A STUDY ON THE DEVELOPMENT OF GUIDELINES FOR THE PRODUCTION OF BITUMEN EMULSION STABILISED RAPS FOR ROADS IN THE TROPICS

By

OKE OLUWASEYI 'LANRE

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Abstract

Eco-friendliness, energy efficiency and cost effectiveness are major drivers responsible for cold recycled asphalt mixtures being considered as alternatives to hot mixtures. Although such mixtures are still regarded in some quarters as second class asphalt, results from field trials on such materials under temperate climates have been reported to be highly impressive and encouraging. Some developed countries with temperate climates have since developed guidelines for the production and use of cold mixtures in road building. However, evidence from the literature shows that little or nothing has been done to ascertain the performance and suitability of such sustainable materials in developing countries located in hot tropical climates. Ascertaining the performances of such will, among other things enable the formulation of guidelines required for producing and using these alternative sustainable materials and methods in developing countries with hot tropical climates, where available funds for road building are increasingly inadequate to meet demand.

The work reported in this thesis attempts to simulate what should be expected in terms of the performance of flexible pavements containing cold mixes of bitumen emulsion stabilized RAP as road base in hot climates. Cold recycling in-plant was deemed appropriate for the obvious reason that it enables control of the quality of mixtures produced. The challenge of sourcing severely aged RAPs required for this study afforded the opportunity of developing a laboratory ageing protocol for producing RAPs with controlled properties, typical of those found in hot tropical belts (with residual binders of very low penetration). The result of the physico-chemical and rheological studies showed that ageing hot mix asphalt at 125ºC does not degrade the binder when compared to that aged at 85ºC, which is the conventional protocol (for temperate climates).

A target mix design based on Overseas Road Notes (ORN) 19 and 31 for 20mm DBM, which the literature suggests is suitable for road base layers of road pavements, yielded an aggregate gradation containing RAP (with residual bitumen of 20dmm penetration), 5mm granite dust and granite mineral filler in the proportion 65:30:5 respectively. Further investigations patterned after Marshall and Hveem mix design methods, indicated that a cationic bitumen emulsion content of 6.5% and pre-wetting water content of 1.5% were suitable. Unlike hot mixtures, cold mixtures due to their peculiarity i.e. intermediate nature (close to unbound granular materials in early life and close to fully bound materials when fully cured), require curing before being assessed for mechanical properties such as stiffness, strength etc.

Performances of the five cold bituminous emulsions mixtures (CBEMs), one with 100% virgin aggregate, the others including RAP with binder penetrations 5, 10, 15 and 20dmm, manufactured at 20ºC and 32ºC (to reflect average minimum and maximum temperatures in hot tropical climates) showed that:

- Properties of CBEM are dependent on the state of curing or maturation attained i.e. early life, intermediate life and fully cured or stable condition;
- High air void content in CBEMs appears to be inevitable;
- Mixing and compaction temperature is very important for achieving relatively low air void contents in CBEMs. For example, mixing and compacting CBEMs at 32ºC gave better results than at 20ºC;
- Indirect Tensile Stiffness Modulus is useful for quickly ranking the CBEMs;

- The RAP CBEMs performed better than the virgin aggregate CBEM in terms of water susceptibility;
- An increase in stiffness modulus up the range from 10dmmCBEM to 15dmmCBEM and to 20dmmCBEM, with higher values than the virgin aggregate CBEM as observed in this work gives the impression that the residual binder in the studied RAPs is active as a result of possible softening or rejuvenation. Alternatively, the stiffness enhancements could possibly have been caused by the alteration of the volumetrics of such RAP CBEMs which consequently enhanced their compactability;
- Overall, RAP CBEMs are better than virgin aggregate CBEM in mechanical performance and durability;
- Fatigue lives of the CBEMs are generally lower than those for hot mix asphalt (HMA);
- The CBEMs are stress-dependent as they all fitted the k-Θ model.

The results of the analytical pavement design showed the importance of using tools such as KENLAYER which account for the non linearity of CBEMs. Although the structural design was a hypothetical case, the results confirmed that the virgin aggregate CBEM was inferior in terms of axle loads to failure compared to the RAP CBEMs, and the RAP CBEMs were inferior to HMA.

