Compaction Temperature on Stiffness at Standard Curing Conditioning

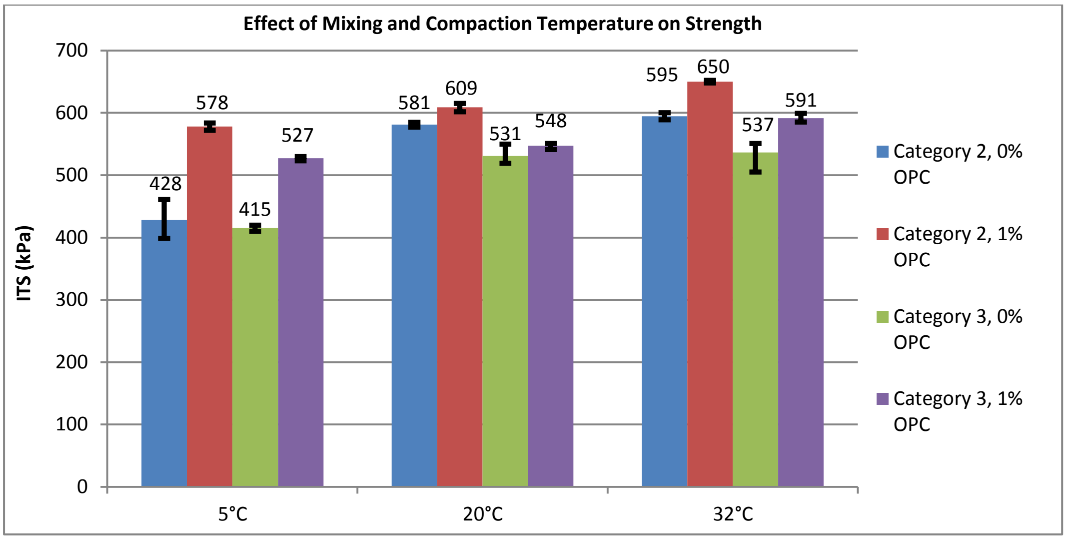


Figure 5-13: Effect of Mixing and Compaction Temperature on Strength at Standard Curing Conditioning

There was an average increase in stiffness of about 73% when mixing and compaction temperature was increased from 5°C to 20°C. It was observed that the increase in stiffness more than doubled when the mixing and compaction temperature was increased from 5°C to 32°C as depicted in Figure 5-12. Attributing factors include evaporation of moisture and better binder affinity to the aggregate at increased temperatures of 20°C and 32°C.

This increase was also observed for the CAEMs tested for its Indirect Tensile Strength (ITS). Figure 5-13 presents the average strength of the CAEMs. There was an increase in the strength properties of the CAEMs as the mixing and compaction temperature increased but it must be noted that the rate of strength increase was less than that observed for stiffness.

The addition of 1% OPC to the CAEMs resulted in an increase in the stiffness and strength properties of the CAEMs irrespective of the mixing and compaction temperature used. At mixing and compaction temperature of 5°C, the addition of 1% OPC resulted in equivalent stiffness and strength values to those at 20°C at 0% OPC. This highlights how the addition of cement could improve properties of CAEMs at cold/wintery climatic conditions. Segment II of the investigation involved curing and conditioning at their respective mixing and compaction temperature, UW for 28 days. Figure 5-14 and Figure 5-15 presents the average stiffness and strength results.

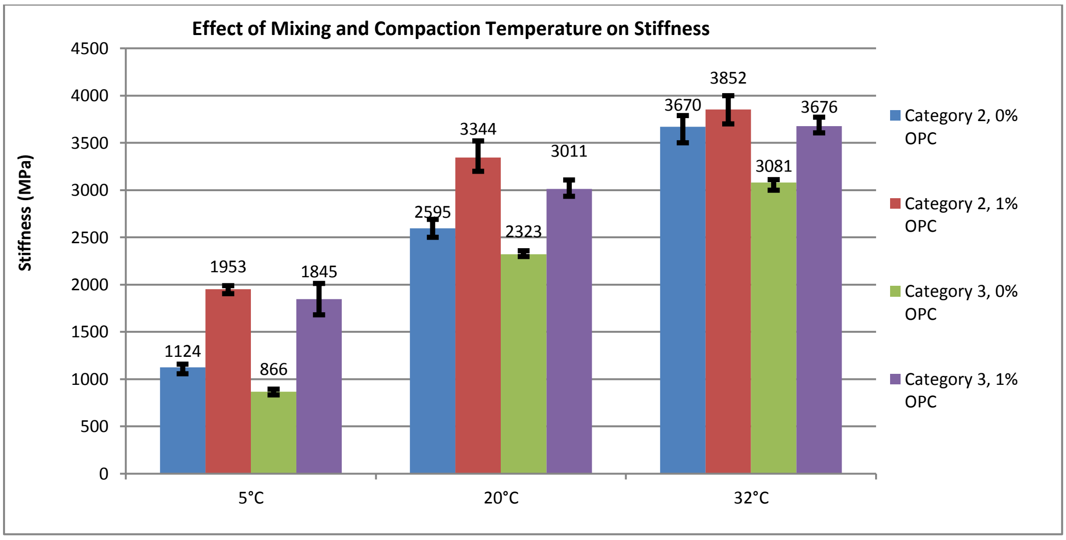


Figure 5-14: Effect of Mixing and Compaction Temperature on Stiffness and Curing Conditioning at Respective Mixing and Compaction Temperatures, UW for 28 days

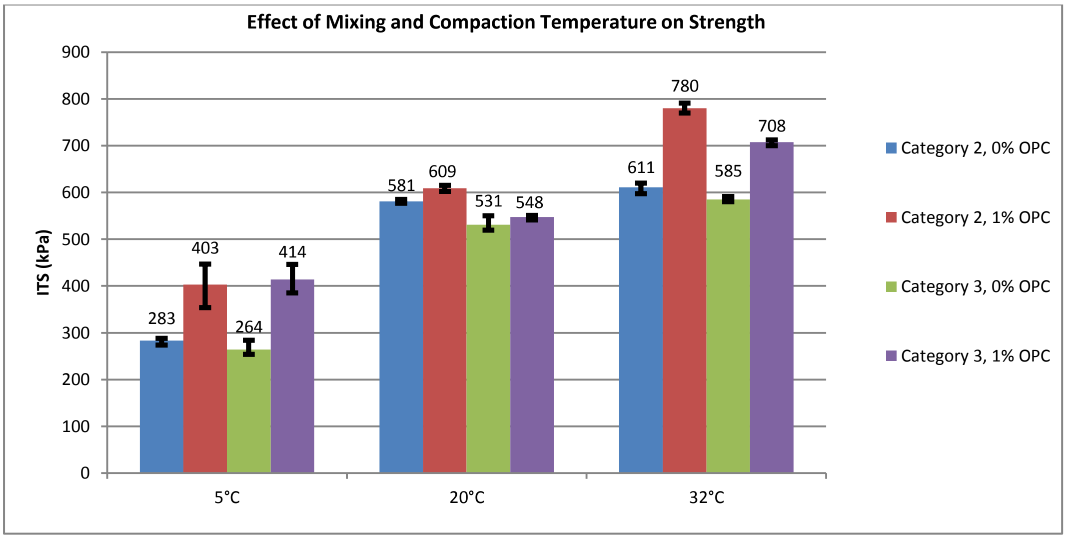


Figure 5-15: Effect of Mixing and Compaction Temperature on Strength and Curing Conditioning at Respective Mixing and Compaction Temperatures, UW for 28 days

The results as displayed in Figure 5-14 and Figure 5-15 show that when the CAEMs were mixed and compacted at 5°C, 20°C and 32°C with curing and conditioning at the same temperatures, UW for 28 days, similar trends for Segment I above were observed, i.e. an increase in the stiffness and strength of the CAEMs when the temperature is increased.

Furthermore, it must be noted that the CAEMs mixed and compacted at 5°C for Segment II had the lowest average stiffness and strength values while the CAEMs at 32°C had the highest average stiffness and strength values overall. Of importance, this study highlights the limited stiffness and strength properties achieved under cold/wintery climatic conditions of 5°C further emphasising the link between moisture loss in the CAEMs and stiffness/strength gain of the mixtures. The study also shows the improvement in the stiffness and strength of the CAEMs when 1% OPC is added. This results in an improvement of the properties of the CAEMs regardless of the mixing and compaction temperature.

5.4.5 Stiffness and Strength Characteristics of the CAEMs after Long Term Curing

The long term stiffness and strength properties of the CAEMs were evaluated in this section. This involved producing Categories 2 and 3 CAEMs at 0% and 1% OPC. The curing process at this stage involved placing the CAEMs in the conditioning cabinet to cure at:

I. 20°C, UW for 365 days
II. 40°C, UW for 365 days

Figure 5-16 and Figure 5-17 depict average stiffness and strength results for Categories 2 and 3 CAEMs. At 0% OPC at 20°C, the CAEMs exceed 3000MPa and the strength values exceed 700kPa for both Categories 2 and 3.

The inclusion of 1% OPC to the CAEMs at 20°C resulted in very similar average stiffness and strength values to those at 40°C, 0% OPC for both Categories 2 and 3. Overall, the mixtures cured at 40°C for the long term produced the best performing CAEMs in stiffness and strength.

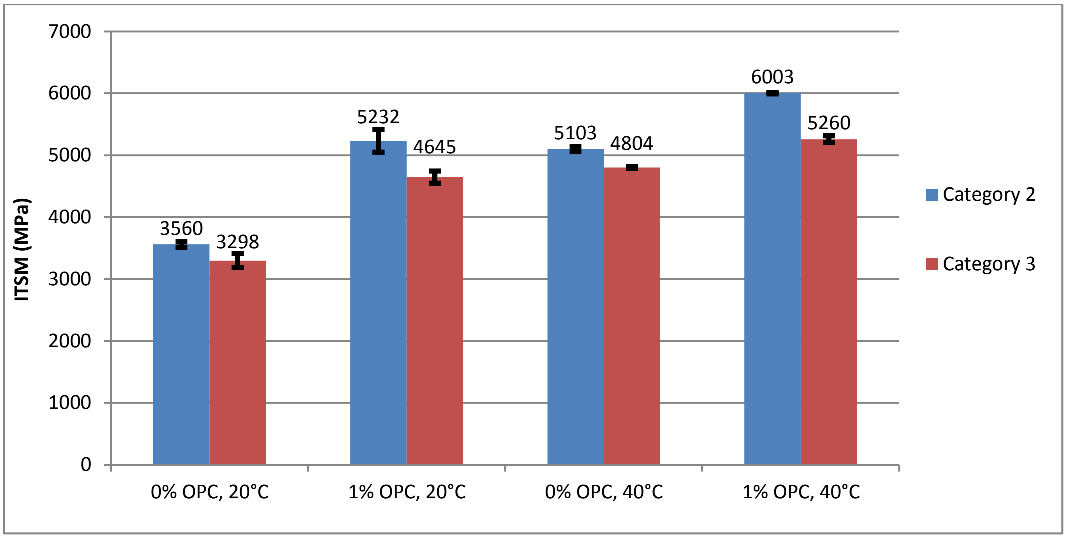


Figure 5-16: Stiffness Results for Long Term Curing of the CAEMs at 20°C and 40°C

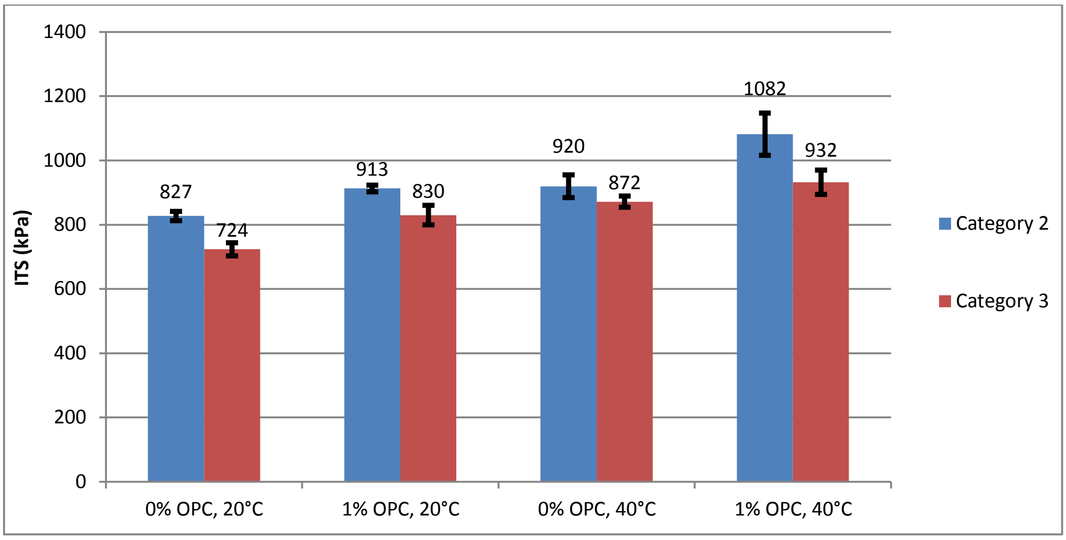


Figure 5-17: Strength Results for Long Term Curing of the CAEMs at 20°C and 40°C

5.5 Resistance to Permanent Deformation of the CAEMs

The Vacuum Repeated Load Axial Test (VRLAT) was used in assessing the permanent deformation characteristics of the CAEMs. The main advantage is that the test can ascertain the influence of aggregate, gradation size and shape as well as the influence of different binders as stated by Oliver et al., (1996).

The test was conducted in this study as it was found to better simulate permanent deformation characteristics in service as shown by Oke (2010) and provided sufficient means to rank the CAEMs on the basis of resistance to permanent deformation as a guide to relative performance in the pavement. Figure 5-4 shows the VRLAT test mode in the NAT machine. The test was conducted in accordance with BS DD 226. The major parameters include:

- Conditioning Stress: 10kPa
- Conditioning Period: 600s
- Confining Pressure: 50kPa
- Test Stress: 100kPa
- Test Duration: 1800 pulses
- Test Temperature: 30°C

Two specimens were tested and the average results are presented and analysed below from Figure 5-18 to Figure 5-20.

Gibb, (1996) stated that there are 3 major phases for full permanent deformation tests to failure. Phase 1 is termed the primary creep phase which is characterised by a rapid increase in vertical strain accumulation and densification of the specimens. Phase 2 is termed the secondary creep phase and this is characterised by a constant strain rate in the mixture. The third phase is termed the tertiary creep phase in which the specimen experiences a rapid increase in vertical strains until the failure of the specimens. Due to the limited number of cycles used in the test, the first and in some cases the second phases were generally observed for the CAEMs.

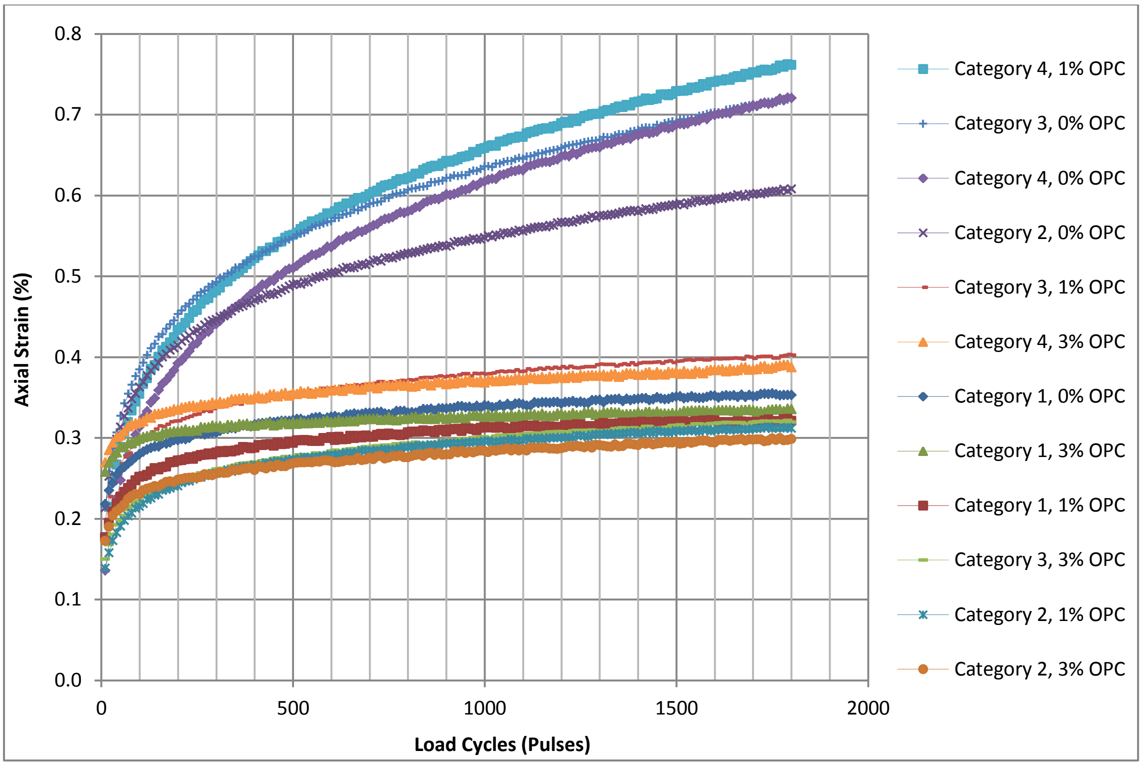


Figure 5-18: Influence of Increasing RAP and OPC Content on the Permanent Deformation Behaviour of the CAEMs

Figure 5-18 shows the total permanent strains after 1800 load cycles for the CAEMs. It is evident that the increase in OPC content resulted in better resistance to permanent deformation.

Overall, Category 2 had the best resistance to permanent deformation when OPC was added to the mixture. At 0% OPC, Category 1 had better resistance than the other CAEMs incorporating RAP due to a better inter-aggregate contact between the virgin aggregates playing a key role in resisting permanent deformation. Category 4 had the least resistance to permanent deformation at each level of OPC content in comparison to the other categories of CAEMs. This could be as a result of significant and rapid degradation of the quality of the high RAP aggregate content of the mixture under loading.

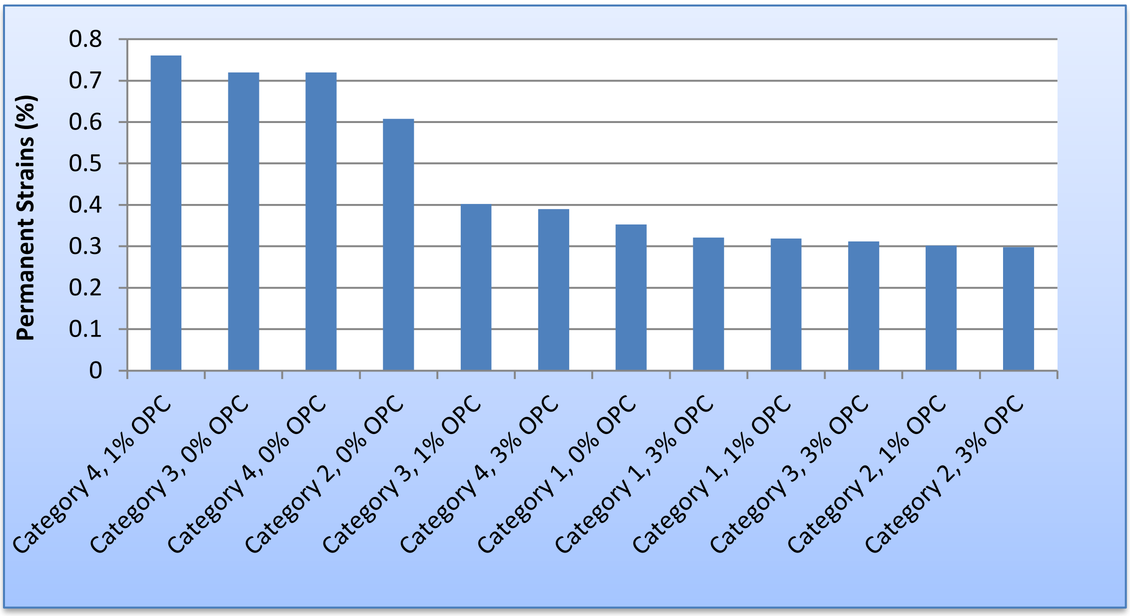


Figure 5-19: Permanent Strains of the CAEMs Showing the Influence of Increasing RAP and OPC Content

Figure 5-19 shows the permanent strains in the CAEMs showing the influence of increasing RAP and OPC content with Category 4 generally having the highest permanent strains and mixtures with 1% and 3% OPC having the least permanent strains. Figure 5-20 allows understanding of how mixing and compaction temperature influences the resistance to permanent deformation of the CAEMs.

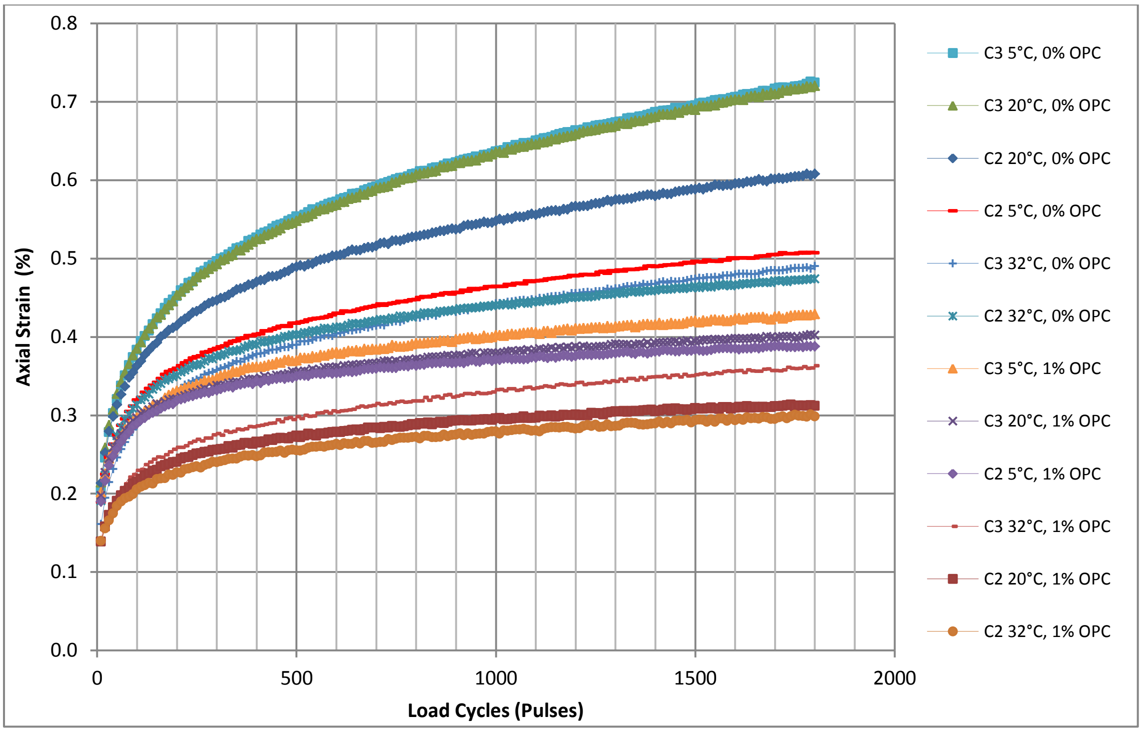


Figure 5-20: Influence of Mixing and Compaction Temperature on the Permanent Deformation Behaviour of the CAEMs at Standard Curing Regime

Categories 2 and 3 at 0% and 1% OPC were investigated at mixing and compaction temperatures of 5°C, 20°C and 32°C. The standard curing regime of 20°C, UW for 28 days was used. The results clearly show that an increase in the mixing and compaction temperature positively influences the deformation behaviour of the CAEMs. For Categories 2 or 3 at 0% or 1% OPC, it was evident that CAEMs mixed and compacted at 32°C had better resistance to permanent deformation in comparison to CAEMs at 20°C and at 5°C. Figure 5-20 further shows the influence of OPC content on the CAEMs. It can be seen that the mixtures incorporating 1% OPC had significantly better resistance to permanent deformation in comparison to mixtures without OPC. It should be noted that the resistance to permanent deformation results obtained for the CAEMs are good as all strains were less than 1%. Finally, the results are comparable to the observed stiffness and strength characteristics of the CAEMs as discussed in Section 5.4.