The findings of this limited investigation suggest that the studied RAP CBEMs are suitable for low volume traffic roads, an indication of the great potential of these sustainable materials when properly harnessed. In the light of this, a short and concise set of guidelines for mix design of RAP CBEMs and structural design of pavements containing such non linear materials was proposed in the thesis.

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Declaration

The research reported in this thesis was conducted at the University of Nottingham, Department of Civil Engineering, Nottingham Transportation Engineering Centre, between October 2007 and October 2010. I declare that the work is my own and has not been submitted for a degree at another university.

Oke, Oluwaseyi ‘Lanre
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1 Introduction

1.1 Overview

Huge sums of money are being committed to the development of roads generally in the developing countries of the world. Smith (1998) reported that over £10 billion are being spent annually on roads in the developing countries, and much of this on road rehabilitation. About ten years after Smith's submission, the situation has not changed, as UNECA (2007) in recent estimates has it that the yearly infrastructural investment requirements in Africa are in excess of USD250 billion over the next 10 years. In Nigeria alone, while the Bureau of Public Enterprises (BPE), disclosed that conservatively, about ₦300 billion (£1.5 billion) would be required to standardise the nation's 34,123 km federal roads within the next ten years (Aderinokun, 2008), the Federal Road Maintenance Agency (FERMA), Nigeria recently reported that ₦3 trillion (£12.5 billion) is required to complete ongoing road projects in Nigeria (FERMA, 2010). These amounts are justifiable, and in fact should be reviewed upwards as the Nigerian road network for example, which is about 194,200km (33% of which is in asphalt) with an estimated total value of about ₦4.6 trillion (£19.2 billion) is indeed one of the nation's single largest assets (Idowu, 2000; and FERMA, 2010). These assets are to be maintained and kept serviceable all the time since road transport is the preferred means of land transport the world over (UNESCAP, 2006; UNECA, 2007; BPE, 2008; and Zammatoro, 2008).

It was during the oil boom years of the seventies, that most developing/oil producing nations of the world witnessed a period of unprecedented road construction. Nigeria was not left out; in fact, the greater percentage of the road network was put in place at that time (Idowu, 2000; and FERMA, 2010). Many of those roads have, now already passed the end of their design lives since they were not designed to withstand today's traffic loadings, while some of those that were, have since failed prematurely due to a number of reasons. Shortly after the oil windfall in Nigeria, the economy nosedived and never showed any sign of recovery until recently, and as such, budgetary provisions for road maintenance and rehabilitation had for a while been dwindling. To buttress this fact, Nnanna (2003) reported that, some roads constructed about 32 years ago in Nigeria have not been rehabilitated once, resulting in major cracks, depressions, broken down bridges and numerous potholes that make road transport slow and unsafe. The state of Nigerian roads has remained poor ever since despite the desperate efforts being made of late to revamp the road network. The roads are deteriorating faster than ever presumed (FERMA, 2007), since the methods of rehabilitation and strengthening of the road network currently being employed are the old methods of total reconstruction and overlays which are very expensive, time consuming and non eco-efficient. Of course this is understandable because the available fund for such exercises is less than adequate for fixing the roads (Nnanna, 2003). UNECA (2007) recently confirmed this assertion that costs for infrastructural renewal and expansion of roads clearly exceeds the capacity of African countries!

In a bid to salvage the situation of the road network in Nigeria, BPE (2008) reported that the Government had initiated a programme aimed at reforming the road sector, which would thus facilitate its total recovery from its poor state. It is heart warming that, among other things, a reviewing and updating of the Federal Ministry of Works standards with emphasis on the Highway Manual Part 1 Design FMW 1973 will be carried out. However, this exercise might not yield significant results, unless provisions are made in the new standard, and also backed with legislation which require the use of innovative methods and alternative materials for road construction and rehabilitation that promote sustainable development which the old standard is bereft of. Good examples of such sustainable methods and materials are cold recycling, reclaimed asphalt pavement (RAP) and bitumen emulsion respectively. It is now a general belief in most developed countries that recycling assists in stretching road funds since old materials are reused and less energy is consumed in the process (PIARC, 2003; and ARRA, 2001). The economic, safety and environmental benefits that these tools (materials and methods) offer would greatly assist in strengthening/overhauling and putting the entire road network into a serviceable condition within a reasonable time if embraced. More importantly, the recent increases in crude oil prices to the extent that a barrel of crude oil, the major source of bitumens (Asphalt Institute and Asphalt Emulsion Manufacturers Association, 1997) and also an essential ingredient for making flexible pavements, reached an all time high of USD146.69 (Gorondi, 2008), should make these materials and methods highly attractive. Even more importantly, total expenditure compared to conventional practices for road rehabilitation could be about 40-60% less when cold recycling of roads is adopted (Hakim and Fergusson, 2010; Thanaya, 2003; FHWA, 1997).