5.6 Fatigue Characteristics of the CAEMs

5.6.1 Introduction

Fatigue is one of the major failure mechanisms in bituminous road structures. The fatigue properties of the CAEMs were investigated in this study using the Indirect Tensile Fatigue Test (ITFT) as it was easy to use and relatively quick to conduct. The ITFT test is depicted in Figure 5-3.

The test was conducted for Categories 1 – 4 after the standard curing conditioning of 20°C, UW for 28 days. The Categories 1 – 4 CAEMs were produced at 0% and 1% OPC with fatigue responses of the CAEMs at 3% OPC investigated only for Categories 2 and 3.

The test was conducted at a temperature of 20°C in accordance with BS DD ABF: 2000 and BS DD ABF 2003. 60±3mm thick specimens were used for the test as it was found that trimming of the specimens could result in jeopardising their integrity. The test stress levels were regulated to a maximum of 325kPa for the CAEMs at 0% OPC due to their fragile state. At 1% OPC, this was limited to a maximum of 350kPa. The CAEMs at 3% OPC had a maximum test stress value of 750kPa.

The fatigue characteristics of a standard DBM 90 was included in the fatigue charts in order for comparisons to be made with the CAEMs. The results are presented below.

5.6.2 Fatigue Responses of the CAEMs

Table 5-3 details the fatigue responses of the CAEMs giving the equations for strain and cycles to failure. All R^2values exceeded 0.90 which shows a good data fit for the relationship except for Category 3 at 3% OPC that had an R^2 value of 0.85. The fatigue responses of the CAEMs are shown in Figure 5-21 to Figure 5-23.

OPC Content	Equation for Strain	Equation for Cycles to Failure	R^2 Values
Category 1: 100% VA			
0% OPC	$7764.8N_f^{-0.578}$	$5 \times 10^6\ \varepsilon^{-1.716}$	0.99
1% OPC	$659.32N_f^{-0.26}$	$2 \times 10^{10}\ \varepsilon^{-3.587}$	0.93
Category 2: 50% RAP, 50% VA			
0% OPC	$4191.6N_f^{-0.455}$	$6 \times 10^7 \varepsilon^{-2.111}$	0.96
1% OPC	$1295.7N_f^{-0.304}$	$5 \times 10^9\ \varepsilon^{-3.00}$	0.91
3% OPC	$364.21N_f^{-0.149}$	$8 \times 10^{16}\ \varepsilon^{-6.56}$	0.98
Category 3: 85% RAP, 15% VA			
0% OPC	$3633.4N_f^{-0.412}$	$3 \times 10^8\ \varepsilon^{-2.349}$	0.97
1% OPC	$1450.8N_f^{-0.318}$	$5 \times 10^9\ \varepsilon^{-3.022}$	0.96
3% OPC	$686.18N_f^{-0.207}$	$1 \times 10^{12}\ \varepsilon^{-4.131}$	0.85
Category 4: 95% RAP, 5% VA			
0% OPC	$5886.1.1N_f^{-0.473}$	$7 \times 10^7\ \varepsilon^{-2.054}$	0.97
1% OPC	$1260.5N_f^{-0.309}$	$2 \times 10^9\ \varepsilon^{-2.951}$	0.91

Table 5-3: Fatigue Responses of the CAEMs

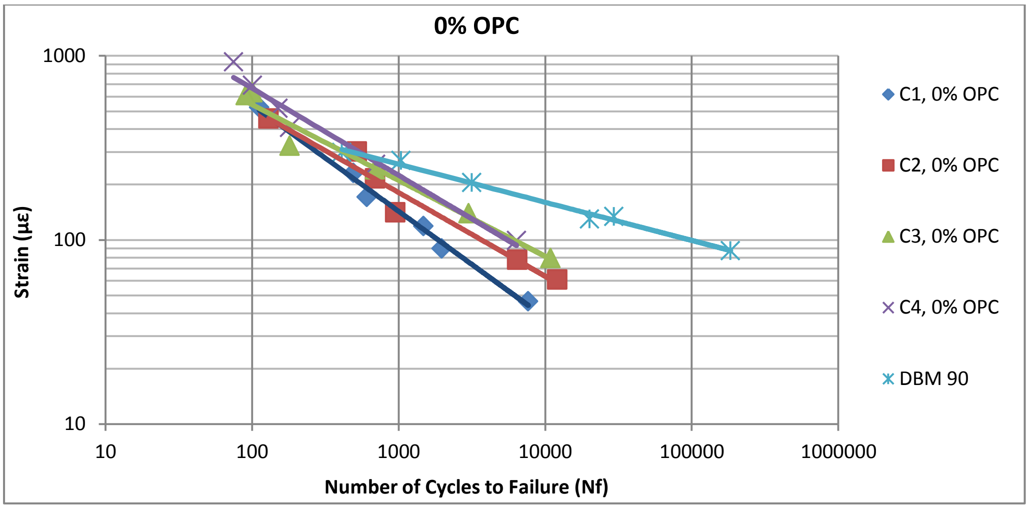


Figure 5-21: Fatigue Response of the CAEMs at 0% OPC

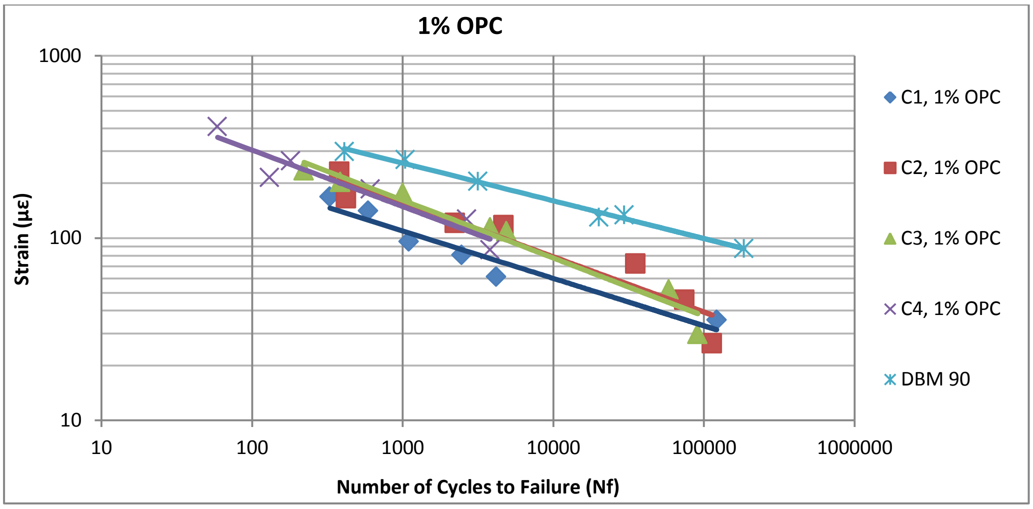


Figure 5-22: Fatigue Response of the CAEMs at 1% OPC

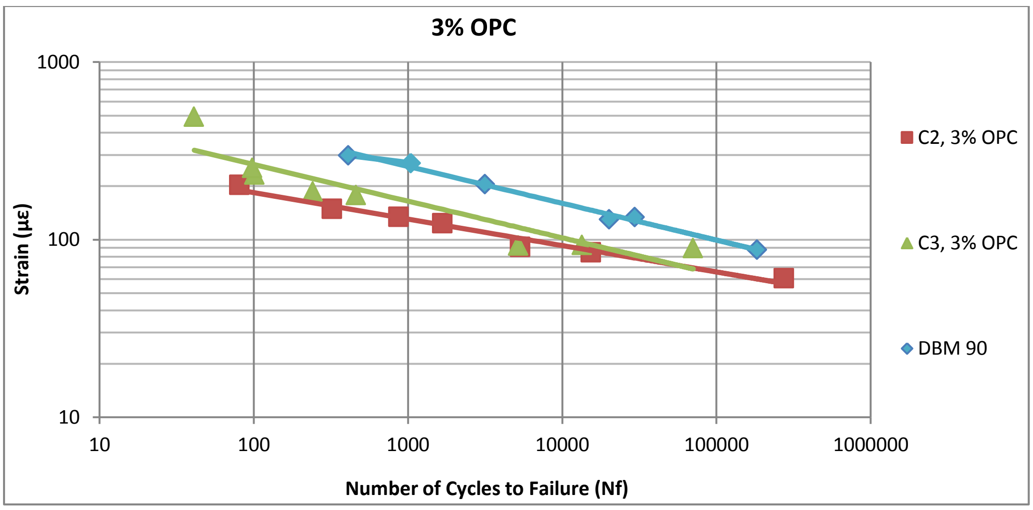


Figure 5-23: Fatigue Response of the CAEMs at 3% OPC

5.6.2.1 Influence of Increasing RAP content

Evaluating the data from Table 5-3, Figure 5-21 and Figure 5-22, it is evident that Category 1 CAEMs were most sensitive to stress changes and demonstrated the lowest resistance to fatigue. It was observed that an increase in RAP content resulted in an increase in the fatigue resistance of the CAEMs. It could be said that the residual binder content in the RAP CAEMs is having a positive influence on the fatigue properties of the mixtures. This trend was also observed by Ebels (2008) and Oke (2010). The aggregate structure of Category 4 having 95% RAP is excessive and degrading at a faster rate as the mixture had the lowest number of cycles in comparison to the other RAP CAEMs. The fatigue life of Category 1 CAEMs was short and performed poorly probably due to having a smaller volume of active binder in the mixture as shown in Table 3-17.

5.6.2.2 Influence of OPC Content

The addition of OPC content clearly resulted in an increase in the resistance to fatigue of the CAEMs. It can be seen that the fatigue lines of CAEMs with the addition of 1% OPC improved considerably in comparison to the mixtures at 0% OPC. It should be noted that the addition of 1% OPC to Category 4 mixtures resulted in brittle mixtures resulting in the specimens splitting very quickly once crack initiation occurred. At 3% OPC for Categories 2 and 3, the benefits were more pronounced. As seen in Figure 5-22 and Figure 5-23, the slope of the line decreased with increasing OPC content giving fatigue characteristics more similar with cement treated materials. This was also observed by Brown and Needham (2000).

Fatigue lives at strains of 30με, 50με, 100με and 200με were calculated. The results are stated below in Table 5-4.

OPC Content	Fatigue Life at Various Microstrains			
	30(με)	50(με)	100(με)	200(με)
Category 1: 100% VA				
0% OPC	14596	6075	1849	563
1% OPC	100600	16100	1340	111
Category 2: 50% RAP, 50% VA				
0% OPC	45703	15546	3599	833
1% OPC	185185	40000	5000	625
3% OPC	16337085	572594	6069	64
Category 3: 85% RAP, 15% VA				
0% OPC	101710	30634	6013	1180
1% OPC	171834	36701	4518	556
3% OPC	790701	95842	5470	312
Category 4: 95% RAP, 5% VA				
0% OPC	64728	22668	5459	1315
1% OPC	87507	19381	2506	324

Table 5-4: Fatigue Life of CAEMs at Various Microstrains

Read (1996) stated that fatigue failure in bituminous mixtures usually occurs in the range 30-200 με. Brown and Needham (2000) agreed stating that strain levels experienced in a pavement structure are likely to be below 200 microstrains with the actual value depending on variables that include mixture type, subgrade thickness, load and layer thicknesses. It is noted in Table 5-4 that at 200με, there is a drop in the number of cycles when comparing 0% and 1% OPC for CAEMs highlighting the brittle behavioural pattern of CAEMs with cement at elevated strains resulting in the quick splitting of the specimen soon after crack initiation. This phenomenon was also noticed with a considerable drop in the fatigue life of the CAEMs for Categories 2 and 3 at 3% OPC when the specimens experienced strains at 200με.

In summary, increased stiffness except at 200με resulted in increased fatigue life. Categories 2 and 3 performed characteristically better than Categories 1 and 4 at strains less than 200με within the context of this study. This could be as a result of the fact that Category 1 CAEMs probably had lesser volume of active binder in the mixture. It could also be stated that there are advantages for the addition of RAP in CAEMs with respect to the fatigue response as opposed to using 100% virgin aggregates as in Category 1 although with Category 4, there was a reduced resistance to fatigue possibly due to the aggregate structure having excessive RAP at 95% resulting in a faster rate of degradation at less than 200με. The inclusion of OPC in the CAEMs improved the resistance to fatigue at both 1% and 3% OPC at low strain levels. It was observed that strains in the CAEMs must be limited to less than 200με due to reduced flexibility in order to prevent the mixtures exhibiting brittle behavioural patterns resulting in the sudden split of the CAEMs soon after crack initiation.

5.7 Conclusions

Based on laboratory investigations and obtained results the following conclusions can be drawn:

5.7.1 Stiffness and Strength Characteristics

- The results showed that at 0% OPC, the stiffness results for Categories 1 – 3 had very similar average values ranging between 2323 MPa to 2692 MPa with no major discernible difference. This was also observed for the strength tests with average values ranging between 531MPa to 598 MPa. Category 4 CAEMs performed the worst with the lowest average stiffness and strength values of 1453MPa and 437kPa respectively.
- The trend observed was that the addition of 1% and 3% OPC significantly improved the stiffness and strength properties of the CAEMs. This was observed for all CAEMs. This was due to faster evaporation of moisture and better binder affinity to the aggregate at increased temperatures of 20°C and 32°C.
- The addition of 1% OPC resulted in an average increase in stiffness of 32% for Categories 1 – 3 with Category 4 having the highest increase at 89%.
- The addition of 3% OPC more than doubled the stiffness properties of all the categories investigated. Substantial increase in strength was also observed.
- Overall, Category 1: 100% VA had better stiffness and strength properties as seen above followed by Categories 2 and 3. Category 4 had the lowest average values in stiffness and strength in comparison to the other CAEMs.

This could be as a result of weaker aggregates in Category 4 due to its high RAP content in comparison to the other categories.

- This highlights the need for a suitable blend between RAP and VA in order to obtain optimal mechanical and performance properties when utilising CAEMs.
- Increase in the mixing and compaction temperature results in the increase in the stiffness and strength properties of the CAEMs.
- There was an average increase in stiffness of about 73% when mixing and compaction temperature was increased from 5°C to 20°C. This more than doubled when the temperature was increased from 5°C to 32°C
- The addition of 1% OPC to the CAEMs resulted in an increase in the stiffness and strength properties of the CAEMs irrespective of the mixing and compaction temperature used.

5.7.2 Deformation Properties

- Overall, Category 2 CAEMs had the best resistance to permanent deformation when OPC was added to the mixture. At 0% OPC, Category 1 CAEMs had better resistance to permanent deformation in comparison to the other CAEMs incorporating RAP.
- Category 4 had the least resistance to permanent deformation at each level of OPC content in comparison to the other categories of CAEMs. This could be as a result of significant and rapid depreciation of the quality of the high RAP content of the mixture under traffic loading.
- Category 4 generally had the highest permanent strains.

- The result clearly shows that an increase in the mixing and compaction temperature positively influences the deformation behaviour of the CAEMs.
- For Categories 2 or 3 at 0% or 1% OPC, it was evident that CAEMs mixed and compacted at 32°C had better resistance to permanent deformation in comparison to CAEMs at 20°C and at 5°C.

5.7.3 Fatigue Response

- It was apparent from the ITFT results that an increase in the test stress resulted in an increase in the strain levels experienced by the CAEMs.
- Category 1 CAEMs were most sensitive to stress changes and demonstrated the lowest resistance to fatigue.
- Category 2 and 3 CAEMs within the context of this study performed better than Categories 1 and 4 CAEMs at strains less than 200$\mu\varepsilon$.
- It could be said that the residual binder content in the RAP CAEMs is having a positive influence on the resistance to fatigue of the mixtures.
- The aggregate structure of Category 4 CAEMs having 95% RAP is excessive and degrading at a faster rate affecting the fatigue resistance of the mixtures.
- The inclusion of cement to the CAEMs improved the resistance to fatigue of the mixtures at both 1% and 3% OPC at strains less than 200$\mu\varepsilon$.
- It was observed that the strains in the CAEMs should be limited to less than 200$\mu\varepsilon$ due to reduced flexibility of the CAEMs in order to prevent the mixtures exhibiting brittle behavioural patterns resulting in the sudden split of the CAEMs soon after crack initiation.
- In all cases the DBM 90 had better fatigue characteristics as expected. The inclusion of 1% and 3% OPC to the CAEMs reduced the gap in fatigue response between the CAEMs and the DBM 90.

6. WHEEL TRACKING TEST

6.1 Introduction

The Wheel Tracking Test (WTT) was conducted in order to ascertain the susceptibility of the CAEMs to fail under loading. The WTT was conducted in a temperature controlled room in order to obtain standard and reproducible results. The test was conducted using general principles as stated in BS EN 12697-22: 2003 as a guide. The WTT is a widely used and accepted test for reproducing the effects of traffic over the surface of a bituminous layer. The WTT was conducted in order to reproduce as closely as possible conditions on the road in order to characterise and evaluate the failure mechanism of the CAEMs under set controlled conditions. Permanent deformation is one of the main failure modes in pavement structures resulting in the decline and deterioration of the pavement eventually leading to the fall in ride quality of the pavement. A schematic of pavement deformation is shown in Figure 6-1.

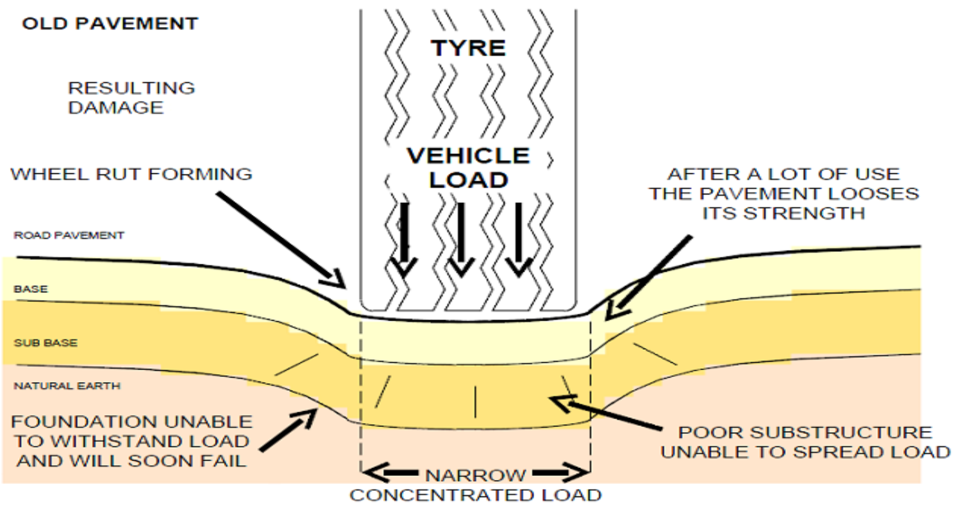


Figure 6-1: Schematic of Pavement Deformation (LGAM, 2014)

Mechanical loading as a result of traffic and climatic effects due to thermal variation in the pavement are two major types of loading that significantly affect pavements as stated by Perraton et al., (2011). Traffic loadings as a result of vehicular movements over the surface of the road after a given amount of repetitions eventually results in failure of the pavement. This is crucial and this chapter aims to investigate this phenomenon and present results of CAEMs incorporating high contents of RAP at 0%, 1% and 3% OPC with Hot Mix Asphalt (HMAs) overlays at varying thicknesses using the WTT. A schematic of the WTT equipment is depicted in Figure 6-2. The WTT was conducted on all categories of mixtures investigated in the research (Categories 1 – 4) at test temperatures of 5°C, 20°C and 32°C as shown in the experimental plan in Figure 6-3.

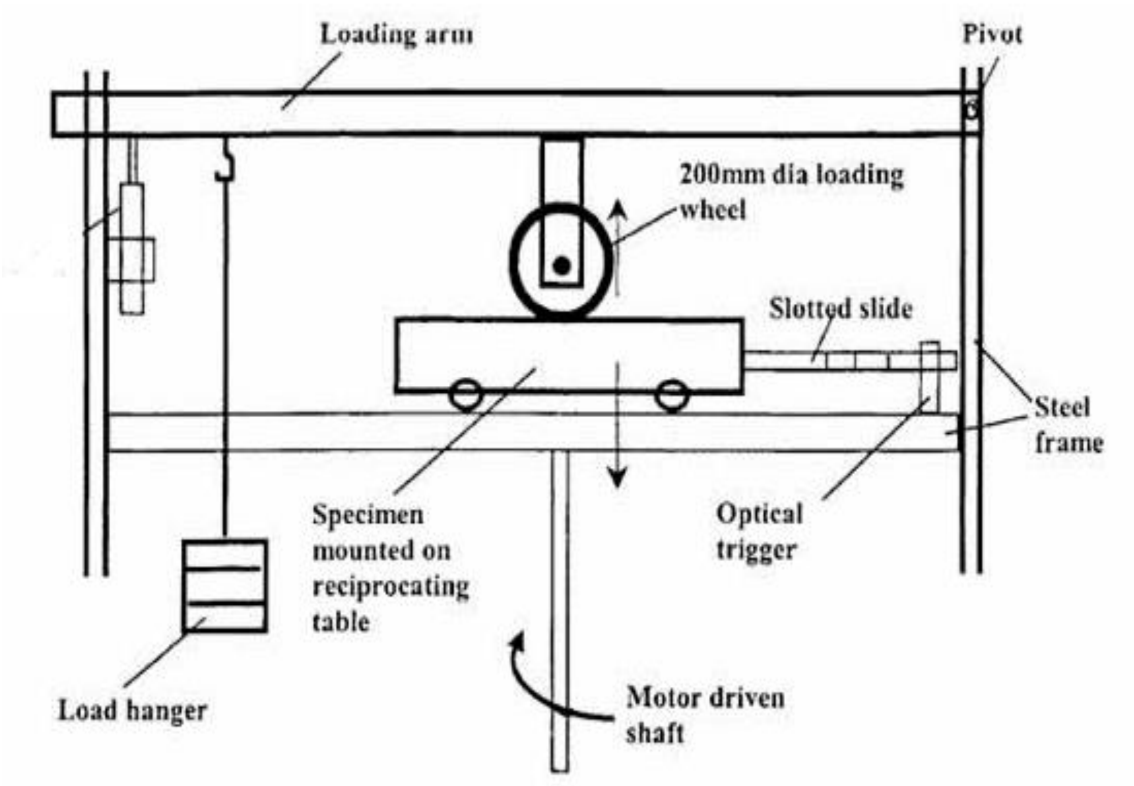


Figure 6-2: Schematic of the Wheel Tracking Test (Bodin et al., 2011)

6.2 Experimental Program

6.2.1 Introduction

Weidong et al., (2006) stated that the WTT is a reasonable solution in evaluating and giving an indication of the crack propagation and rutting performance of asphalt pavement structures as it reflects the cumulative permanent deformation in all the layers Bodin et al., (2012), Perraton et al., (2011), Ogundipe (2011) and Weidong et al., (2006) have all tested using the WTT to evaluate and give an indication of the failure mechanisms of asphalt pavements.

6.2.2 Materials

The materials for the specimens are as stated:

The base layer: The base layer comprised of the various categories of CAEMs (Categories 1-4) investigated in the research at 0%, 1% and 3% OPC with test temperatures of 5°C, 20°C and 32°C to simulate cold/wintery, temperate and hot/tropical climatic conditions. Three pavement structures were identified as appropriate to simulate various pavement structures and monitor pavement response under wheel loading. The effects of wheel loads at 1.35kN and 0.9kN were investigated. The pavement structures investigated are shown in Figure 6-3.

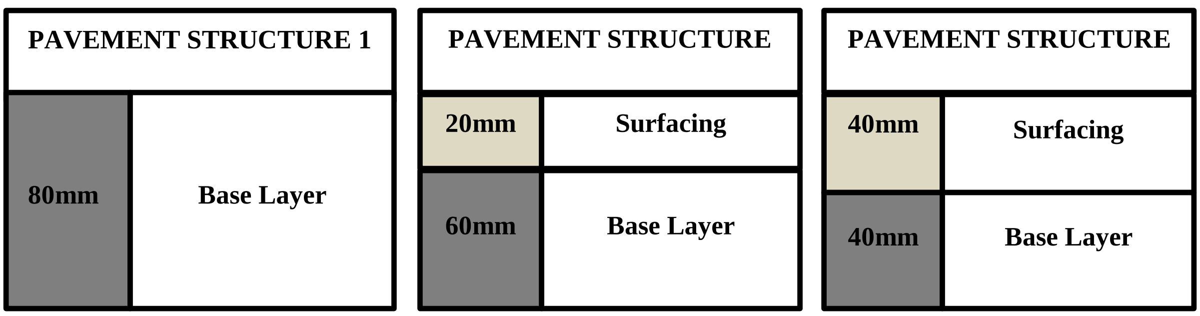


Figure 6-3: Pavement Structures Investigated for Wheel Tracking Test

Surfacing: The surfacing was a HMA top layer simulating the overlay. It was composed of 0/10mm Asphalt Concrete (AC 10 surf 70/100) with gradations as depicted in Figure 6-4 and Table 6-1 in accordance with specifications as stated in BS EN 13108-1:2006.

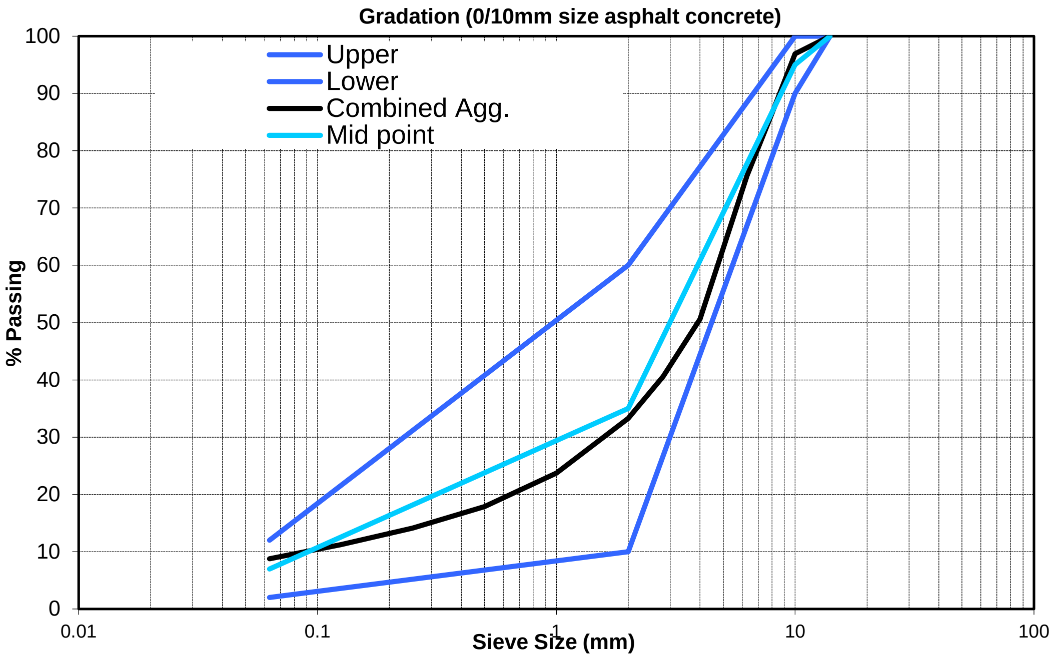


Figure 6-4: Aggregate Gradation for HMA Surfacing Overlay

The binder used for the HMA overlay had a penetration of 84dmm and a softening point of 44.6°C. The rotational viscosity at 160°C and 170°C are 88.2cP and 62.4cP respectively. The binder content for the mix was 5% and target air voids was 6%.

Foundation: The foundation comprised of a 12mm thick rubber mat was used in the WTT and this was found to be suitable in simulating an elastic foundation and induce a bending stress (Ogundipe, 2011). The stiffness of the rubber was measured as 9.5MPa. The rubber mat was a constant and always placed underneath the beam on the base of the steel mould on the reciprocating table of the WTT for all tests conducted.

Sieve Size (mm)	% Passing
14	100
10	96.9
8	86.6
6.3	75.6
4	50.6
2.8	40.6
2	33.3
1	23.8
0.5	17.9
0.25	14.1
0.125	11.2
0.063	8.8

Table 6-1: Aggregate Gradation for HMA Surfacing Overlay

Bonding: Tack coat was applied in between the CAEM layers and the HMA layer for the mixtures with overlays. The tack coat was applied in order to ensure adequate bonding between the layers and ensure that the pavement structure acts as a single unit thereby providing adequate strength. The tack coat was applied in accordance with BS 594987:2010. The standard states that the rate of spread for tack coats should be 0.2kg/m^2 of residual bitumen. The tack coat used was a cationic bitumen emulsion with residual binder content of 60%. The rate of spray for the tack coat was 0.33kg/m^2. To allow the emulsion break, the tack coat was applied uniformly and in a consistent manner at least 6 hours prior to the application of the HMA overlay.

6.2.3 Sample Preparation

The test specimens were made by manufacturing slabs of dimensions 305mm x 305mm. The thickness of the beams varied to replicate the various pavement structures investigated in the research study as stated in Figure 6-3.

The CAEM layer was produced and compaction was done using the roller compactor at room temperature and curing was at 20°C, UW for 28 days. This was followed by applying the tack coat 6 hours prior to the application of HMA overlays.

The aggregate materials and binder for the HMA overlay were heated to 185°C and compacted in a 305mm x 305mm mould to the required thickness levels depending on the pavement structure of which five beams of length 305mm and width of 50mm were obtained. Strain gauges were glued to the beam at the bottom side of the CAEM and HMA layers.

6.2.4 Test Procedure

The WTT was conducted in a temperature controlled room which facilitated the influence of test temperature at 5°C, 20°C and 32°C to be investigated in order to simulate cold/wintry, temperate and tropical climatic conditions. The wheel tracker moved in a forward and backward motion at a frequency of 0.8Hz under a loaded wheel with a travel length of 225mm passing on the top surface of the specimen. The outside diameter of the solid tyre fitted to the wheel was 200mm and the width was 50mm. Figure 6-5 shows the WTT with the wheel and attached strain gauges.

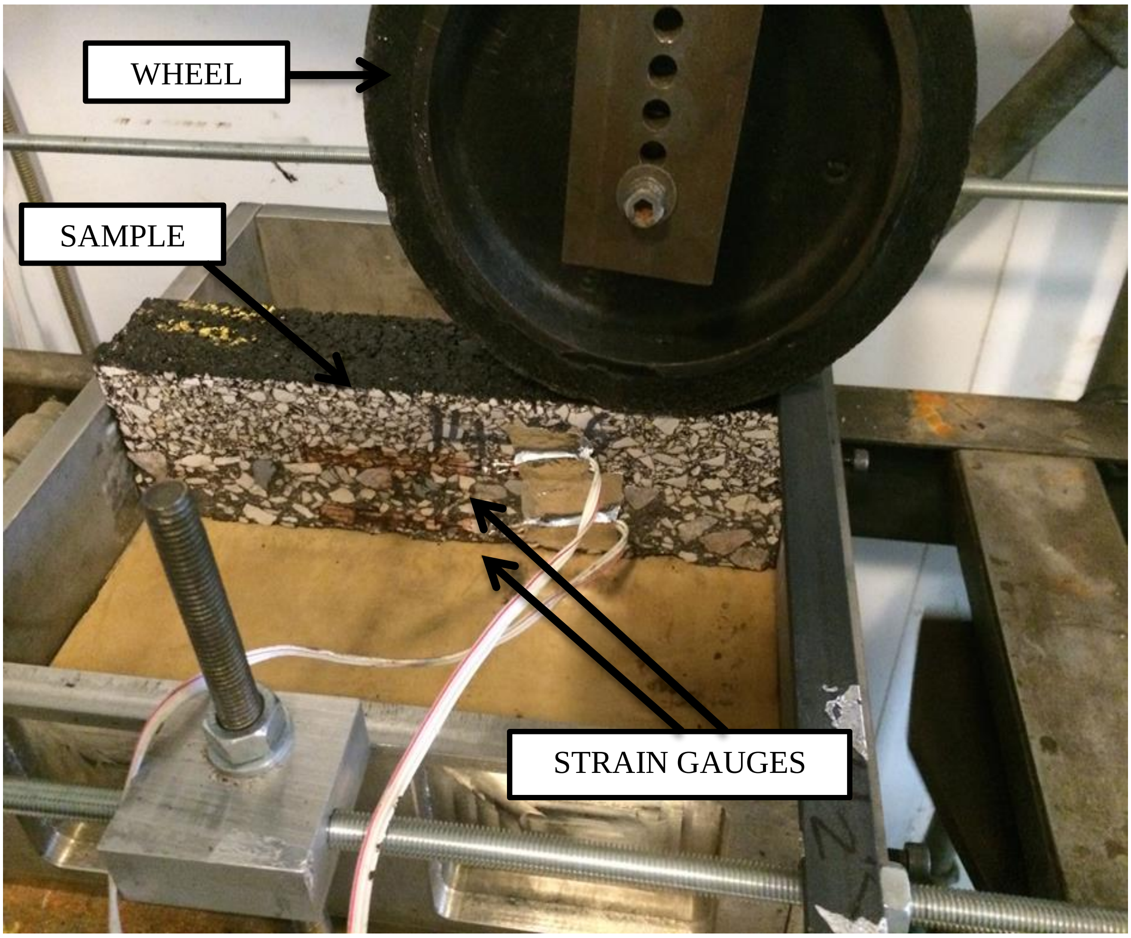


Figure 6-5: Cross Section of the WTT with Attached Strain Gauges

The test was carried out by placing the 12mm rubber mat on the base of the steel. The beams with the strain gauges were conditioned at the test temperature for a minimum of 4 hours in a conditioning cabinet prior to the test. The beams were placed centrally on the rubber mat in such a manner that the wheel loaded them symmetrically as the reciprocating table moved in a backwards and forwards motion. A typical beam for the WTT is shown in Figure 6-6.

The strain gauges were linked to the data acquisition system followed by releasing the tyres onto the beam at the test load.

The test load used was 1.35kN which is representative of tyre pressures on highways (Ogundipe, 2011) unless otherwise stated where the effects of a reduced load on the CAEMs at 0.9kN was investigated.

WTT tests are usually expressed based on the number of wheel cycles to failure. The number of cycles to failure was recorded and the strain levels at the bottom side of the CAEM base layer and the HMA surfacing layer were obtained from the readings on the attached strain gauges. An average of two beams was used to obtain a result for each condition tested.

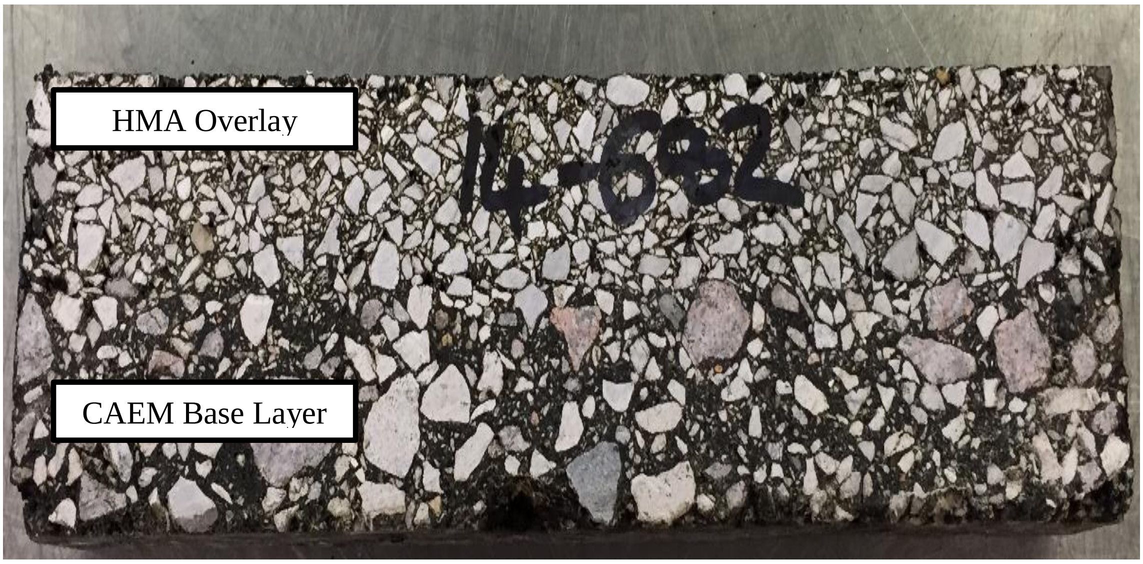


Figure 6-6: A Typical Beam for WTT

6.3 Typical Specimen Damage

6.3.1.1 80mm CAEM Base Layer

Figure 6-7 shows the onset of longitudinal cracks on the 80mm CAEM base layer with eventual disintegration of the material and complete failure as seen in Figure 6-8.

Figure 6-7: Onset of Longitudinal Cracks on 80mm CAEM Base Layer

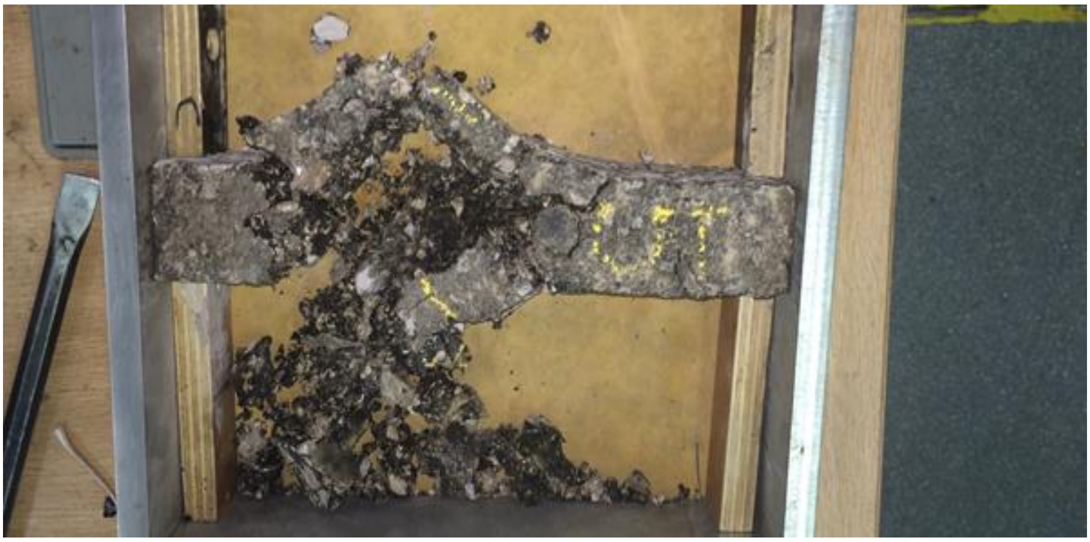


Figure 6-8: Total Failure of the 80mm thick CAEM Base Layer

6.3.1.2 60mm CAEM Base Layer + 20mm HMA Surfacing Layer

Figure 6-9: WTT for 60mm thick CAEM with 20mm HMA surfacing Layer

Figure 6-9 show significant bottom up crack propagation from the 60mm thick CAEM base layer progressing up to the bottom of the 20mm HMA surfacing layer. At Figure 6-10, significant deterioration of the CAEM base layer is visible with the HMA layer still intact and providing the required strength.

6.3.1.3 40mm CAEM Base Layer + 40mm HMA Surfacing Layer

Figure 6-10: WTT for 40mm thick CAEM with 40mm HMA surfacing Layer

6.4 Test Results for WTT

The results from investigating the effects of wheel tracking on the CAEMs are analysed and presented in this section. The influence of temperature, layer thickness, increasing RAP content, the effect of load and the influence of OPC on the CAEMs under wheel loading are discussed in this section. A typical strain reading analysis is shown in Figure 6-11 which shows readings from both the HMA and CAEM layer. The average of the trough readings (when the wheel load was at the edge of the beam) was subtracted from the crest readings (when the wheel was at the centre of the beam) in order to obtain the transient strains.

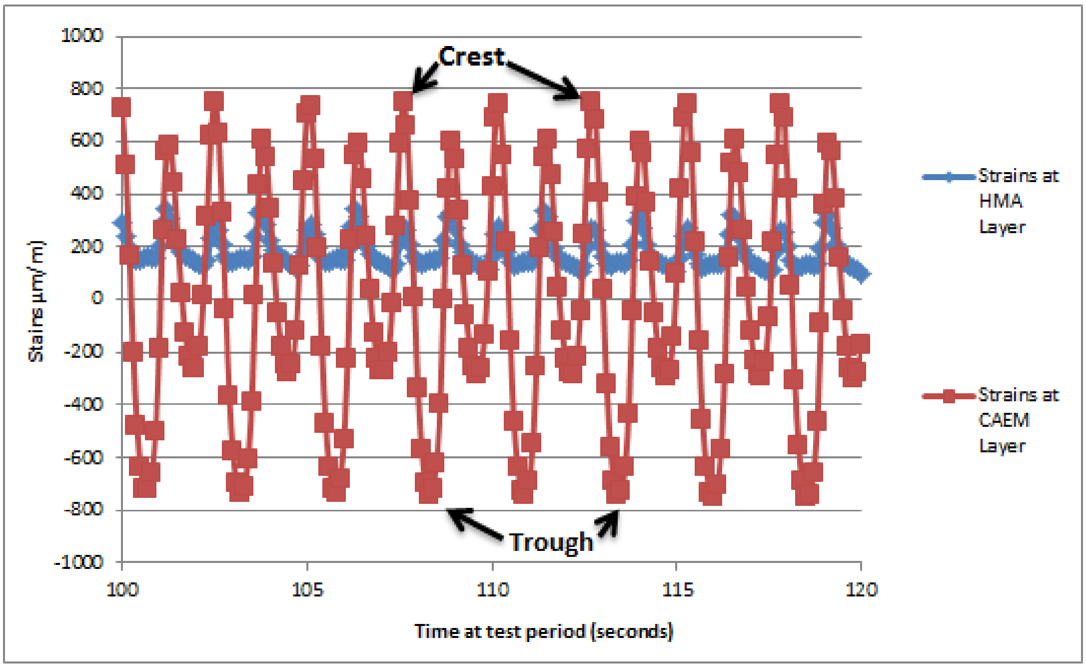


Figure 6-11: Typical Strain Reading Analysis

The transient strains and the number of cycles to failure provided vital information into the cracking mechanism and deformation characteristics of the CAEMs. The results are the average of two replicates tested in each case scenario.

6.4.1 The Influence of Temperature

Temperature plays a critical role in permanent deformation susceptibility of mixtures. For this study, the influence of temperature was investigated at 5°C, 20°C and 32°C with an applied load of 1.35kN. Figure 6-12 to Figure 6-15 show the transient strain level results under wheel loading for the CAEMs.

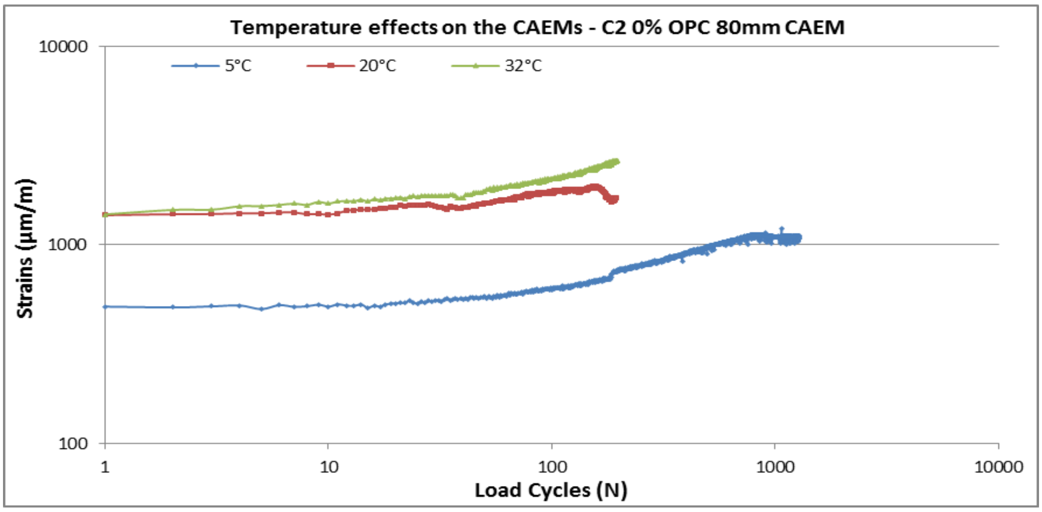


Figure 6-12: Strain Levels Under Wheel Loading for Category 2, 80mm thick CAEMs at 0% OPC

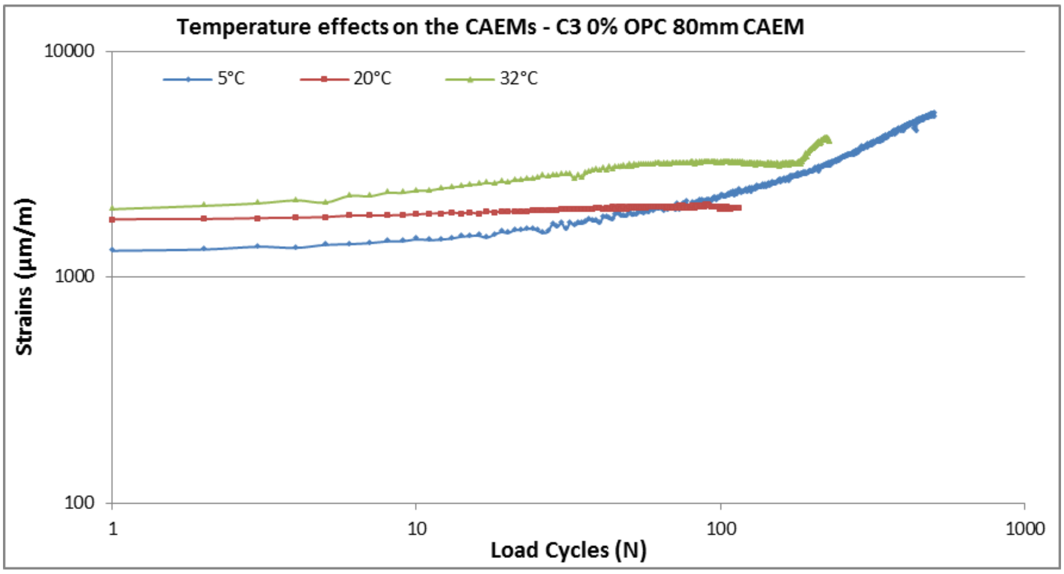


Figure 6-13: Strain Levels Under Wheel Loading for Category 3, 80mm thick CAEMs at 0% OPC

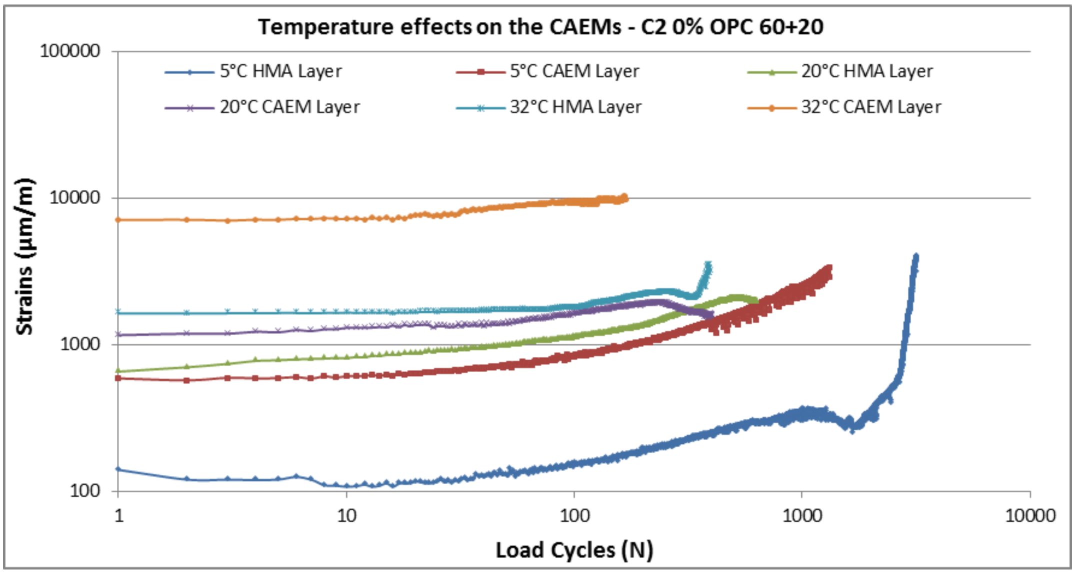


Figure 6-14: Strain Levels Under Wheel Loading for Category 2, 60mm thick CAEM Layer + 20mm thick HMA Layer at 0% OPC

Testing bituminous mixture reveals the presence of an irreversible, permanent deformation called visco-plastic deformation. From a microstructural perspective, visco-plastic deformation is related to the motion (sliding and rotation) of the mineral aggregates, bound together by the bitumen phase (Perraton, 2011).