On this note, a study aimed at developing guidelines for the production of bitumen emulsion stabilised RAPs for roads in the developing countries of the world located in hot tropical belts will not be out of place.

1.2 Statement of Problem

In the developed countries of the world of which most have temperate climates, emulsified bitumen had variously been used in times past as the binding medium for surface treatment works in which fresh aggregates were used (PIARC, 2008; Thanaya, 2003; Ibrahim, 1998; Asphalt Institute and Asphalt Emulsion Manufacturers Association, 1997; and Needham, 1996). It had also been used on several occasions in cold recycling; most especially full depth reclamation works (ARRA, 2001; and PIARC, 2003). ARRA (2001) also reported further that, in the developed countries, cold asphalt recycling and the use of RAP date back to the 1900s. Results from field trials of such cold mixes under these climates have been reported to be highly impressive and encouraging.

Notwithstanding, these materials and methods for road rehabilitation are still being met with a cold attitude by road managers/developers in some of these developed countries, notably UK, Italy (Brown and Needham, 2000; and Bocci et al, 2002) among others, mainly due to some inherent problems associated with the qualities of the pavements produced in the process which cause them to be regarded as 'inferior' to conventional Hot Mix Asphalt Pavements (Needham, 1996). In fact, Carswell (2004), in a review, reported that some rate such materials and methods as 'Asphalt 2nd Class'.

On the other hand, developing countries of the world most especially those located in Africa (though with the exemption of South Africa (PIARC, 2008, Jenkins, 2000)), such as Nigeria, Ghana, Benin, Burkina Faso, Republic of Congo, among others are yet to realize the advantages of these sustainable materials, tools and techniques for road restoration (Eyo, 2004). The climate (hot) of these parts of the world promises to be an enhancing factor for such materials and methods over what obtains in other places, as most researchers are of the opinion that the presence of water in cold mixes is the main inhibitive factor to the development of early life strength in pavements made from such cold mixes, while roads are almost always desired by road engineers and users alike to be available for use immediately after construction.

PIARC (2008) in a recent survey involving 15 countries from five continents, i.e., Africa (3), Asia (2), Australia (1), Europe (8) and North America (1), identified some of the factors which are barriers to the uptake of recycling in the road construction industry as:

1. Lack of client awareness of the benefits and performance of recycling,
2. Lack of necessary regulation and legislation to encourage recycling,
3. Lack of appropriate standards and specifications,

4. Existing test methods which are unsuitable for alternative materials,
5. Lack of appropriate quality controls (concerns over the reliability and quality control of new methods and alternative materials),
6. The economics of recycling (the perception that new methods and materials will be more expensive than traditional ones),
7. Conditions of contract (conditions of contract which do not encourage innovation or flexibility),
8. Supply and demand (the difficulty of balancing supply and demand for alternative materials),
9. Planning (difficulty getting planning permission for recycling centres in or near urban areas),
10. Environmental concerns (concerns about pollution of the environment through leachate or dust generation).

While issues 1, 2, 6, 7, 8, 9 and 10 above are essentially non-technical, 3, 4 and 5 are technical, and must be determined through research and experience in countries where they are lacking. This research work will endeavour to address some of the problems associated with the identified technical problems i.e. factors 3 and 4 mentioned above, which have been hindering cold mixes from being fully embraced in developing countries. The main hindrance to the implementation of cold recycling and pavement recycling generally in such countries is lack of guidelines and standards that would facilitate a smooth implementation.