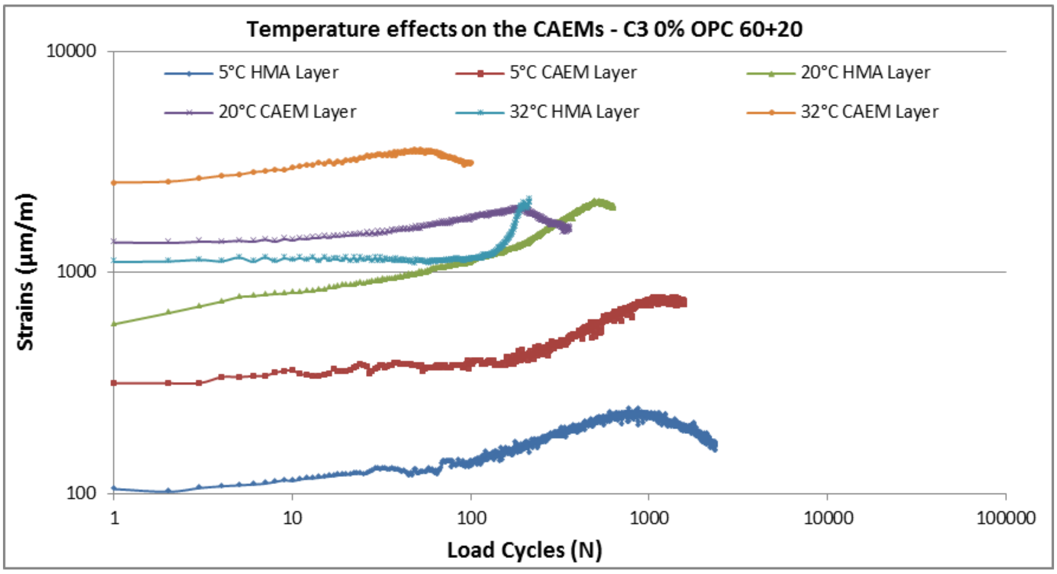


Figure 6-15: Strain Levels Under Wheel Loading for Category 3, 60mm thick CAEM Layer + 20mm thick HMA Layer at 0% OPC

The results show that as temperature increases, there is an increased susceptibility of the beams to fatigue cracking at multiple points usually starting from the CAEM layer propagating up to the HMA layer. This fatigue cracking propagates to the surface eventually leading to deformation of the beam. The lowest strains were observed for the beams tested at 5°C. The visco-plastic deformation is limited at low temperatures due to the high stiffness and strength of the beams at this temperature which restricts its flexibility limiting the ability of the beam to absorb strain concentrations.

The beams at 20°C and 32°C had very similar strain levels although it was observed that the specimens tested at 32°C were most susceptible to cracking propagation resulting in early failure of the beam in comparison to the beams tested at 20°C. This is due to the fact that at increasing temperatures, the shear strength of the beam reduces in response to the shear and tensile stresses generated by the wheel load creating an avenue for increased strains experienced as seen in Figure 6-12 to Figure 6-15 which eventual results in the rapid deterioration of the beam. At increased temperatures, the lubricating effect of bitumen increases and facilitates visco-plastic deformation in the beam.

It should be noted that in most cases the end of the strain on the plot does not always signify failure as sometimes the strain gauges failed before eventual failure of the beam. Figure 6-16 and Figure 6-17 show the number of cycles to failure of the beams investigated above. Mixtures tested at 5°C had the highest number of cycles to failure in comparison to the beams tested at 20°C and 32°C concurring to facts stated above.

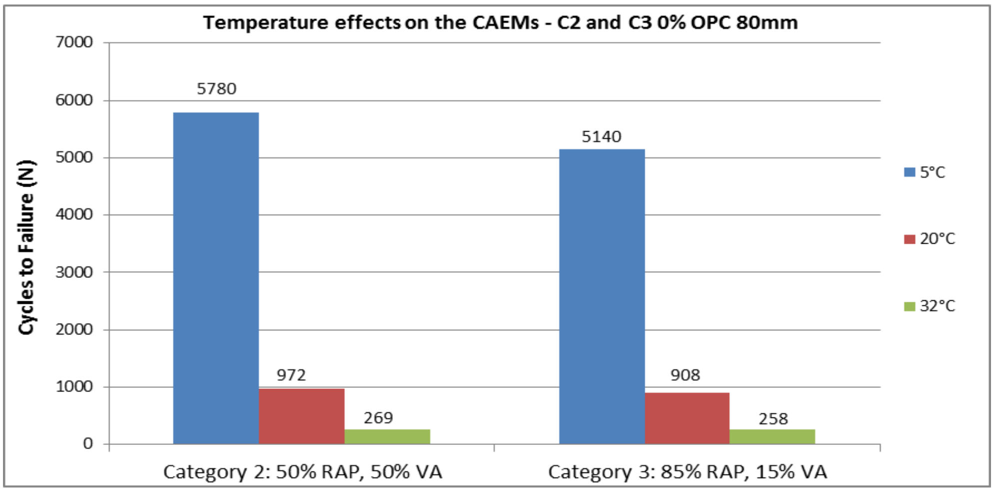


Figure 6-16: Cycles to Failure for Categories 2 and 3, 80mm thick CAEM Layers at 0% OPC

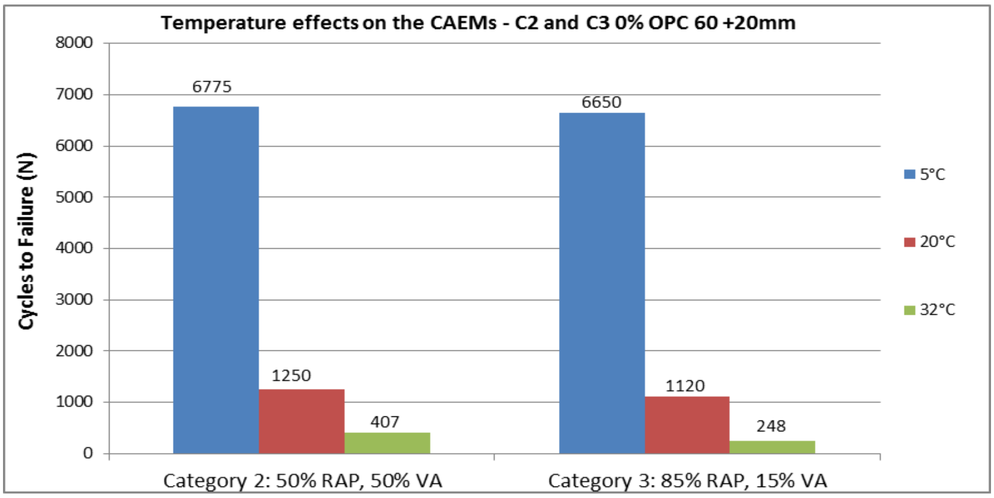


Figure 6-17: Cycles to Failure for Categories 2 and 3, 60mm thick CAEM Layer + 20mm thick HMA Layer at 0% OPC

In summary, the visible trend observed indicates that as the test temperature increases, there was an increase in strains experienced by the beams resulting in increased susceptibility of the beams to fatigue cracking and eventual disintegration of the beam. The specimens with less strain concentrations gave better performance than those with higher strain concentrations.

6.4.2 The Influence of Layer Thickness

In this study, to investigate the influence of overlay thickness, proportions as stated in Figure 6-3. The analysis considered and focused on the results of the tests carried out at 20°C for Categories 2 and 3 mixtures at 0% and 1% OPC as the trends observed and stated were identical.

The trend observed was that the mixtures with the thicker HMA layer (40mm thick CAEM Layer + 40mm thick HMA) had considerably lower strains at the bottom of the HMA overlay and thus performed better in terms of number of cycles to failure in comparison to the mixtures with thinner HMA layer (60mm thick CAEM Layer + 20mm thick HMA Layer) and no HMA layer (80mm CAEM). The results are presented from Figure 6-18 to Figure 6-21 for Categories 2 and 3 at 0% and 1% OPC.

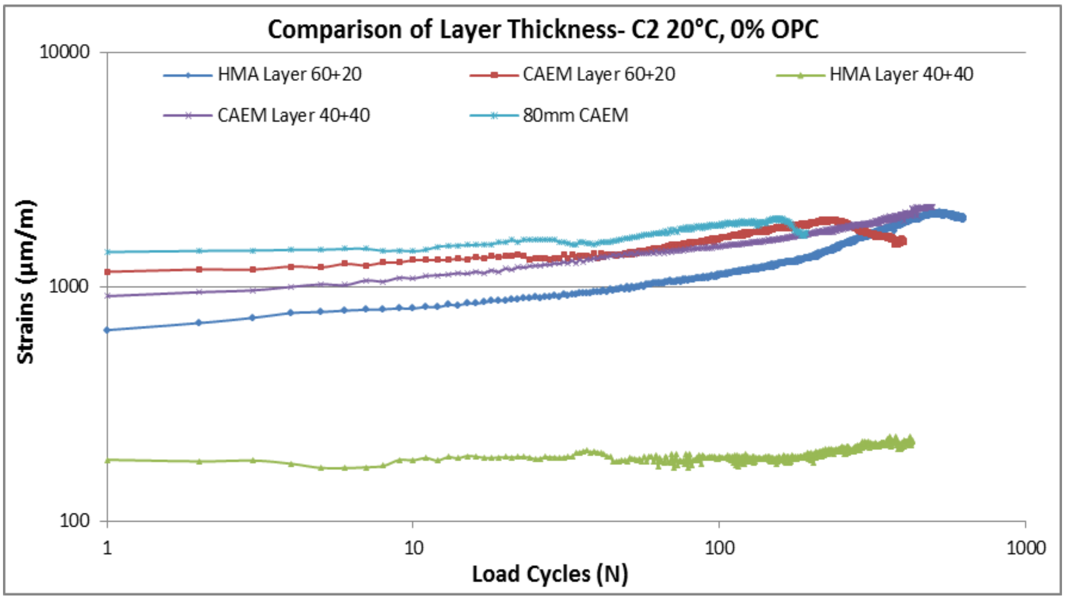


Figure 6-18: Strain Levels Under Wheel Loading Comparing the Influence of Layer Thickness for Category 2 at 20°C, 0% OPC

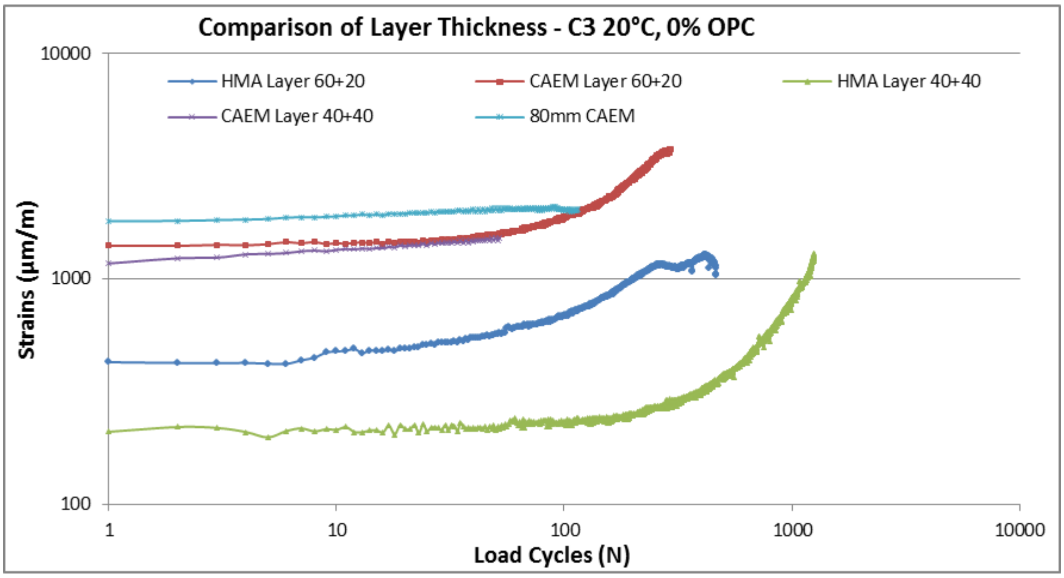


Figure 6-19: Strain Levels Under Wheel Loading Comparing the Influence of Layer Thickness for Category 3 at 20°C, 0% OPC

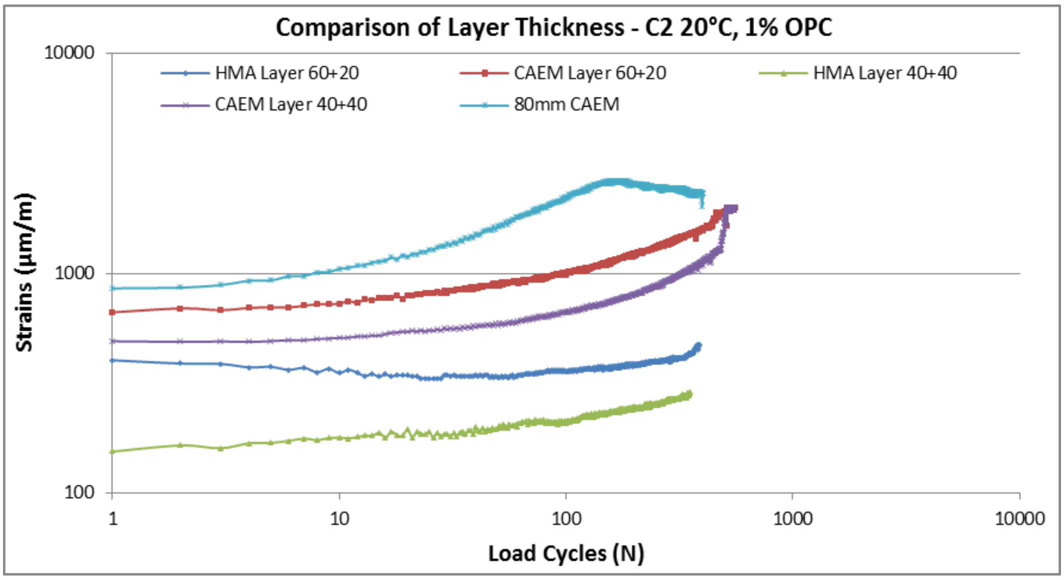


Figure 6-20: Strain Levels Under Wheel Loading Comparing the Influence of Layer Thickness for Category 2 at 20°C, 1% OPC

It was observed that the increased overlay thickness resulted in less tensile strains at the bottom of the HMA surfacing layer. At crack initiation mostly from the CAEM layer, the cracks had to propagate through more depth before reaching the surface of the HMA overlay.

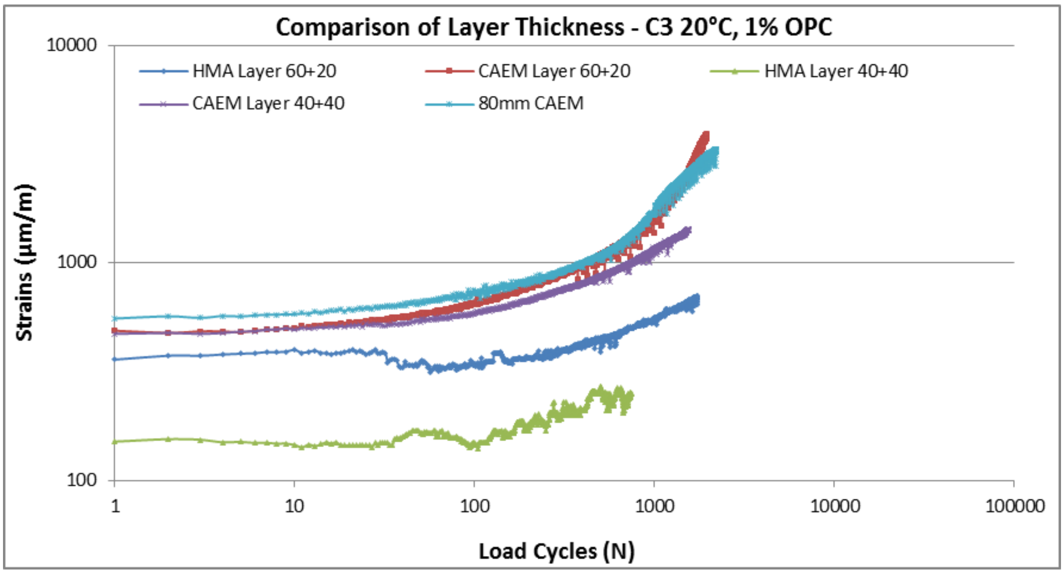


Figure 6-21: Strain Levels Under Wheel Loading Comparing the Influence of Layer Thickness for Category 3 at 20°C, 1% OPC

Figure 6-22 and Figure 6-23 shows the number of cycles to failures. The mixtures with the thicker HMA surfacing layer were more effective in delaying the crack propagation to the top of the surface layer resulting in a higher number of cycles to failure. The specimens with no HMA surfacing layer (80mm thick CAEM base layer) had the lowest number of cycles to failure in all case scenarios. Significant cracks propagated through the CAEMs which resulted in early life failure. The thickness of the HMA surfacing layer influenced the time to failure of the beams highlighting the need for the incorporation of optimally performing HMA surface layers above CAEMs with RAP.

Sanders (2001) stated that prediction of crack growth using this data type would be difficult and inaccurate. In most cases, the HMA layer was intact and in good condition while the underlying CAEM layer had significantly deteriorated with visible bottom up cracking at multiple locations rising up to the HMA layer. This trend was visible albeit at different rates of deterioration for Categories 2 – 4.

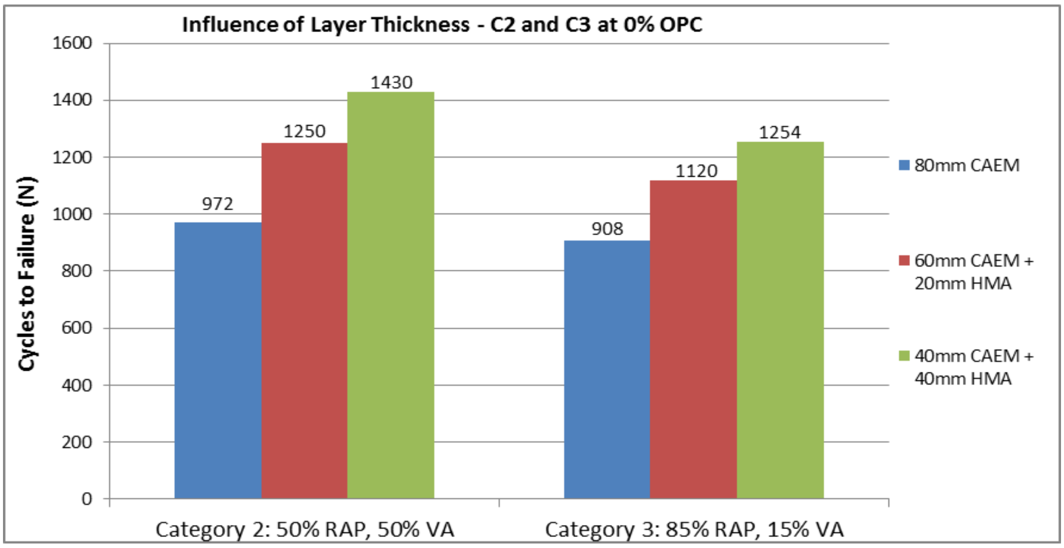


Figure 6-22: Number of Cycles to Failure for Categories 2 and 3 at 20°C, 0% OPC at Various Layer Thicknesses

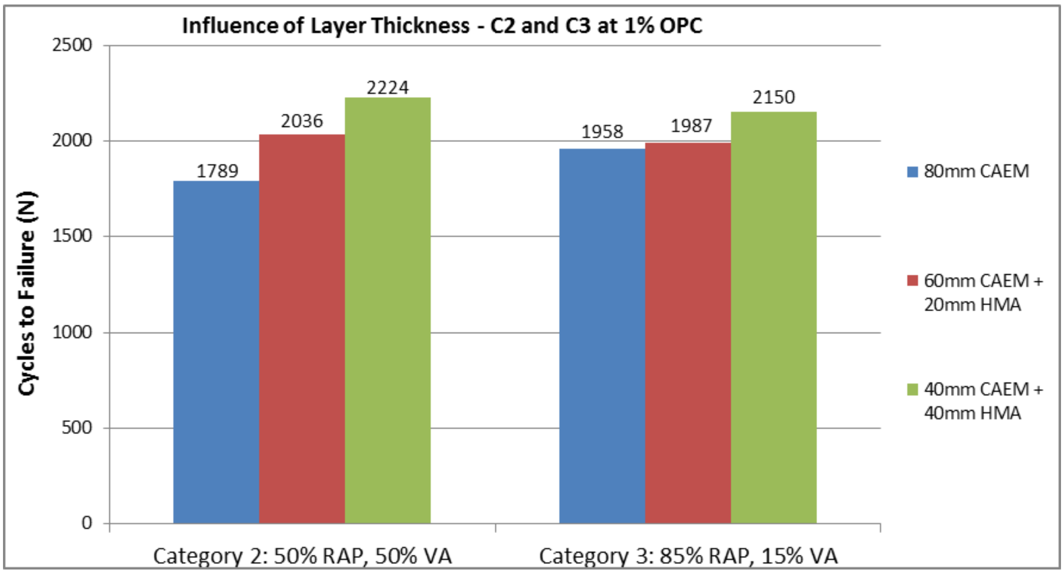


Figure 6-23: Number of Cycles to Failure for Categories 2 and 3 at 20°C, 1% OPC at Various Layer Thicknesses

6.4.3 The Effect of Increasing RAP Content

The effect of increasing RAP content was investigated using the four categories of mixtures studied that include:

- Category 1: 100% VA
- Category 2: 50% RAP, 50% VA
- Category 3: 85% RAP, 15% VA
- Category 4: 95% RAP, 5% VA

The results are presented only for 80mm CAEMs for the various categories of CAEMs at 0% and 1% OPC. The test was conducted at 20°C at 1.35kN load. The results showing the strain levels on the CAEMs under this loading are presented below in Figure 6-24 and Figure 6-25.

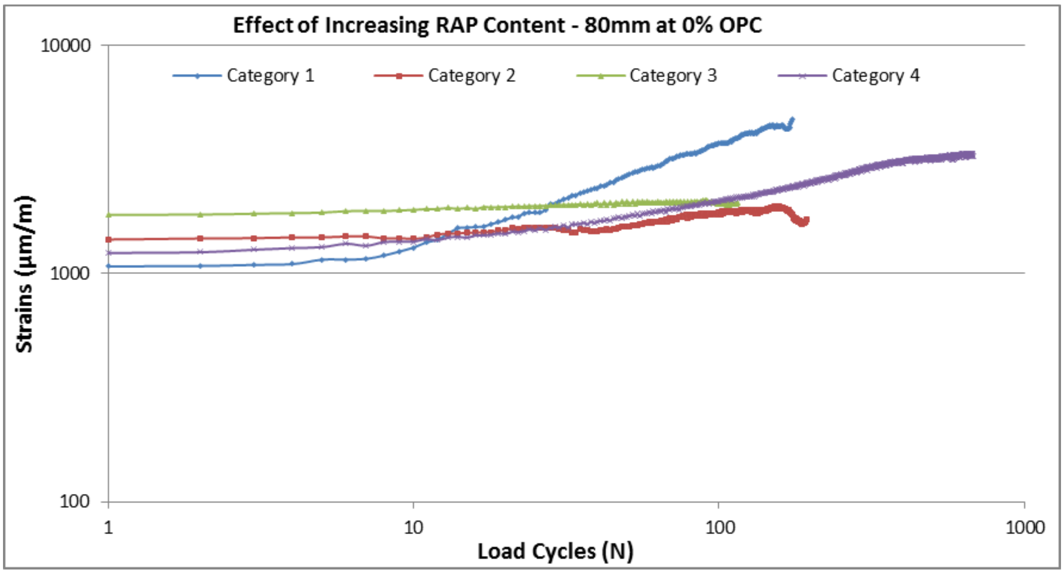


Figure 6-24: Strain Levels for Categories 1 – 4 for 80mm thick at 0% OPC

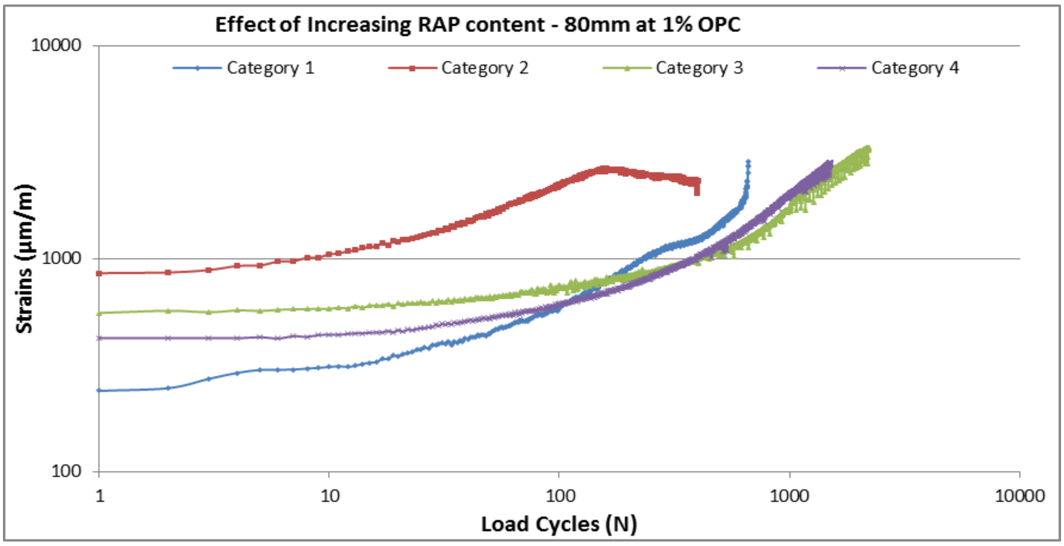


Figure 6-25: Strain Levels for Categories 1 – 4 for 80mm thick CAEMs at 1% OPC

Category 1 at 0% OPC had strains that continously increased which resulted in the failure of the beam after 203 cycles. This trend was also observed at1% OPC although failure of the beam occurred after 696 cycles as seen in Figure 6-26. At 0% OPC, the strains for Categories 2 and 3 were relatively level at strains of 1500 and 1900 which culminated in failure of the beams after 972 and 908 cycles respectively.

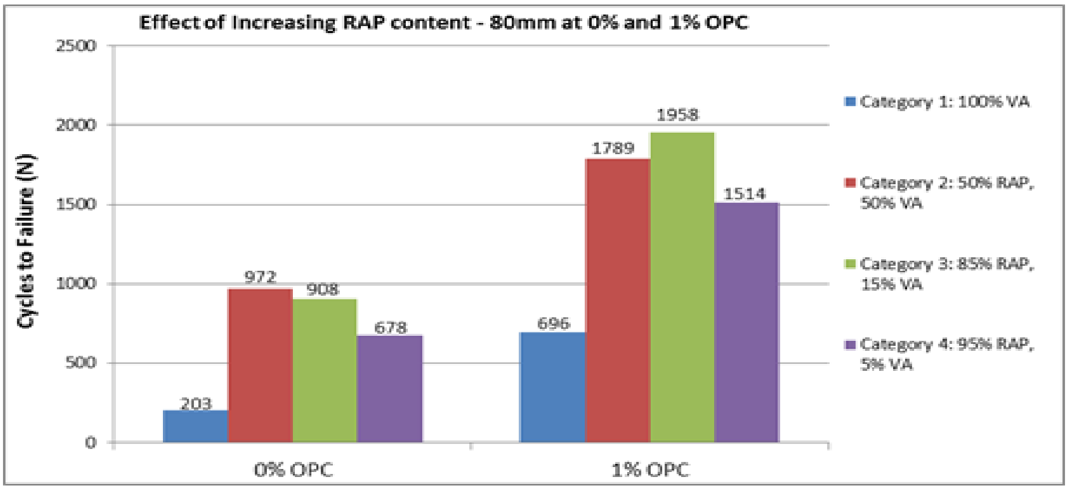


Figure 6-26: Number of Cycles to Failure for Categories 1 – 4 for 80mm thick CAEMs at 0% and 1% OPC

Category 4 had very similar trends to Category 1 as there was a rapid increase in strain levels observed although it had a higher number of cycles to failure in comparison to Category 1 but failed in fewer cycles in comparison to Categories 2 and 3.

Figure 6-27 and Figure 6-28 present the strains in the CAEM layer for 60mm thick CAEMs + 20mm thick HMA at 0% and 1% OPC while Figure 6-29 and Figure 6-30 present the results for 40mm thick CAEMs + 40mm thick HMA at 0% and 1% OPC.

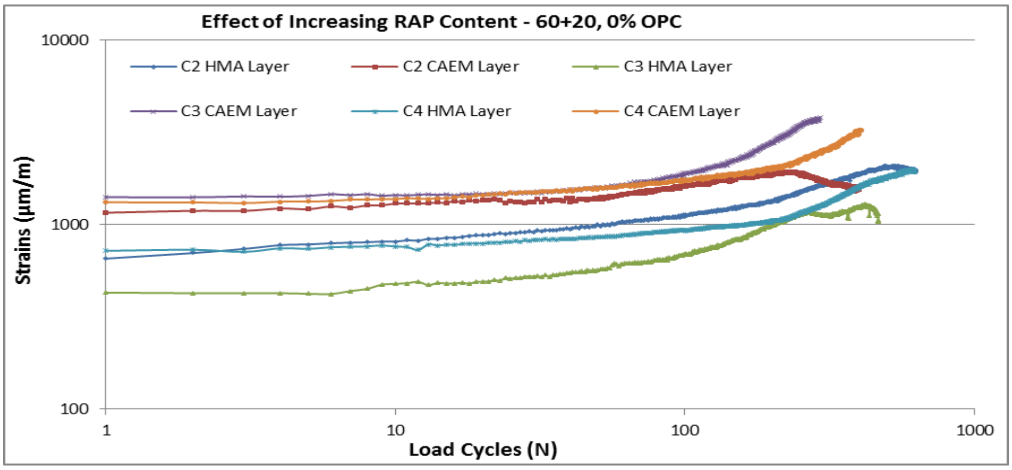


Figure 6-27: Strain Levels for Categories 2 – 4 for 60mm thick CAEMs + 20mm thick HMA at 0% OPC

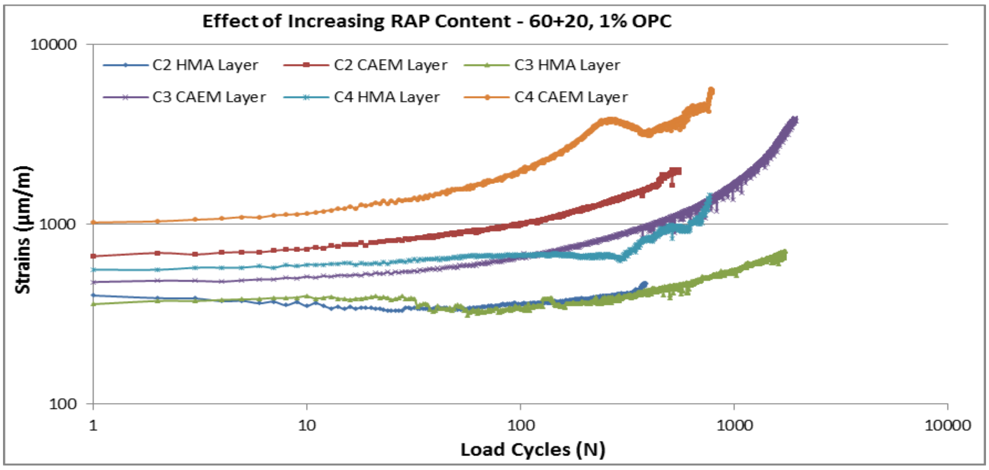


Figure 6-28: Strain Levels for Categories 2 – 4 for 60mm thick CAEMs + 20mm thick HMA at 1% OPC

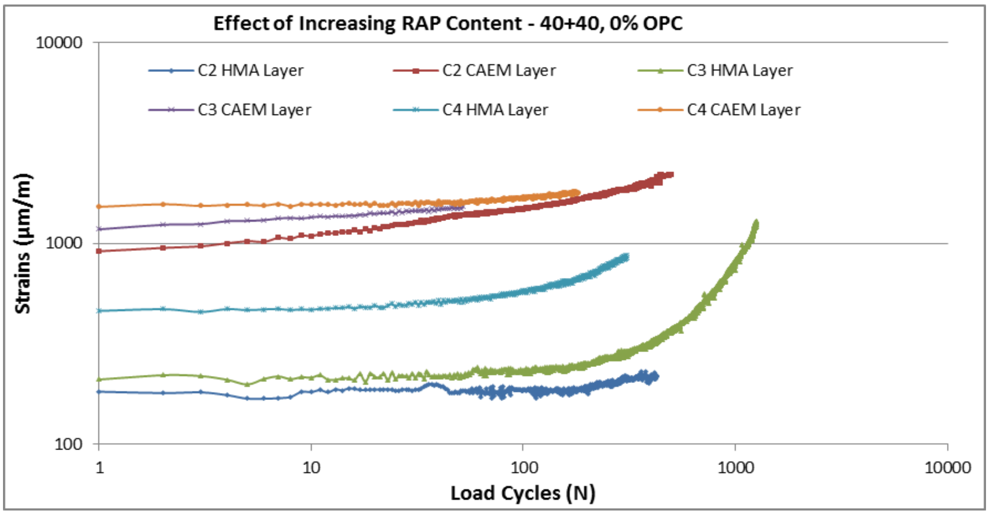


Figure 6-29: Strain Levels for Categories 2 – 4 for 40mm thick CAEMs + 40mm thick HMA at 0% OPC

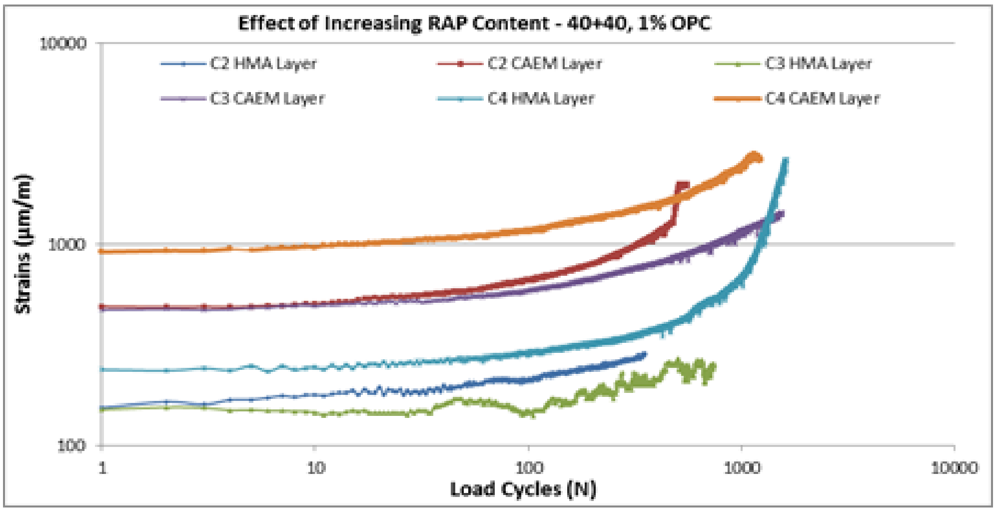


Figure 6-30: Strain Levels for Categories 2 – 4 for 40mm thick CAEMs + 40mm thick HMA at 1% OPC

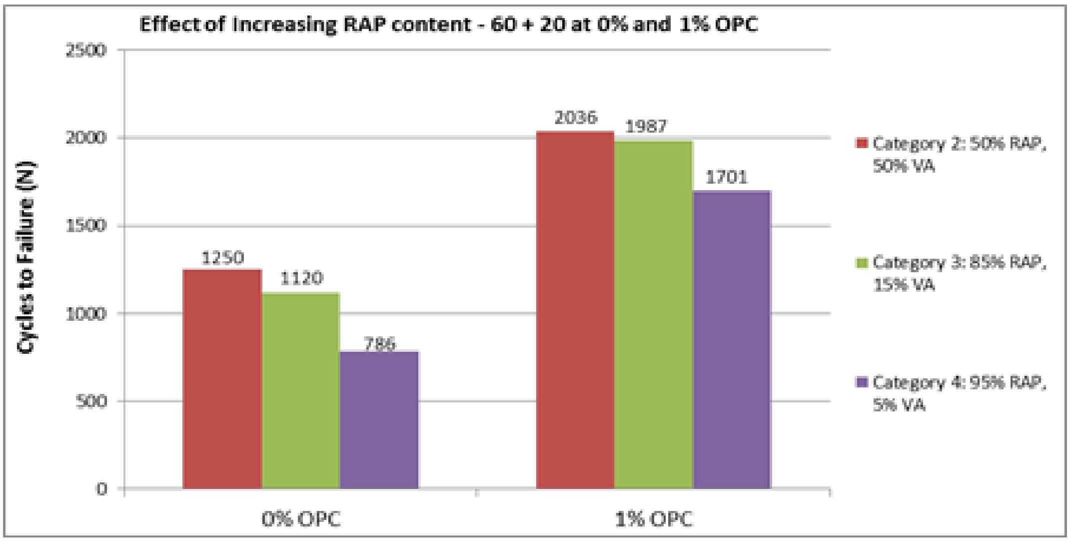


Figure 6-31: Number of Cycles to Failure for Categories 2 – 4 for 60mm thick CAEMs + 20mm thick HMA at 0% and 1% OPC

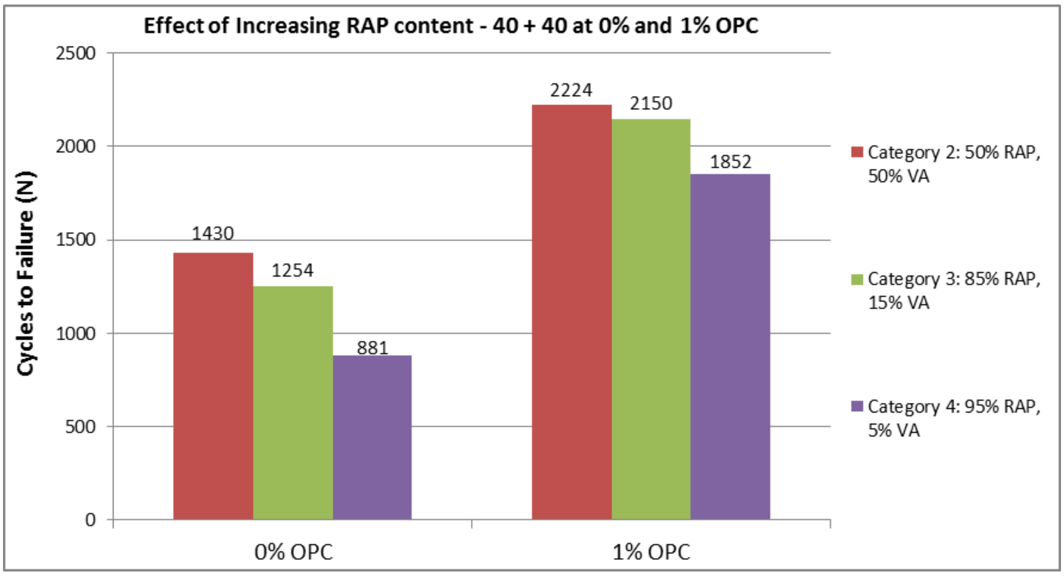


Figure 6-32: Number of Cycles to Failure for Categories 2 – 4 for 40mm thick CAEMs + 40mm thick HMA

The number of cycles to failure for 60mm thick CAEMs + 20mm thick HMA at 0% and 1% OPC and 40mm thick CAEMs + 40mm thick HMA at 0% and 1% OPC are shown above in Figure 6-31 and Figure 6-32.

6.4.4 The Influence of OPC

The study investigated the effect of OPC content by producing mixtures at 0%, 1% and 3% OPC content. The results presented above in Figure 6-24 to Figure 6-32 gives a very good indication of the positive influence of adding 1% OPC to the CAEMs. This significantly lowers the strains resulting in the increase to the number of cycles to failure in comparison to mixtures without OPC with further details in Table 6-2.

Pavement Structure	Category	Number of Cycles to Failure		Percent Increase (%)
		0% OPC	1% OPC	
80mm thick CAEM	Category 1	203	696	243
	Category 2	972	1789	84
	Category 3	908	1958	116
	Category 4	678	1514	123
60mm thick CAEM + 20mm thick HMA	Category 2	1250	2036	63
	Category 3	1120	1987	77
	Category 4	786	1701	116
40mm thick CAEM + 40mm thick HMA	Category 2	1430	2224	55
	Category 3	1254	2150	71
	Category 4	881	1852	110

Table 6-2: WTT Results Showing the Influence of 1% OPC on CAEMs

The study aimed to further investigate the purported benefits of incorporating high cement content at 3% OPC to gain knowledge and understanding on typical responses of the CAEMs at this level. The mixtures with 3% OPC were added only on 80mm thick Category 2 and Category 3 mixtures tested at 20°C. The results are shown below in Figure 6-33 and Figure 6-34.

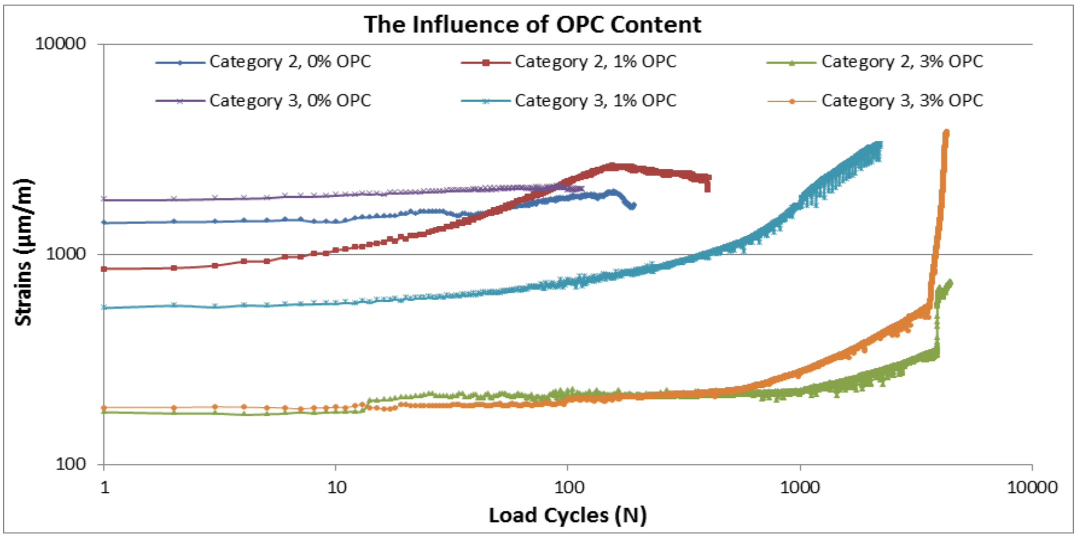


Figure 6-33: WTT Results Showing the Influence of 3% OPC on CAEMs

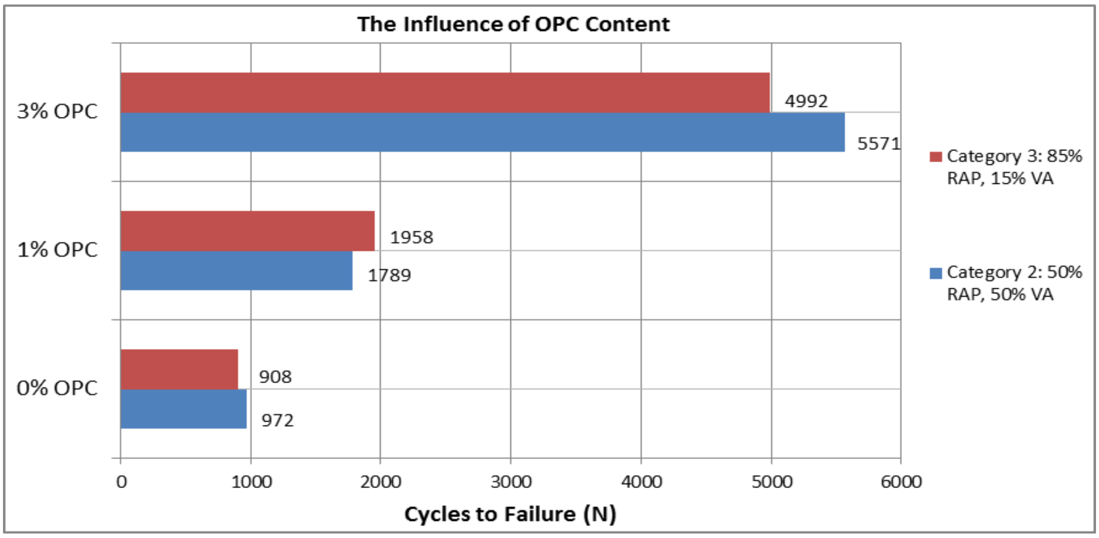


Figure 6-34: Number of Cycles to Failure for Categories 2 and 3 for 80mm thick CAEMs with 3% OPC

The addition of 3% OPC resulted in significantly lower strains with values less than 500 microstrains at the bottom of the CAEM base layer. 3% OPC also resulted in an increase in the number of cycles to failure with a 211% and 154% increase for mixtures with 3% OPC in comparison to mixtures with 1% OPC for 80mm thick CAEMs for Categories 2 and 3 as seen in Figure 6-33 and Figure 6-34.