It is hoped that guidelines for the production of cold mixes for roads in such countries should succeed this present study. Many researchers from works carried out mainly in the developed countries of the world have reported the major problems with cold mix as, low and slow development of stiffness of mix. These problems have been associated with lack of proper understanding of the interaction between the aged binder in the reclaimed asphalt pavement and the recycling agent applied during recycling and also slow curing of mix and high voids contents (Thanaya, 2007; Thanaya, 2003; Ibrahim, 1998; and Needham, 1996) just to mention a few. Some of these problems, which are directly or indirectly tied to the high water content of the mix and binder properties, could potentially be alleviated by virtue of the prevailing climatic conditions (hot climate) in most of these developing countries. Also, the advantage that easier logistics are involved in the delivery of cold mix (Needham, 1996) compared to hot mix makes it more appropriate for road building in such developing countries where most areas are to say the least still relatively remote.

Thus, this study will simulate what should be expected in terms of the performance of pavements produced from cold mixes of bitumen emulsion stabilized reclaimed asphalt pavement (RAP) with a focus on hot climates. It is envisaged that at the end of this research work, the results and findings obtained should facilitate the development of guidelines for the production of bitumen emulsion stabilised RAPs for roads in the hot tropical belt of the world, paralleling those developed in the UK that provide a framework which encourages both innovation and the use of recycled materials (PIARC, 2008).

1.3 Aims and Objectives

The primary objective of this research is to examine the performance of bitumen emulsion stabilised reclaimed asphalt pavement (RAP), with the expectation of establishing a practical procedure for the use of reclaimed asphalt pavement (RAP) in road base construction in Nigeria and other hot tropical climates of the world. The specific task will basically involve a comprehensive laboratory investigation. Thus in order to meet the primary objective of this research, the following tasks will be carried out:

1. A literature review/search on asphalt pavement recycling followed by recommendations on the most appropriate method for the scenario being considered i.e. hot climate and developing countries;
2. Develop an efficient and effective ageing protocol for bituminous mixtures that will facilitate easy mass production of RAPs which replicate in the laboratory both in condition and quality what obtains in a severely aged asphalt pavement typical of developing countries located in the hot tropical belts of the worlds;
3. Implement a cold mix design for mixes made from severely aged RAPs, and bitumen emulsion, suitable for road pavements in developing countries with hot climates;
4. Investigate the resistance of the developed cold recycled mix to fatigue cracking, permanent deformation, and durability related defects through modified tests in the Nottingham Asphalt Tester (NAT) both in the unconfined and confined testing modes, that simulate conditions that pavements are normally exposed to in developing countries with hot climates;
5. By involving RAPs with different stages of ageing severity in 3 and 4 above, establish whether the residual bitumen in severely aged RAPs are in any way contributing to the performance of cold recycled mixtures at all;
6. Conduct a pavement design (structural design) integrating the results obtained from performance tests in 3-5 for the purposes of predicting performance in service over time for pavements incorporating cold recycled mixture.

The results obtained during the study would be used to develop guidelines for the production of cold mixes suitable for hot climates. Cut-off points and the categorisation of RAP qualities that suits the construction of surfacing and base courses in hot climates would be defined. Accomplishing these tasks should enable meeting the primary objective of this research work as they should ultimately assist in the Development of Guidelines for the Production of Bitumen Emulsion Stabilised RAPs for Roads in the Tropics.

1.7 Scope of Study

The present study is limited to cold recycling *ex situ* with RAPs made from granite and bitumen emulsion (cationic bitumen emulsion) as recycling agent, while cold mixtures purely made from virgin aggregates and binder will be used as a control for comparative analysis of results. Granite will be used as the aggregates because it is the material that is normally used for road building in Nigeria- which is the case study. For this reason and compatibility, a cationic type of emulsion will be used for the research work. The RAP used in the study has been produced in the laboratory. This should mimic/simulate what is obtained in a typical hot tropical climate RAP and to an extent also assist in bringing to the barest minimum the variations that could occur as a result of sourcing RAP materials from industry. Samples will be tested under conditions characteristically typical of hot (tropical) climates. The proposed investigative activities will essentially be limited to the laboratory.

The Asphalt Institute- MS 19, the Asphalt Recycling and Reclaiming Association Basic Asphalt Recycling Manual, the World Road Association (PIARC) Pavement Recycling Guidelines and relevant Transport Research Laboratory documents for hot climates, will all be used as guides for the cold mix designs that will be used in the research work.

Since the major defects that are prevalent on the Nigerian road network are due to fatigue and permanent deformation, the tests that will be conducted during this research work to examine the performance of the cold mixes will be the relevant tests in the NAT. Similarly, a test that partly addresses durability i.e. water susceptibility will be conducted.

1.8 Justification and Relevance of Study

As will be discovered in succeeding chapters of this thesis, much has been done in the developed countries of the world, of which most have temperate climates, to ascertain the performance of cold bituminous emulsion mixes made from virgin materials. Similarly, though on a lower scale, the use of reclaimed asphalt pavement (RAP) under such circumstances has also received attention. Some of these countries have since developed guidelines for the production of cold mixes.

Results from field trials of such cold mixes under such climates have been reported to be highly impressive and encouraging, though, inherent problems such as low early life strength and high porosity which are related to high water content of such cold mixes, are still to be addressed.

However, little or nothing has been done to ascertain the performance of such cold mixes or cold recycling of asphalt pavements in developing countries with hot tropical climates. Ascertaining the performances of these mixes will, among other things, assist in the integration of such sustainable construction methods and materials into road building practices in the developing countries, where available funds for road building are increasingly inadequate to meet demand. Apart from assisting in stretching funds available for roads, they have generally been acclaimed to be both energy efficient and eco-friendly when used for road construction or upgrading.

This current work is thus an exercise tailored towards filling the gap in knowledge in respect of this area of need which has received little or no attention. Laboratory investigations which simulate/predict what should be expected in terms of the performance of pavements produced from cold mixes of bitumen emulsion stabilized RAP in hot climates will be conducted. More importantly, the goal is that, at the end of this research work, findings will culminate in the development of guidelines for the production of bitumen emulsion stabilised RAPs suitable for roads in hot tropical climates of the world.

1.9 Thesis Organisation

There are nine chapters in all in this Thesis with this present **Chapter** being the **1st**. This Chapter opened with an overview in which background information for this study was given. This was followed by a statement of the problem and subsequently by the aims and objectives of the research work. Among other important things, a justification was given for the conduct of the research work.

Chapter Two is basically a literature review that discusses pavement engineering and current practices with a special focus on Nigeria. The chapter begins similarly with an overview and then followed by a discussion on common pavement design methods and the current practice in Nigeria. An effort was similarly made to identify materials being used for roads in Nigeria, and common pavement defects and their causes. Road restoration practices were also discussed. Among other important things, road recycling is introduced as a sustainable road restoration practice, and bitumen emulsion is properly elaborated on.

Chapter Three similarly is a literature review but now focused on cold mixes. The chapter opens by similarly giving an overview information which leads to an explanation of the process and causes of ageing in bituminous material. Methods of laboratory ageing of bituminous materials are also reviewed. Some of the other important issues discussed are directly related to the general performance of cold mixtures.

Chapter Four essentially reports the results of the procedure used for laboratory ageing of bituminous mixtures that was developed in this work.

Chapter Five discusses cold mix design as carried out in this study. Results of the preliminary investigations are reported in the chapter.

Chapter Six details the results of the performance of the cold mixed bitumen emulsion mixtures after examining the materials using tests such as Indirect Tensile Stiffness Modulus, fatigue (stress controlled) etc.

In **Chapter Seven**, the results of the pavement design (structural design) are reported.

Chapter Eight lists the recommended guidelines based on the results reported earlier in Chapters 4, 5 and 6. This is done with due regard to the experiences of other researchers.

The last **Chapter** of this thesis is the **9th** which details the conclusions and recommendations made based on the entire study.