6.4.5 The Effect of Load

This study also aimed to understand how reduced traffic loading affected the CAEMs susceptibility to cracking and deformation. A standard wheel load of 1.35kN which is representative for tyre pressures on highways was reduced to 0.9kN to simulate lightly trafficked roads (Ogundipe, 2011). As with the study on the influence of OPC, focus was on 80mm thick CAEMs for Categories 2 and 3 tested at 20°C with 0% and 1% OPC. The test set up was exactly the same with the wheel load reduced to 0.9kN as the major difference. Results of this analysis are presented below in Figure 6-35 and Figure 6-36.

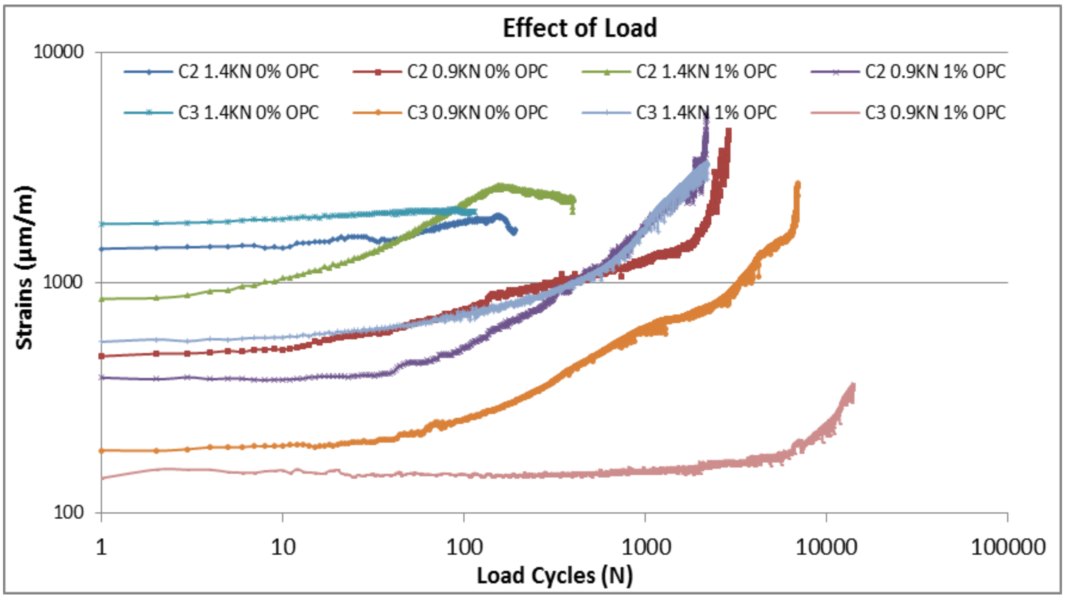


Figure 6-35: Strain Levels Showing the Effect of Load on CAEMs

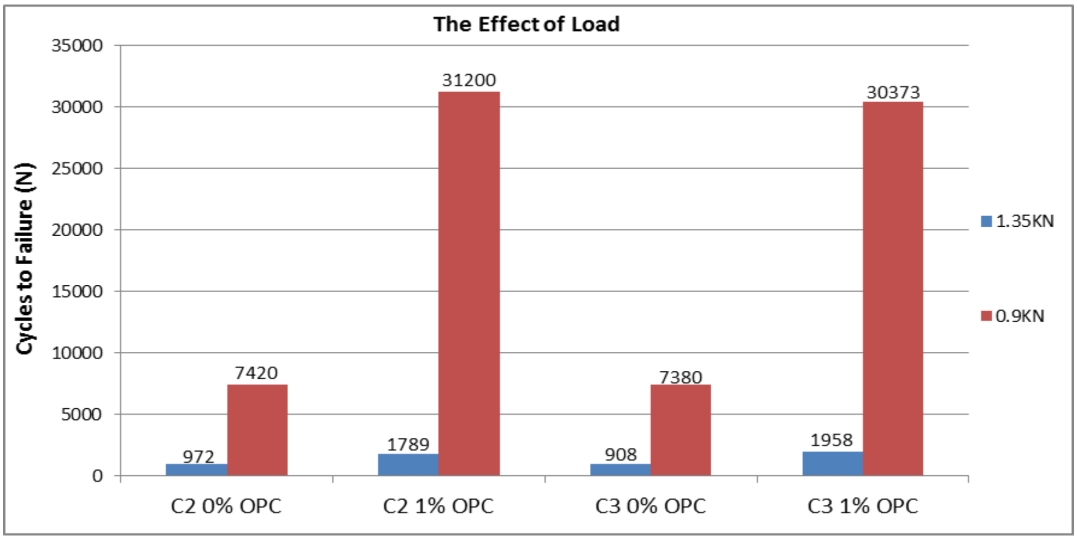


Figure 6-36: Number of Cycles to Failure Showing the Effect of Wheel Loadings

Figure 6-35 and Figure 6-36 indicates that at the reduced load of 0.9kN, the mixtures had higher number of cycles to failure with figures of 7420 and 7380 cycles for Categories 2 and 3 at 0% OPC and 31200 and 30737 cycles for Categories 2 and 3 respectively at 1% OPC. Figure 6-37 provides information on the number of cycles for cracks to propagate from CAEM base layer to bottom of the HMA Layer in comparison to the total number of cycles to failure of the specimen.

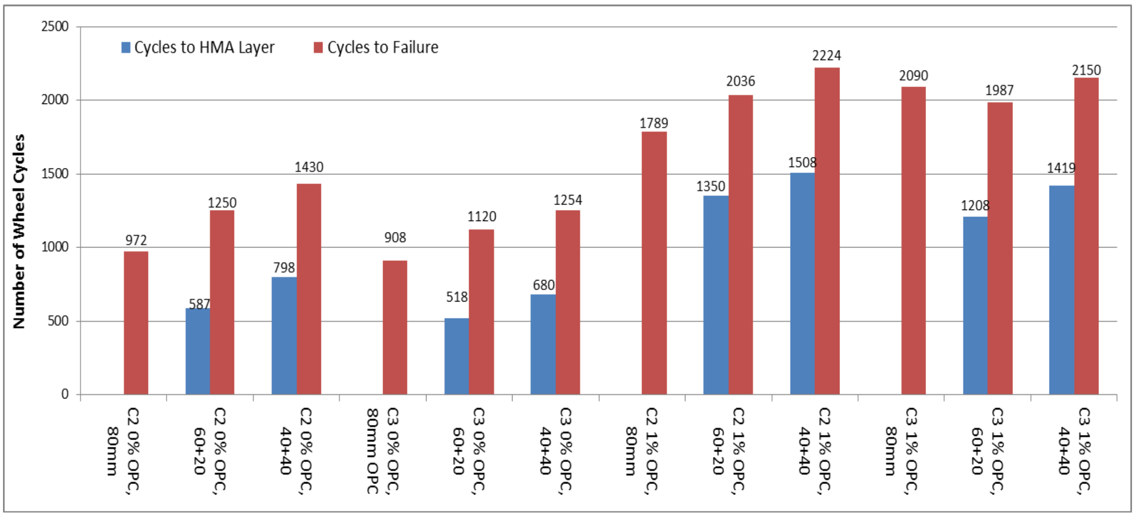


Figure 6-37: Cycles to cracks at bottom of HMA Layer VS Cycles to Failure

6.5 Conclusions

- The test simulated the susceptibility of the CAEM base layer incorporating HMA overlays to fail under WTT loading.
- The relationship between strains in the CAEMs and HMA overlay due to temperature, HMA overlay thickness, RAP content, influence of OPC content and effect of load on crack initiation, propagation and rate of crack growth was investigated.
- Results from the test showed that the performance of the specimens was affected by the test temperature. Increased test temperatures especially at 32°C resulted in increased strains in the CAEMs which led to multiple cracks and eventual disintegration of the specimens.
- It was discovered that the beams with thicker HMA surfacing overlays had lower strains in comparison to the mixtures with thinner HMA surfacing overlays.
- The mixtures with no HMA surfacing overlays performed the worst having the highest observed strains and the shortest number of cycles to failure.
- Category 4 mixtures performed the worst under these conditions having the lowest number of cycles to failure in comparison to Categories 2 and 3.
- The test highlighted once again the purported benefits of adding 1% OPC to CAEMs. The strains experienced by the CAEMs reduced and the number of cycles to failure increased; more than doubling in most cases.
- When high load magnitudes of 1.35kN were applied, higher strains were experienced by the CAEMs leading to loss of stiffness, multiple crack initiation and propagation and eventual failure/disintegration of the mixture.

- The CAEMs lasted significantly longer and experienced lower strains at reduced loads of 0.9kN. The addition of 3% OPC to CAEMs and the use of effective surfacing and sub-layers at expected high load magnitudes are recommended.

7. STRUCTURAL DESIGN AND MODELLING OF FLEXIBLE PAVEMENTS INCORPORATING CAEMS

7.1 Introduction

Structural analysis of flexible pavements has greatly developed since Boussinesq conducted initial studies in which soils were modelled as a linear-elastic material (Gupta and Kumar, 2014). The structural design of pavements is required to ensure that the pavement serves its function structurally and functionally in an economical and feasible manner for the designed life period. Most analytical models used for the design of flexible pavements are based on linear elastic theories which assume that each layer is homogenous, linear elastic with a constant modulus of elasticity, isotropic and the surface layer is free from shearing and normal stresses. Nidhi and Nagakumar (2003) stated that this might not always be the case as the design of pavement structures is complex in nature. The mechanistic-empirical approach to the design of flexible pavements is an emerging technology which takes into account fatigue cracking and rutting which are the two major distress factors in pavements as highlighted by Muniandy et al., (2013). Huang (2004) explained that the use of the mechanistic-empirical approach for structural analysis of flexible pavements is a viable option. He stated that this method seeks to relate the mechanics of the materials to an input (wheel loads) to explain phenomena related to the output (stresses, strains and deflections) experienced within the pavement structure. The mechanistic-empirical approach proved useful and was adopted for use in the structural design and analysis for this study.

7.2 Design Considerations

7.2.1 Overview

This chapter focuses on investigating the purported effects of the CAEMs designed in this research project on the relevant responses (stresses and strains) using the 16 hypothetical pavement structures as stated in Table 7-1. The mechanistic-empirical approach to structural analysis was conducted using the software called KENLAYER which was able to simulate non-linear elastic behaviour of CAEMs. The pavement structures investigated as presented in Table 7-1 were selected in such a way that they featured a wide range of practical options used in industry.

Pavement Structure	Surfacing Thickness (mm)	Base Thickness (mm)	Sub-Base Thickness (mm)
1	30	150 (3)	200
2	50		
3	75		
4	100		
5	30	200 (4)	200
6	50		
7	75		
8	100		
9	30	250 (5)	200
10	50		
11	75		
12	100		
13	30	300 (6)	200
14	50		
15	75		
16	100		

Table 7-1: Pavement Structures Investigated

7.2.2 Failure Modes

Brown (2012) stated that in order to design any civil engineering structure it is critical to have a good understanding of its failure modes in service. Brown (2012) further stated that pavement structures will mostly not fail suddenly; rather they will gradually deteriorate with time, under repeated traffic loadings, climatic and environmental factors. Traffic loading is a major factor in the deterioration of pavements taking into account the number and magnitude of the wheel loads.

7.2.3 Non-linearity of CAEMs

CAEMs are stress dependent and thus are non-linear in elastic response as observed by Ebels (2008) and Oke (2010). This behaviour was also observed for cold mix asphalts incorporating foamed bitumen as well; as shown by (Jitareekul, 2009 and Kuna, 2015). Thus, the work described in this chapter was analysed bearing in mind that the layer of interest (road base) is non-linear in elastic behaviour. Non-linear elastic analysis was considered to obtain the mechanistic responses (stresses and strains) for both the CAEM base layer and the granular sub-base of the pavement structure investigated as suggested by Huang (2004), Ebels (2008) and Oke (2010).

7.2.4 Functionality of KENLAYER

A major function of KENLAYER is its ability to provide solutions for an elastic multilayer system under a circular area. The solutions are superimposed for multiple wheels applied iteratively for non-linear layers and collocated at various times for viscoelastic layers as stated by Huang (2004).

KENLAYER is capable of analysing layered systems under single, dual, dual-tandem or dual-tridem wheels with each layer behaving differently either as a linear elastic, non-linear elastic or viscoelastic material. Huang (2004) stated that 3 methods have been incorporated into KENLAYER for non-linear analysis. Method 1 is the most accurate but consumes the most time. This method involves sub-dividing the stress dependent layer which in this study is the CAEM layer. Please note that the number in parentheses in Table 7-1 indicates the number of sub layers adopted for the non-linear layer in the study.

Huang (2004) stated that using a non-linear elastic approach and dividing the cold mix layer into sub-layers, each with a unique stiffness value called the seeding stiffness value results in higher stress ratios which are better estimates than those obtained by linear elastic calculations. Method 1 was used in this study Methods 2 and 3 are detailed and documented by Huang (2004).

7.2.5 Design Approach

The approach for the analysis involved obtaining the stress and strain responses of the pavement layers as a result of wheel load inputs which enables the identification of critical points within the pavement structure. The stress and strain responses are then compared to the allowable stresses and strains obtained from laboratory tests. The two principal modes of failure are cracking of bound layers and rutting in the wheel tracks resulting in two major points for critical stress and strain analysis that are located at the bottom of the asphalt base layer and the top of the subgrade where maximum values of key parameters develop as stated by Huang (2004).

For design purposes in this study, fatigue was the major failure criterion investigated resulting in the horizontal tensile strain at the bottom of the CAEM base layers to be used as the design parameter. This location is due to the fact that fatigue cracking of the bituminous layer under repeated wheel loads is initiated by the level of tensile strain rather than the stress experienced and therefore this parameter is generally used to characterise the fatigue of bituminous materials (Ebels, 2008; Oke, 2010). The maximum strains at the bottom of the CAEM layers were noted. The effect of the various CAEM base layer thicknesses as shown in Table 7-1 and the effect of increasing the surfacing layer thickness was analysed with the results used to produce hypothetical design charts.

7.2.6 Shift Factors

It is generally agreed that the allowable number of load repetitions is related to the tensile strain at the bottom of the asphalt layer. Thom (2008) observed that a relationship is found between tensile strains in asphalts under traffic load and fatigue tests conducted in the laboratory. With respect to this, cracking in asphalt pavements in-service as a result of tensile strains is deemed to be related to failure in fatigue tests under repeated loading. Shift factors need to be introduced to establish a relationship between laboratory tests and field performance. This creates a concern in selecting the appropriate shift factor in order to establish a relationship between laboratory and field performance. This is usually dependent on type of loading, frequency of loading, test temperature etc. as stated by Oliveira et al., (2008). Oke, (2010) in a similar research found a shift factor of 440 appropriate for use with similar CAEMs. This was adopted for use in this study.

7.3 Methodology

A hypothetical flexible pavement with a CAEM base layer used is depicted in Figure 7-1. A single tyre wheel loading system of 40kN (80kN being the single standard axle), 700 kPa tyre pressure and tyre contact radius of 143mm was used as recommended by (Oke, 2010).

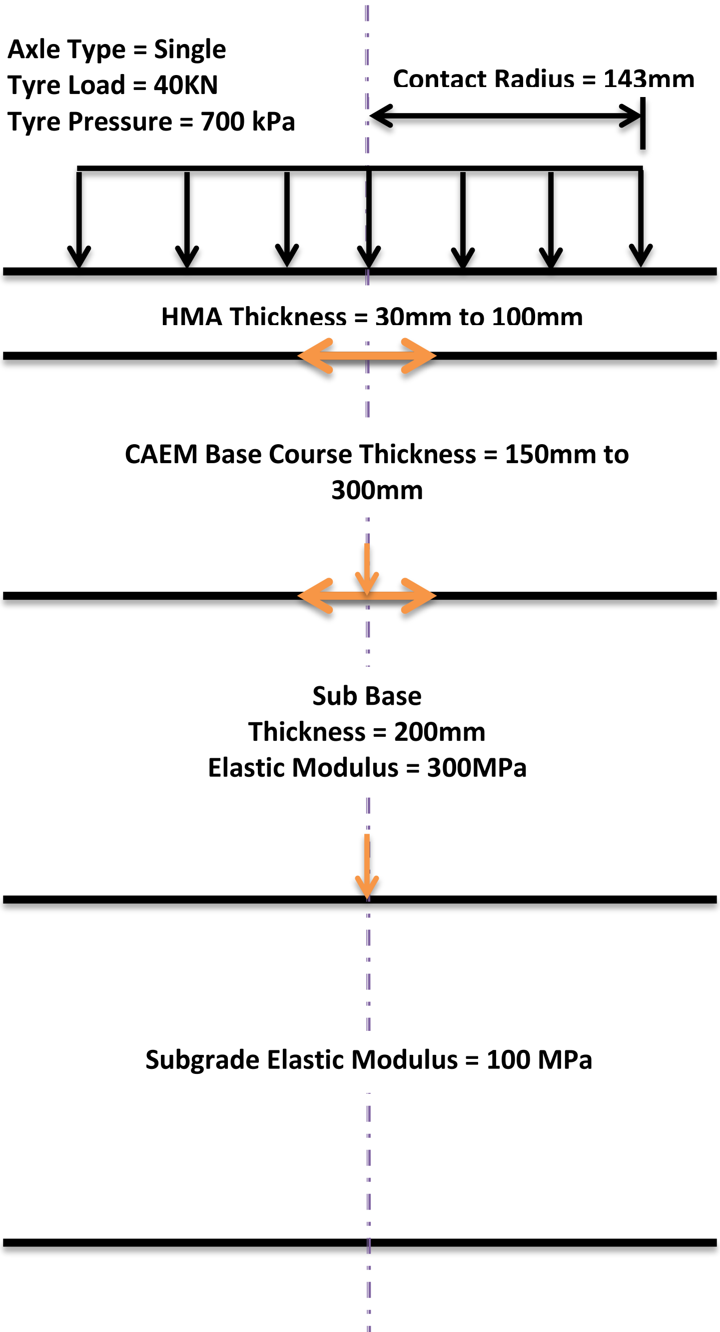


Figure 7-1: Typical Pavement Structure Considered for Analysis

Ebels (2008) stated that the single wheel loading allows for the critical horizontal position to lie directly under the wheel load. The main structural element of the pavement structure is the base layer overlaid with a thin (30-100 mm) HMA surfacing layer. The role of the surfacing layer is to provide a safe and durable surface for road traffic with sufficient skid resistance. The sub-base and subgrade together are considered as the pavement foundation. For designing the pavements a standard wheel load of 40kN was considered and the actual load spectrum converted into an equivalent number of standard loads. For non-linear elastic analysis using KENLAYER the parameters of the traditional K-θ model (K_1 and K_2) apply which are detailed in Huang (2004). The K-θ model is used to fit unbound granular materials and the stress state of specimens as seen in Figure 7-2 and Figure 7-3.

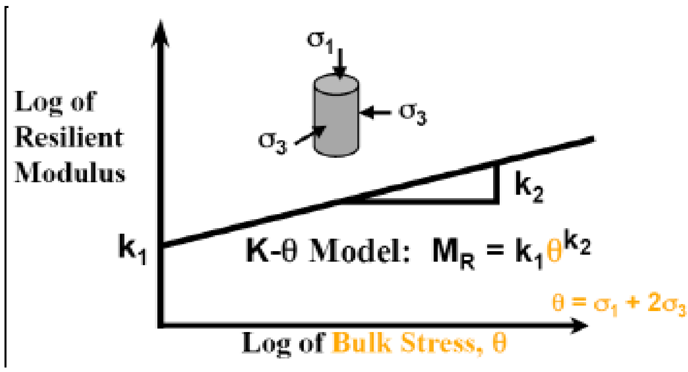


Figure 7-2: K-θ Model (Mishra and Tutumluer, 2007)

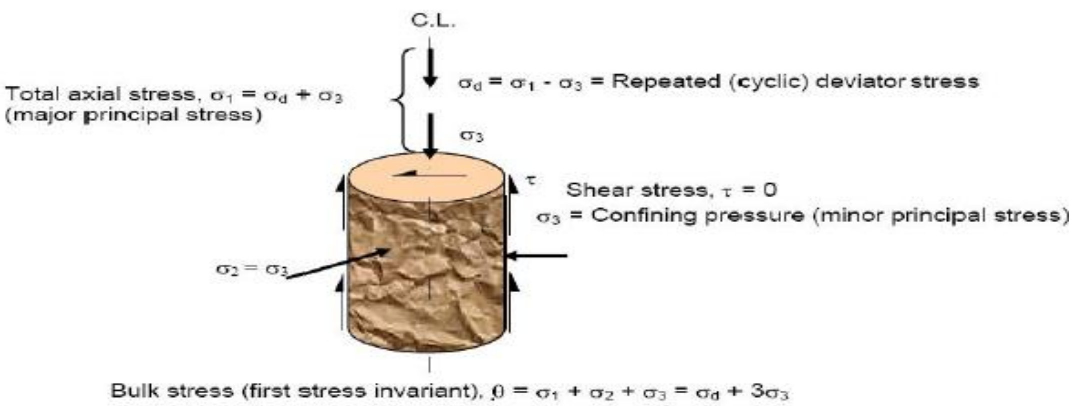


Figure 7-3: Stress State for Resilient Modulus Testing

It should be noted that the K_1 and K_2 values were estimates from previous research works that include Huang (2004), Ebels (2008) and Oke (2010). This is due to constraints with laboratory test equipment and the fact that the sub-base and subgrade layers were not investigated in this study. The examples worked out in this chapter are therefore hypothetical and serve the purpose of illustrating how the material properties determined in this study taking into account the mechanical and performance tests conducted in Chapter 5 influence the modelling and structural analysis of asphalt pavements incorporating CAEM base layers. The K_1 and K_2 values used are detailed in Table 7-2.

Category	K1 (MPa)	K2	Density (kg/m^3)
Category 1, 0% OPC	27.750	0.550	2090
Category 2, 0% OPC	37.251	0.452	2089
Category 3, 0% OPC	33.880	0.441	2052
Category 4, 0% OPC	26.310	0.490	2020

Table 7-2: K_1, K_2 Values and Densities

The densities for the sub-base and subgrade are 2000kg/m^3 and 1800 kg/m^3 respectively. It was assumed that the layers were fully bonded. The Poisson's ratio adopted was 0.35 for all the layers except the subgrade which was 0.4 as suggested by Huang (2004). The following medium condition parameters suggested by Huang (2004) were also used in the analysis for the subgrade and sub-base layers which include:

- Coefficient of Earth Pressure = K_0 = 0.8
- Sub-base K_1 = 30.912
- Subgrade K_1 = 52.992

The surfacing layer on top of the CAEM base layer had a stiffness modulus of 3000MPa. This was a constant for all cases in the analysis. The seeding values for the CAEM base layer equally sub-divided ranged between 1000MPa – 600MPa. This seeding value is needed because an iterative procedure is used. At each iterative step, a constant stiffness value is calculated by KENLAYER based on the magnitude of stresses obtained in the previous iteration. The moduli of non-linear layers are adjusted as the stresses vary while the moduli of the linear layers remain the same. KENLAYER calculates the stresses at the mid-depth of each sub-layer to determine the stiffness. After stress values due to single or multiple wheels are calculated, the elastic moduli of non-linear layers are recalculated and a new set of stresses is determined. This is repeated by the KENLAYER program until the moduli converges to a specific tolerance limit (Huang, 2004). The outputs from the KENLAYER analysis include vertical, horizontal and shear stresses and strains and vertical displacement values. The stiffness values for each of the sub-layers are also presented.