2 A Review on Flexible Pavement Engineering and Current Practices

2.1 Overview

Ellis (1979) defined highway pavement engineering as, the process of designing, constructing, and maintaining highway pavements in order to provide a desired level of service for traffic. Fwa (2003) gave a clearer picture in which he stated that, pavements are designed and constructed to provide durable all-weather travelling surfaces for safe and speedy movement of people and goods with an acceptable level of comfort to users. He further opined that these functional requirements of pavements are achieved through careful considerations in the following aspects during the design and construction phases: (a) selection of pavement type, (b) selection of materials to be used for various pavement layers and treatment of subgrade soils, (c) structural thickness design for pavement layers, (d) subsurface drainage design for the pavement system, (e) surface drainage and geometric design, and (f) ridability (capable of being ridden over) of the pavement surface. However, the two major considerations in the structural design of highway and airport pavements are material design and thickness design.

This chapter essentially reviews issues that impinge on flexible pavement engineering as it relates to both developed and developing countries. Ellis (1979) in his work reported that the important differences between pavement engineering in developing countries and industrialised countries are the greater variability of construction materials, quality of construction, and the larger fluctuations in the volume and weight of road traffic that are typically encountered in developing countries. Meanwhile, current practices in developing countries compared to the developed ones for road pavement restoration/rehabilitation/strengthening, and the need to integrate alternative methods and materials for pavement engineering among them have been discussed.

2.2 Common Flexible Pavement Design Methods

It is known generally that civil engineering structures are designed to, be safe, aesthetically appealing, function well and be economical. In a broader sense, Haas et al (1994) listed the basic objectives of a pavement design process as:

1. Maximum economy, safety, and serviceability over the design period,
2. Maximum or adequate load-carrying capacity in terms of load magnitude and repetitions,
3. Minimum or limited deterioration over the design period,
4. Minimum or limited noise or air pollution during construction,
5. Minimum or limited disruption of adjoining land use,
6. Maximum or good aesthetics.

Das (2005) summarised the evolving trend in the basic objectives of a mix design for flexible pavements in Table 2.1. Mamlouk (2006) in his work stated that many methods have been advanced for the design of flexible pavements. These methods have been described to range from very simple to highly sophisticated in nature. Pavement design methods can be grouped into four distinct approaches namely:

1. Methods based on experience (old, based on previous experience);
2. Methods based on soil formula or simple strength tests (old as well, and generally assumes that pavement is supported by the subgrade and that all other layers are required for maintaining smoothness and control of dust);
3. Methods based on statistical evaluation of pavement performance (empirically generated and thus limited to the conditions under which they were generated) and;
4. Methods based on structural analysis of layered systems (most fundamental approach as basic material responses such as stresses, strains and deformations are considered. Though this method is reliable and accurate most of the time, it has the disadvantage of requiring extensive testing and computations).

In an earlier work, RILEM 17 (1998) categorised all the available approaches as shown in Table 2.2. Out of these, many countries have adopted approaches which have evolved through individual experiences. Some are highlighted in Table 2.3. However, recently, PIARC (2008) and Das (2005) reported that trends have since started tilting towards performance related and performance based approaches. It is worth noting that most or all of these methods are specifically meant for hot mixtures (HMA). Cold mixtures have received attention too though not as much as HMA. Most cold mix design methods are similar to those for HMA with no universally accepted method or procedure. Examples of guidelines already in use for cold mix design are, Basic Asphalt Recycling Manual (ARRA, 2001), TG2 (Asphalt Academy, 2009), Wirtgen Cold Recycling Manual (Wirtgen, 2004), MS-14 and MS-19 (Asphalt Institute, 1989 and 1997). These are all based on Hveem and Marshall methods.

Table 2.1: Requirements of Bituminous Mix Design (Das, 2005)

Past	Present
Stability Durability Economy	Stiffness Permanent Deformation Fatigue Temperature susceptibility Low temperature cracking Moisture susceptibility Freeze-thaw Permeability Economical Environment friendly Workability Economy

Table 2.2: Various Mix Design Approaches (RILEM 17, 1998)

Mix Design Method	Description
Recipe method	Recipe 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.
Empirical mix design method	In empirical mix design methods, optimization of several variables is done by mechanical testing, taking into account some specifications as limits which evolved through prior experience. Variables considered in this approach may not be used as direct measures of performance.
Analytical method	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 analyzed in a mixture, which is compacted in the laboratory by some procedure close to the field compaction process.
Performance related approach	In performance related mix design, 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	A performance based 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 2.3: Mix Design Approaches Adopted in Various Specifications/Organisations (RILEM 17, 1998)

Specification/organization	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

2.3 Flexible Pavement Design Practice in Nigeria

Nnanna (2003) reported that the road system in Nigeria (Latitude 9º 4'55.20''N, Longitude 8º 40'31.00"E) is classified into four categories namely:

(a) The Federal Trunk 'A' Roads- These are under the Federal government and they are developed and maintained by the Federal Government;
(b) The Federal Trunk 'F' Roads- These were formerly under state ownership, but were taken over by the Federal Government. With a view to upgrading them to Federal highway standards;
(c) The State Trunk 'B' Roads- These are under the ownership and management of the component States;
(d) The local Government Trunk 'C' Roads- These are under Local Government ownership and management.

The design and construction approach to be employed when a road is to be built in Nigeria depends on the category of the proposed road. The importance attached increases from Trunk 'C' Roads to Trunk 'A' Roads as listed above. However, the method of flexible pavement design in Nigeria is not dissimilar to what obtains in other tropical zones of the world. The design basically is empirical in nature, and is based on the procedures and specifications outlined in the manual: Highway Manual Part I: Design of 1973, produced by the Federal Ministry of Works and Housing, Nigeria. Normally after the CBR value for the subgrade and the estimate of traffic have been determined, the thickness of the pavement structure can be determined from the Chart shown in Figure 2.1. This manual is normally used in conjunction with relevant British Standards, Overseas Road Notes (TRRL Road Note 31 in particular), American Society for Testing and Materials (ASTM) design manuals, American Association of State Highway and Transport Officials (AASHTO) design manuals, The Asphalt Institute design manuals, and other reference materials.

Meanwhile, Ellis (1979) stated that very few of the various methods of pavement design that are in general use throughout the world have been devised specifically for the design of pavements in tropical countries. Two exceptions are TRRL Road Note 31 (suitable for the design of medium and lightly trafficked roads), and the French CEBTP design manual for tropical roads. Other popular methods of pavement design, such as the AASHTO method and its derivatives, though derived empirically in industrialised countries with temperate climates, are nevertheless often used for the design of pavements in the tropics. There are very large differences between the designs produced by various methods of pavement design, even when the same assumptions of subgrade strength and traffic loading are made.