7.4 Stress and Strain Distributions

Using Pavement Structure 6 as stated in Table 7-1 as a case study with surfacing layer thickness of 50mm, base layer thickness of 200mm (divided into 4 equal sub-layers) and a sub-base layer of 200mm, the stress and strain distributions through pavements for Categories 1 – 4 at 0% OPC were analysed using KENLAYER. Figure 7-4 shows the vertical stress distribution in the various categories of CAEMs. The figure shows that there is a reduction in vertical stress as pavement depth reduces. The result also showed that vertical stress distribution in the CAEMs was not significantly different for all the pavement structures considered.

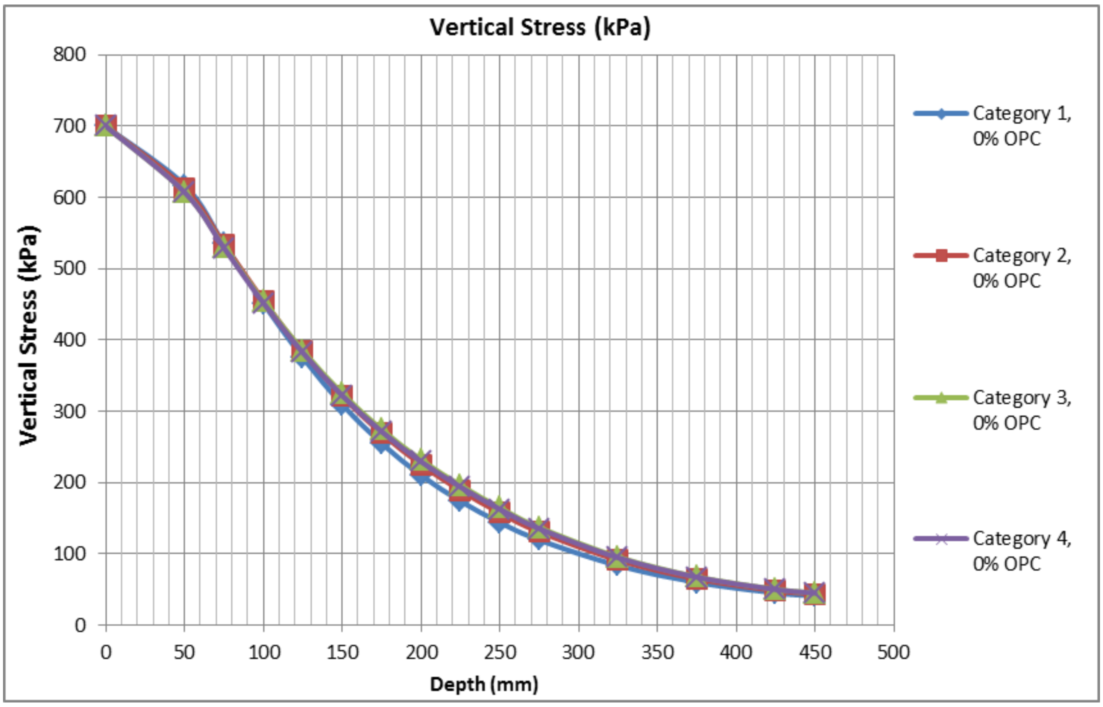


Figure 7-4: Vertical Stress Distribution through Pavement Structure 6

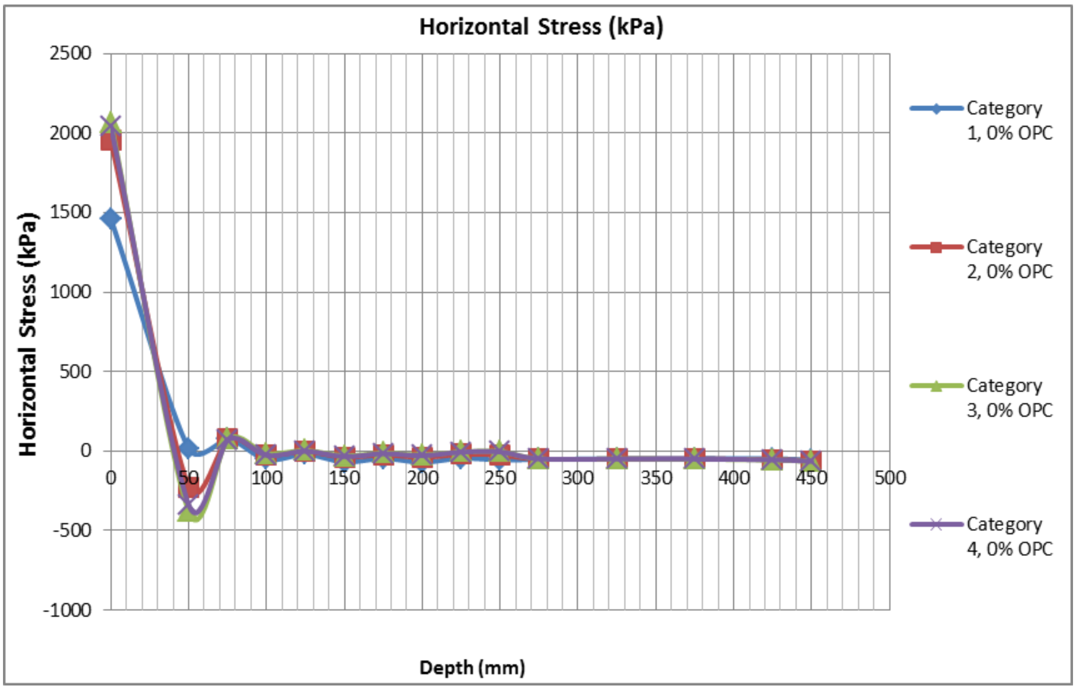


Figure 7-5: Horizontal Stress Distribution through Pavement Structure 6

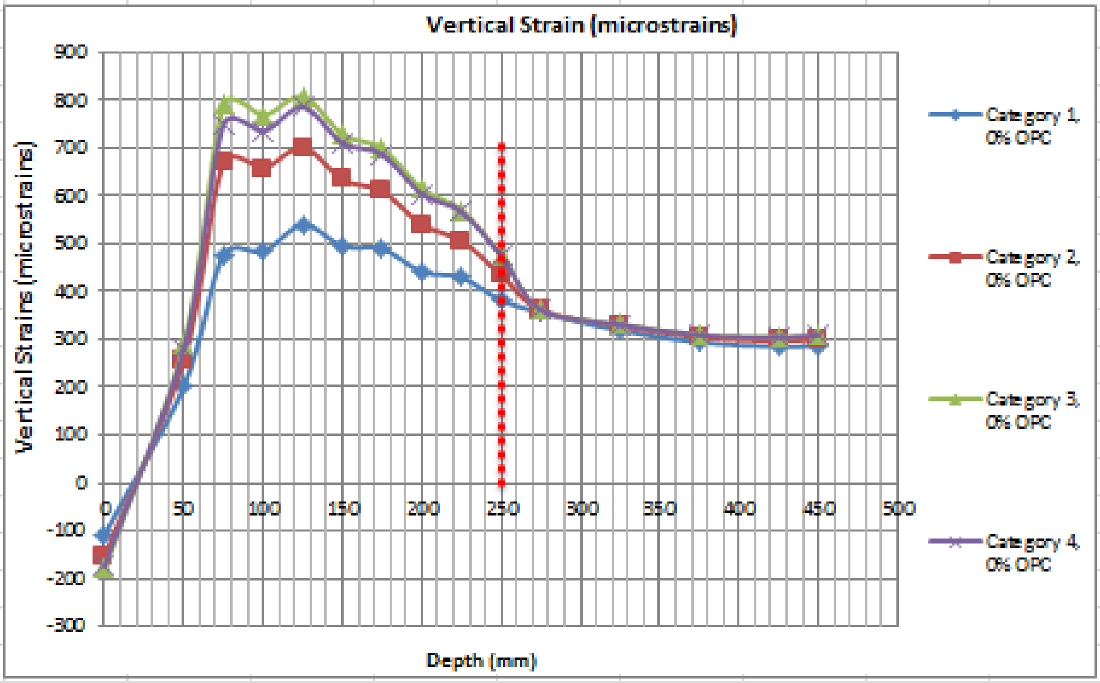


Figure 7-6: Vertical Strain Distribution through Pavement Structure 6

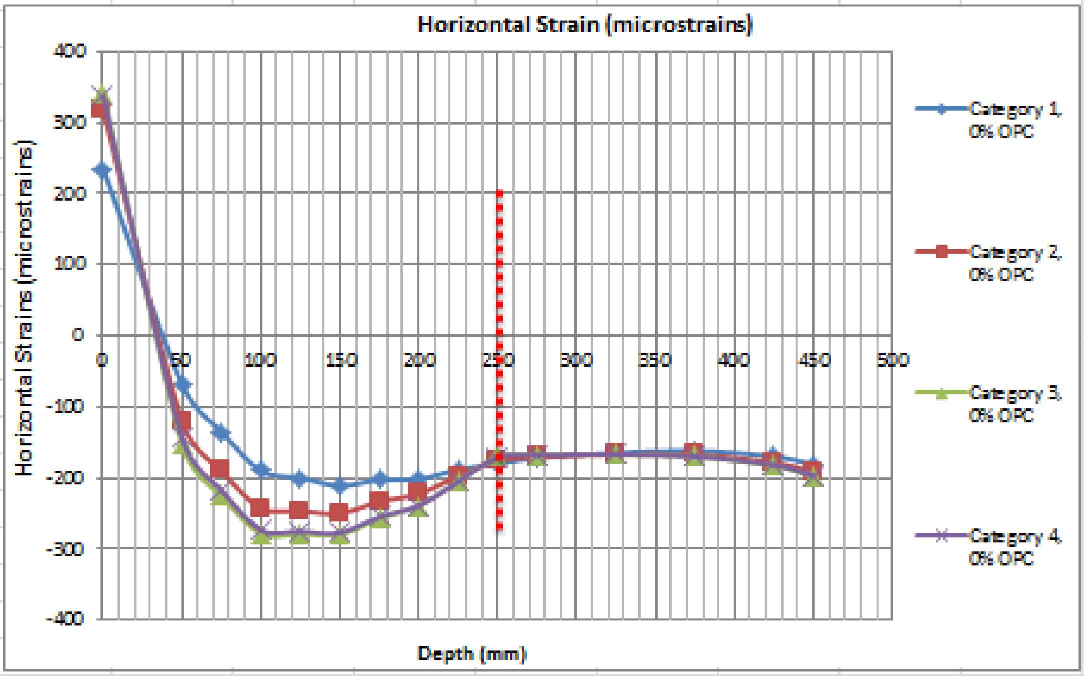


Figure 7-7: Horizontal Strain Distribution through Pavement Structure 6

Figure 7-5 shows the horizontal stress distribution. The horizontal stress generally reduced from a relatively high stress value at the top of the HMA surfacing layer to reduced stresses as the pavement depth increased. It was observed that there was a step change in horizontal stress transitioning from the surfacing layer to the CAEM base layer. Category 1 had the least horizontal stress but overall, Categories 2 – 4 had similar trend characteristics.

Figure 7-6 and Figure 7-7 depict the vertical and horizontal strain distribution for the various pavements with Categories 1 – 4 CAEMs base layers. The red dash lines in Figure 7-6 and Figure 7-7 show the bottom level of the CAEM layers in the hypothetical pavement structure. The figures indicate considerable differences in strains for the various categories of CAEM base layers. The maximum horizontal and vertical strains were towards the middle of the sub-divided CAEM base layers. This trend was also observed and reported by Ebels (2008) and Oke (2010) in which similar CAEM materials and pavement structure were used for the KENLAYER analysis.

It was observed as seen in Figure 7-6 and Figure 7-7 that the RAP CAEMs generally had lower horizontal tensile strain values in comparison to Category 1 which comprised of virgin aggregate CAEM. Figure 7-8 to Figure 7-11 presents the stress and strain distributions for Category 2 and 3 at 0%, 1% and 3% OPC for Pavement Structure 6.

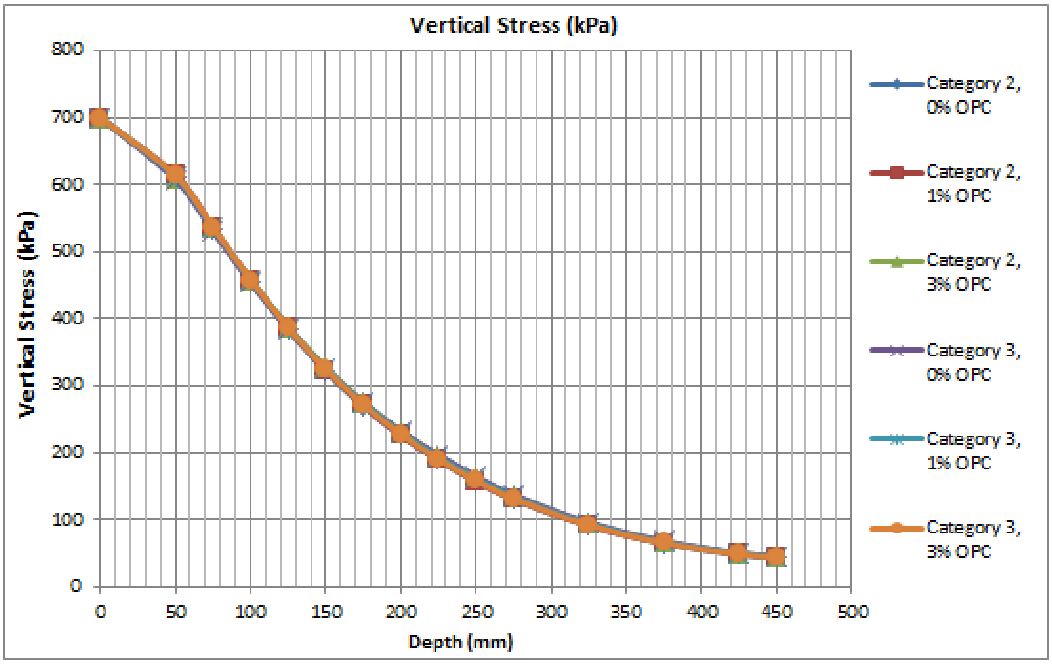


Figure 7-8: Vertical Stress Distribution for Categories 2 and 3 at 0% to 3% OPC

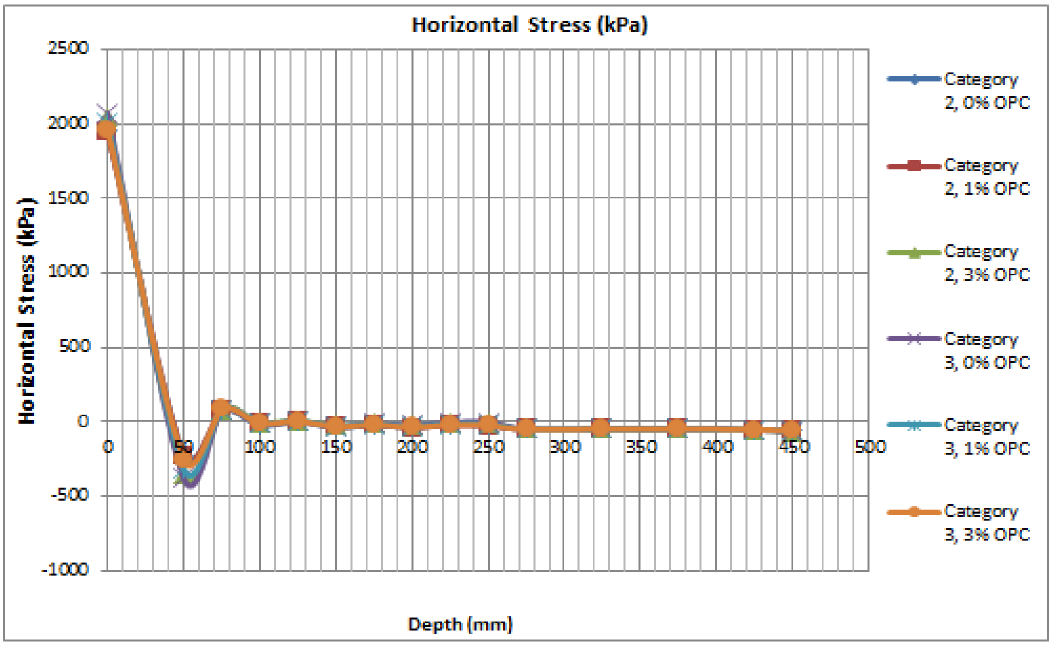


Figure 7-9: Horizontal Stress Distribution for Categories 2 and 3 at 0% to 3% OPC

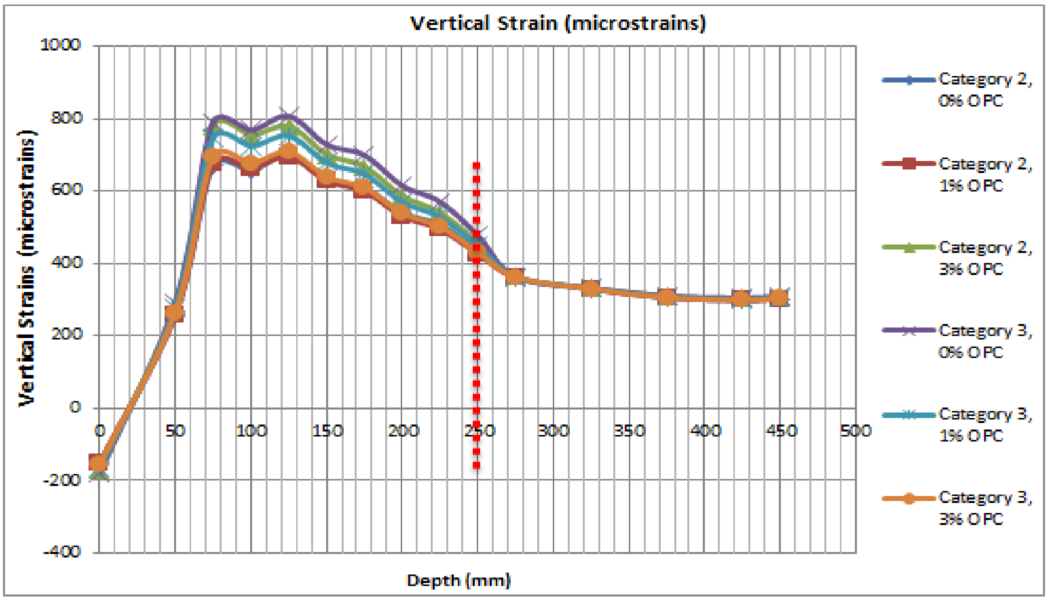


Figure 7-10: Vertical Strain Distribution for Categories 2 and 3 at 0% to 3% OPC

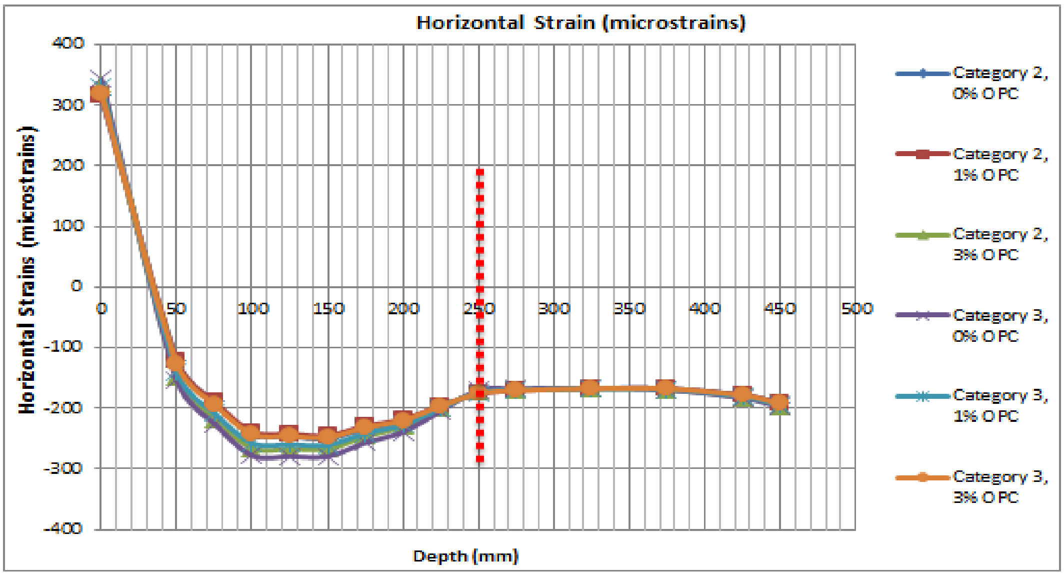


Figure 7-11: Horizontal Strain Distribution for Categories 2 and 3 at 0% to 3% OPC

The results showed very similar trends as discussed above for mixtures without OPC. Figure 7-10 indicated that increasing from 0% to 3% OPC resulted in a reduction in the vertical strains of the CAEMs and vice versa as seen for the horizontal strains in Figure 7-11 although it should be noted that this was more pronounced for Category 3 than Category 2.

7.5 Design Charts

KENLAYER showed that the maximum horizontal strains were closer to the middle of the sub-divided CAEM layers which resulted in these strains being more appropriate for design purposes. The maximum strain values were therefore used for design purposes. Fatigue life of the structural layer was based on the ITFT results obtained and as stated in Chapter 5. As stated in earlier sections, a shift factor of 440 was adopted. The design charts are presented in Figure 7-12 to Figure 7-15. The charts show that an increase in the thickness of the base course and surfacing layer resulted in an increase in the fatigue resistance of the pavement structure. It was also observed that Category 1 had the least fatigue life with an increase in RAP content resulting in noticeable improvement in the fatigue characteristics of the CAEMs with Category 3 and 4 having the highest number of load applications. This could be attributed to the influence of the residual binder in the RAP. The K_1 and K_2 values could have resulted in the high values especially for Category 4 with durability issues being a factor when the totality of the mechanical and performance properties of Category 4 CAEMs is evaluated.

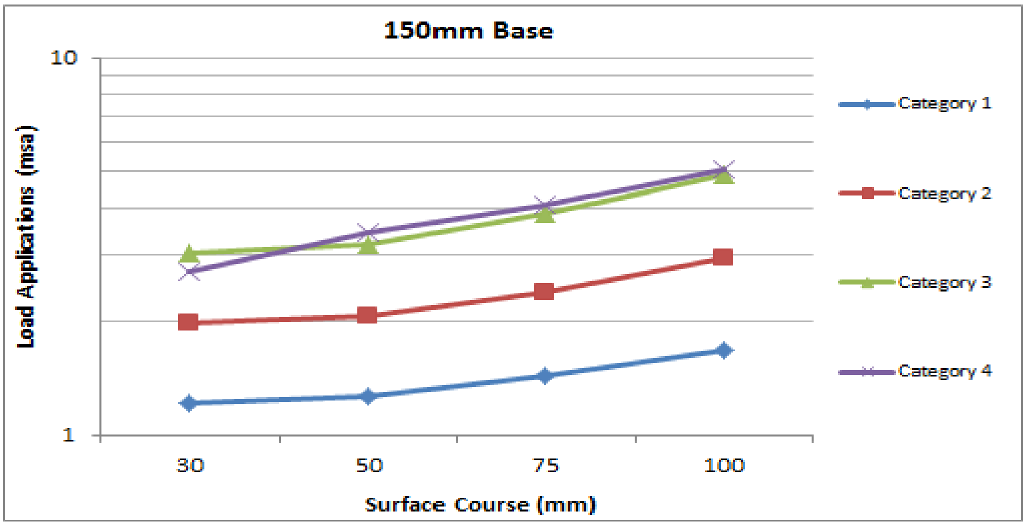


Figure 7-12: Design Chart for 150mm Base Course Pavement Structure

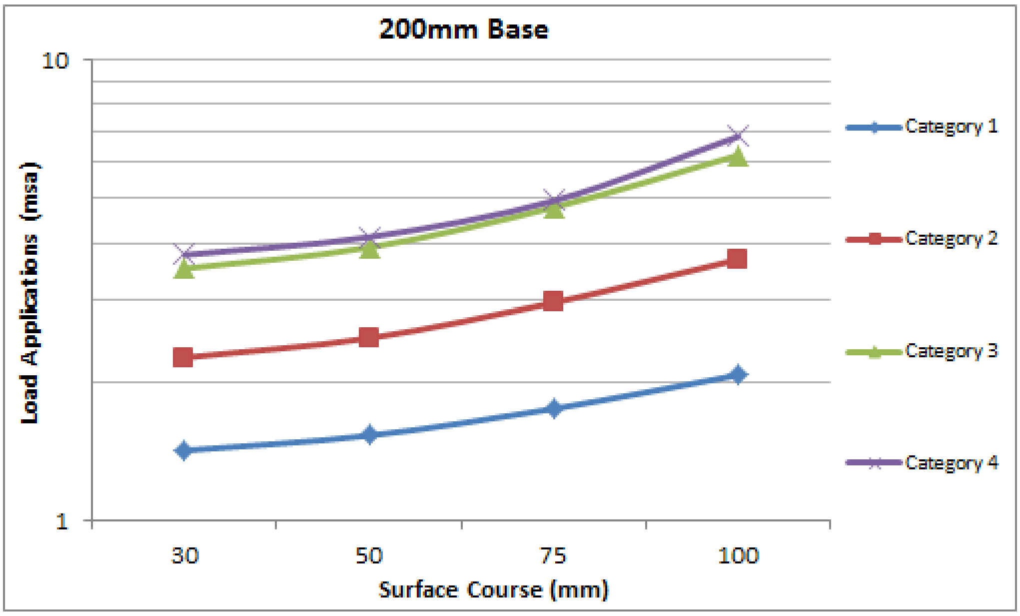


Figure 7-13: Design Chart for 200mm Base Course Pavement Structure

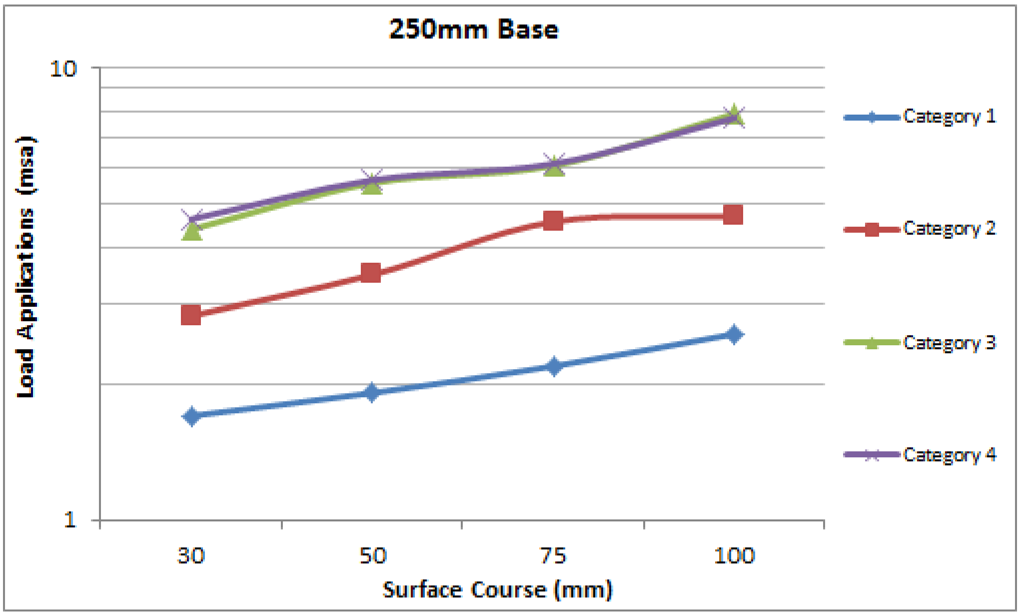


Figure 7-14: Design Chart for 250mm Base Course Pavement Structure

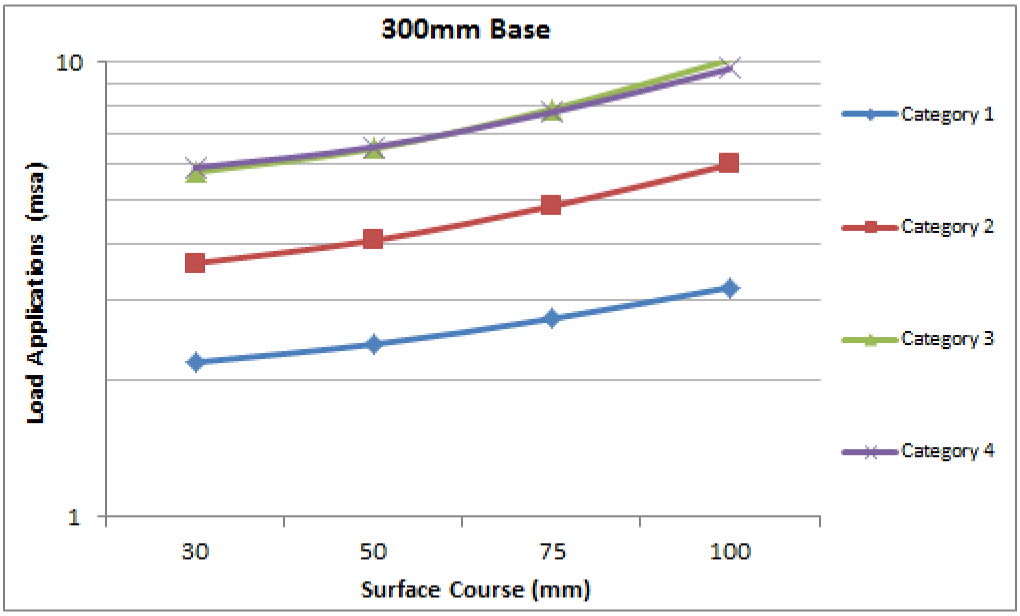


Figure 7-15: Design Chart for 300mm Base Course Pavement Structure

The inclusion of OPC improves the mechanical and performance properties of the CAEMs although it should be noted as observed in Chapter 5 that the inclusion of OPC reduces the flexibility of the CAEMs which results in brittle behavioural patterns resulting in the sudden split of the CAEMs soon after crack initiation which is pronounced when the CAEMs are at elevated strain levels (>200με). It is believed that this phenomenon is responsible for the decrease in life of the CAEMs when OPC content is increased as seen in Figure 7-16.

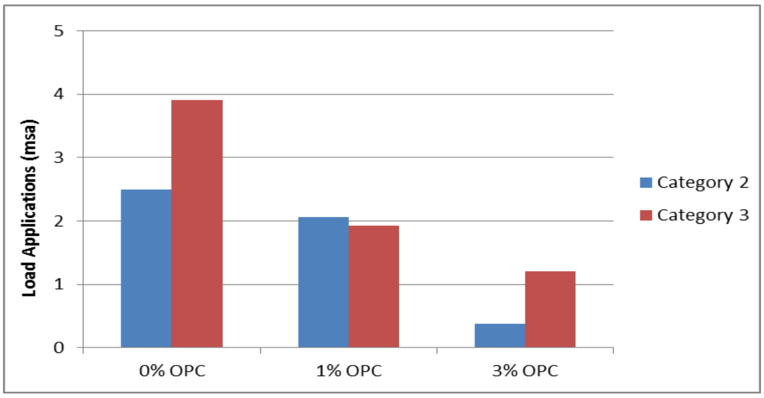


Figure 7-16: Design Chart for Category 2 and 3 at 0%-3% OPC for Pavement Structure 6

7.6 Summary

This chapter has presented the structural design and modelling of flexible pavements incorporating the CAEMs designed in this research project as the base layer. The mechanistic-empirical approach for structural analysis of flexible pavements was used in the study. KENLAYER was used in conducting the mechanistic-empirical structural analysis. Although, the use of KENLAYER was time consuming, it was a good tool for the analysis due to the fact that the software could simulate the non-linear elastic behaviour of the CAEMs. 16 hypothetical pavement structures were investigated varying the thickness of the surfacing and the CAEM base layer in such a way that they featured a wide range of practical options used in industry. The study focused on Categories 1-4 at 0% OPC with the influence of OPC investigated using Pavement Structure 6 for Categories 1-4 at 0, 1 and 3% OPC which showed that the inclusion of OPC resulted in reduced fatigue life. The vertical stress distributions for the various categories of CAEMs were very similar although the vertical and horizontal strains were different. It was observed that the maximum vertical and horizontal strains were towards the middle of the sub-divided CAEM base layers as opposed to the bottom. The RAP CAEMs generally had lower horizontal tensile strain values in comparison to Category 1. The fatigue life of the structural layer was based on the ITFT results. A shift factor of 440 was used. The design charts showed that an increase in the thickness of the base course and surfacing layer resulted in an increase in the fatigue life of the pavement structure. Increased RAP content resulted in an increase in the fatigue life of the CAEMs. Category 1 was the most sensitive and had the least fatigue life overall in all cases. The design charts were useful in quickly ranking the fatigue characteristics of the various categories of the CAEMs.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 Introduction

This research project focused on conducting an in-depth investigation and analysis into the design and optimisation of Cold Asphalt Emulsion Mixtures (CAEMs) incorporating high contents of Reclaimed Asphalt Pavements (RAP). The conclusions presented in this section are from the work conducted in this project. Recommendations for future research are suggested at the end of this chapter.

8.2 Conclusions

8.2.1 Major Conclusions from Mix Design of Cold Asphalt Emulsion Mixtures

The materials for use in CAEMs should be effectively characterised and evaluated in order to ascertain mix suitability. The physical properties of the materials used for this study are discussed in Chapter 3. Alongside the bitumen emulsion and virgin aggregates, particular attention should be given to the characterisation of RAP in order to ascertain if the residual binder in the RAP is “active” or “inactive”. The procedure for manufacturing the CAEMs should be optimised. Key parameters including Pre-wetting Water Content (PWC) and Optimum Bitumen Emulsion Content (OBEC) should be obtained for the set of materials to be used. The Indirect Tensile Stiffness Modulus (ITSM) and the Indirect Tensile Strength (ITS) tests proved useful in quickly ranking the CAEMs.

A detailed study was carried out to obtain the best mixing sequence for the addition of pre-wetting water and bitumen emulsion for the CAEMs. This is summarised in Figure 8-1.

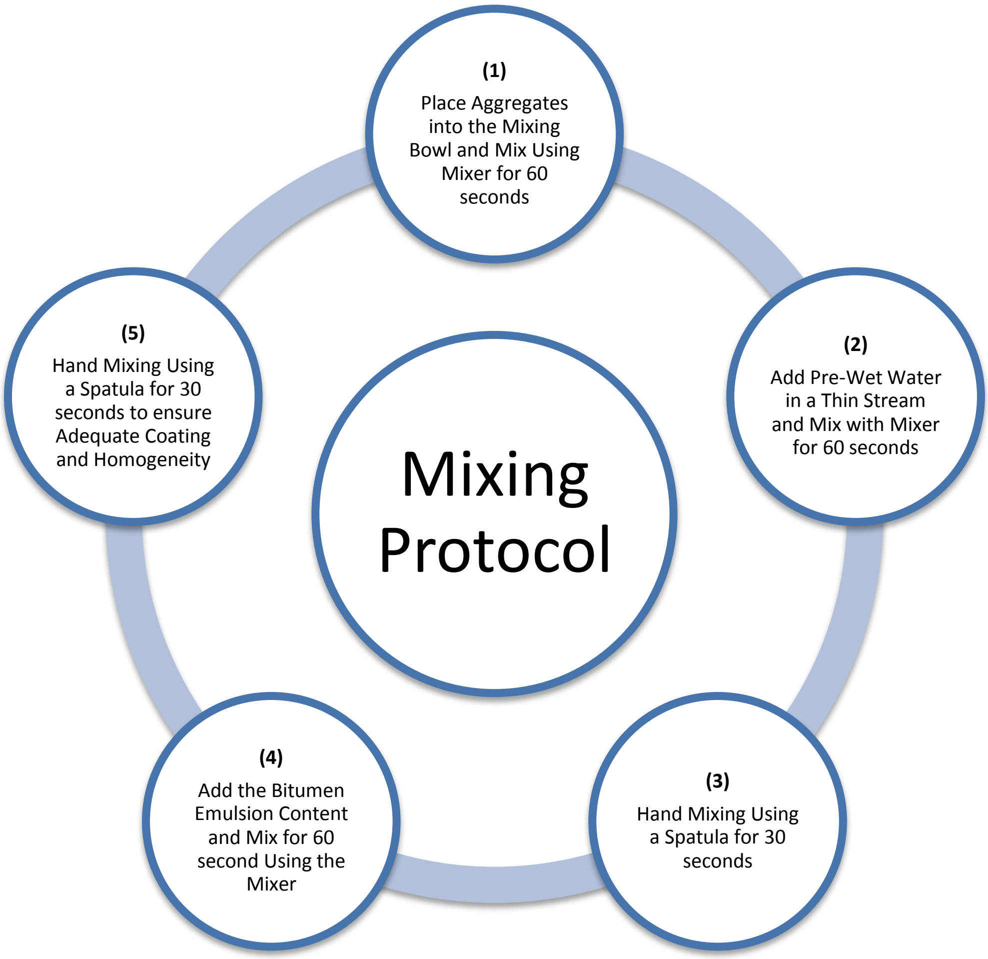


Figure 8-1: Summary of Mixing Protocol

This was to ensure a homogenous, consistent and well coated mix was obtained in order to have optimum compaction and performance properties. Inadequate mixing procedures can reduce compactability and workability.

The OBEC and PWC for the various categories are as stated below:

- Category 1: 6.5% BEC and 1.5% PWC
- Category 2: 5.25% BEC and 2.75% PWC

- Category 3: 4.25% BEC and 1.75% PWC
- Category 4: 3.75% BEC and 1.75% PWC

The mix design protocol proposed for CAEMs incorporating high RAP content is shown below in Figure 8-2.

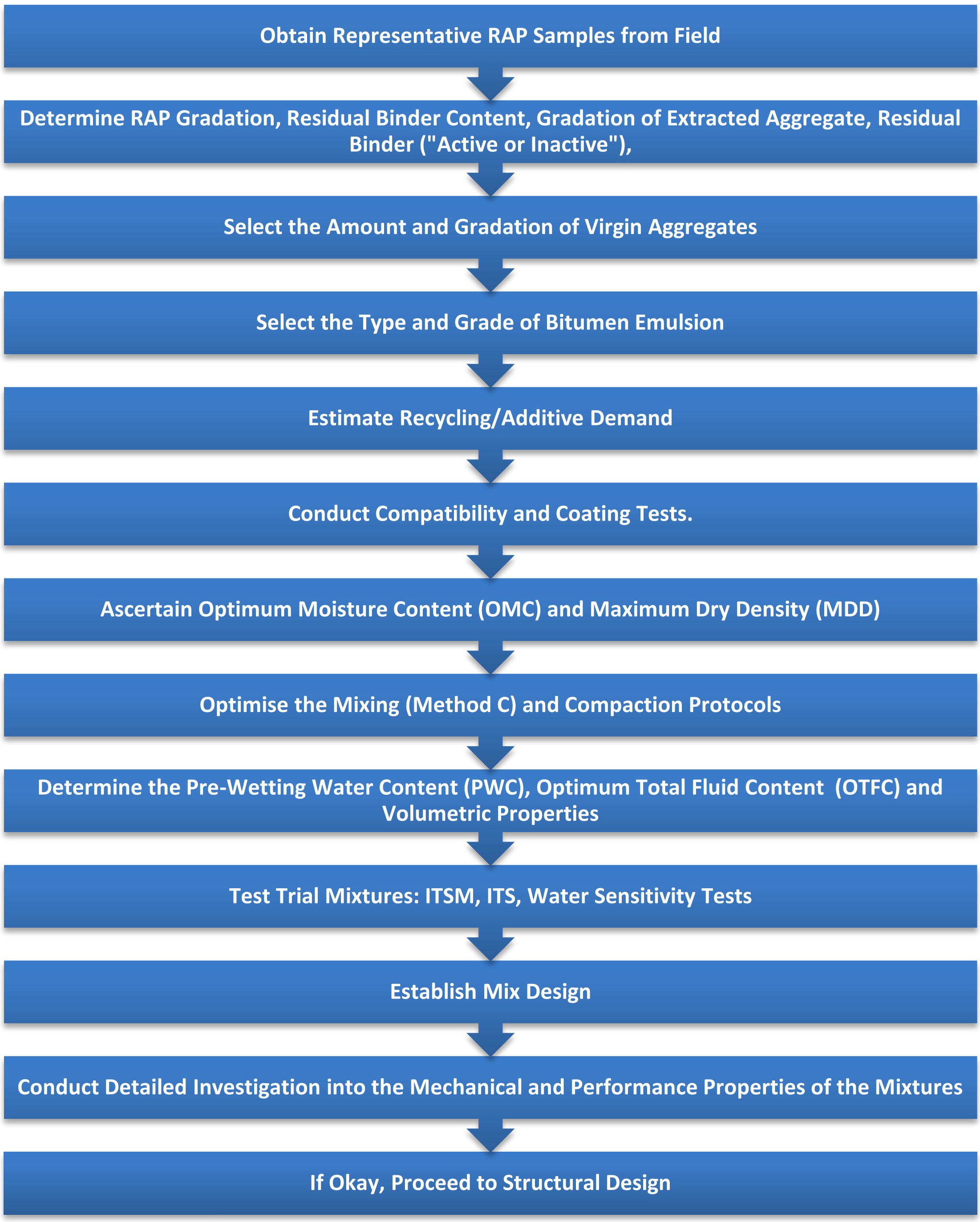


Figure 8-2: Proposed Mix Design Protocol for CAEMs with High RAP Content

8.2.2 Curing Mechanism of CAEMs

Curing temperature has a significant effect on the rate of curing and the 28-day stiffness and strength values. Higher curing temperatures resulted in a higher rate of moisture loss resulting in improved mechanical and performance properties of the CAEMs. The results showed that 1% cement addition is an appropriate means of compensating for the effects of adverse weather during curing.

8.2.3 Mechanical and Performance Properties of the CAEMs

Key performance indicators were characterised into stiffness modulus, fatigue response, resistance to permanent deformation and water sensitivity of the CAEMs.

8.2.3.1 Stiffness and Strength Characteristics

The results showed that at 0% OPC, the stiffness results for Categories 1 – 3 had very similar average values ranging between 2323 MPa to 2692 MPa with no major discernible difference. This was also observed for the strength tests with average values ranging between 531MPa to 598 MPa. Category 4 CAEMs performed the worst with the lowest average stiffness and strength values of 1453MPa and 437kPa respectively. The trend observed was that the addition of 1% and 3% OPC significantly improved the stiffness and strength properties of the CAEMs. This was observed for all CAEMs.

Overall, Category 1: 100% VA had better stiffness and strength properties followed by Categories 2 and 3. Category 4 had the lowest average values in stiffness and strength in comparison to the other CAEMs. This could be as a result of weaker aggregates in Category 4 due to its high RAP content in comparison to the other categories. Increasing the mixing and compaction temperature resulted in an increase in the stiffness and strength properties of the CAEMs. The need for a suitable blend between RAP and VA in order to obtain optimal mechanical and performance properties when utilising CAEMs is crucial. The CAEMs had improved stiffness and strength properties after long term duration of 365 days at 20°C and 40°C with figures in similar range as HMA mixtures.

8.2.3.2 Fatigue Properties

It could be stated that the residual binder content in the RAP CAEMs is having a positive influence on the resistance to fatigue of the mixtures. Category 1 CAEMs were most sensitive to stress changes and demonstrated the lowest resistance to fatigue. Category 2 and 3 CAEMs within the context of this study performed better than Categories 1 and 4 CAEMs at strain levels less than 200με.

The inclusion of cement to the CAEMs improved the resistance to fatigue of the mixtures at both 1% and 3% OPC at strains less than 200με. In all cases, the HMA had better fatigue characteristics as expected.

8.2.3.3 Deformation Characteristics

The results from the VRLAT clearly showed that an increase in the mixing and compaction temperature positively influenced the deformation behaviour of the CAEMs. There was a positive influence on the ability of the CAEMs to resist deformation with the addition of 1% OPC. Wheel Tracking Tests (WTT) was conducted in order to ascertain the susceptibility of the CAEMs under wheel tracking. The results showed that temperature plays a critical role in permanent deformation susceptibility of mixtures. As the temperature increased, there was an increase in the susceptibility of the beams to fatigue cracking at multiple points usually starting from the CAEM layer propagating up to the HMA layer leading to eventual disintegration. It was observed that an increase in overlay thickness resulted in less tensile stresses and strains at the bottom of the HMA surfacing layer.

8.2.4 Structural Analysis

Structural analysis was conducted using KENLAYER due to its ability to effectively compute mechanistic responses of non-linear elastic materials such as the CAEMs investigated in this research. It was observed that the maximum vertical and horizontal strains were towards the middle of the sub-divided CAEM base layers as opposed to the bottom. The design charts showed that an increase in the thickness of the base course and surfacing layer resulted in an increase in the overall fatigue life of the pavement structure. The RAP CAEMs generally had lower horizontal tensile strain values in comparison to Category 1 which comprised of virgin aggregate CAEMs. This could be attributed to the influence of the residual binder in the RAP with Category 4 having the highest values although it performed the worst in stiffness and strength. The inclusion of OPC improves the mechanical and performance properties of the CAEMs although it should be noted as observed in Chapter 5 that the inclusion of OPC reduces the flexibility of the CAEMs which results in brittle behavioural patterns resulting in the sudden split of the CAEMs soon after crack initiation which is pronounced when the CAEMs are at elevated strain levels (>200$\mu\varepsilon$).

8.3 Recommendations for Future Research

This section discusses areas of research that could proffer more in-depth knowledge and understanding for CAEMs incorporating high contents of RAP. Firstly, through this research study at the University of Nottingham, it would be recommended that different types, gradations and sources of aggregate materials be investigated in order to improve on this study. Currently, there are significant advancements in the manufacture of bitumen emulsions. Further research could be done on the use of various types of bitumen emulsions including polymer modified bitumen emulsions to produce CAEMs at high RAP contents in order to investigate two key areas that currently affect CAEMs: Early life strength and their long term mechanical and performance properties including durability issues. Can its use answer the research question: How does the use of polymer modified bitumen emulsions to produce CAEMs at high RAP contents influence its mechanical and performance properties? Further research is needed to investigate how curing regimes influence the ageing characteristics of binder used in producing the CAEMs. This study identified the increased benefits associated with the inclusion of 1% and 3% OPC to the CAEMs.

One major concern was the reduced flexibility of the CAEMs resulting in the brittle and sudden failure of the CAEMs at about 200με. Key areas for future research could investigate the use of alternative fillers such as hydrated lime, pulverised fuel ash and granulated blast furnace slag especially at high RAP contents. It would be important to ascertain their mechanical and performance properties especially taking into account the water susceptibility of the produced CAEMs. Would the inclusion of 1-3% rubber crumbs improve the reduced flexibility of the CAEMs when OPC is added to the mix? Future research is needed to investigate and identify appropriate shift factors that could be applied for CAEMs to effectively bridge the gap between laboratory fatigue tests and field conditions. It is advised that further work should be done to obtain actual resilient modulus results for these mixtures. This would be vital in verifying obtained results and observed trends.

The Wheel Tracking Test (WTT) conducted provided insightful data on the behaviour of CAEMs with varying overlay thicknesses. It is recommended that this test be expanded using different variations of load and rubber thickness. It is necessary for future research to make use of improved instrumentation of the WTT to include measurement of rut profile and development of models to account for crack initiation and propagation of CAEMs. Finally, there is the need for large scale testing such as the Pavement Test Facility (PTF), the Model Mobile Load Simulator (MMLS3) and possibly field trials to assess the performance of CAEMs.

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