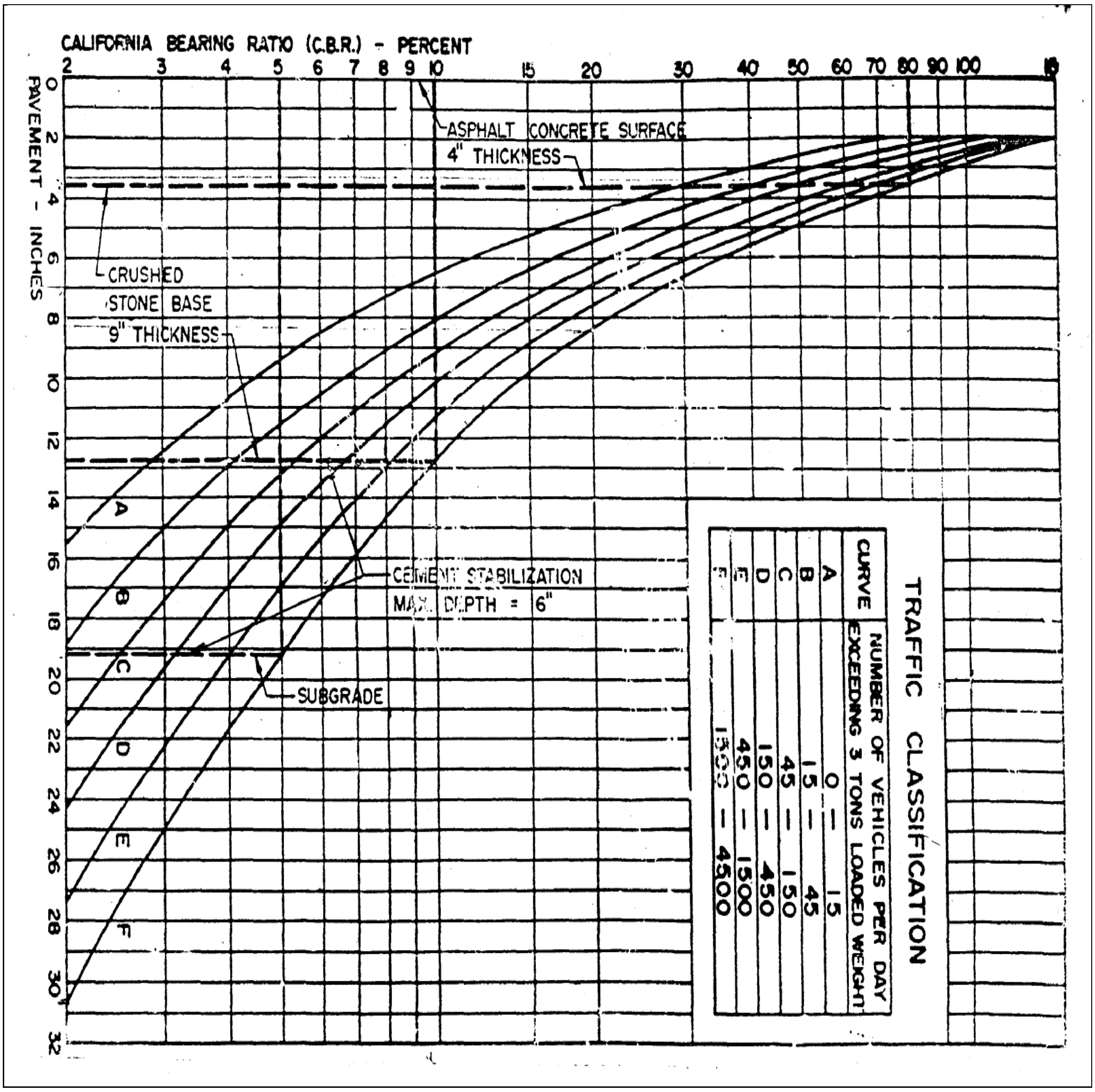


Figure 2.1: Flexible Pavement Design Curve (FMWH, 1973)

Thus, the specifications/procedures in such manuals are applied by highway engineers in Nigeria with caution bearing in mind that these manuals were produced mainly based on the experiences of highway engineers in the developed countries of the temperate regions of the world in the first instance. A few of the factors that readily come to mind when designing a road in Nigeria are; climatic conditions, wide variability in the characteristics of available highway materials, uncontrolled axle loads, overuse of the road network and improper/lack of maintenance. Pavement temperature in hot climates for example has been reported as high as 59 ºC (Johnston and Gandy, 1964).

These are very dissimilar to what obtains in the developed countries of the temperate regions of the world. Generally, because of the large variability of these factors, Ellis (1979) suggested that the design of road pavements in developing countries must either include a higher 'factor of safety' than is usual in industrialised countries, or a higher risk of failure must be accepted. It has been observed that the latter course is the most commonly adopted in recent times due to the fact that, it minimises the demands made by road building on scarce resources, and the disadvantages of partial or premature pavement failures are much reduced by the short 'design life' generally adopted and relative ease with which repairs can be made on typically un-congested roads (Ellis, 1979).

This buttresses the reason why most roads are now designed to last for between 4 - 15years, which is quite different from the vogue of 25years design life in the sixties and seventies. From experience, common flexible pavements in Nigeria are either Surface-Dressed or in Hot Asphalt Concrete (HMA or HMAC). Campbell (2009) reported that rigid pavements are not common in Nigeria. Surface Dressing is usually employed as a stop gap measure knowing full well it might not last more than 4years or 5years at most. Thus major designs are focused on Hot Asphalt Concrete.

2.4 Common Highway Materials in Nigeria

Common materials which constitute a typical flexible pavement in Nigeria are asphalt (Bitumen + filler + aggregate), aggregates (fine and coarse), cement, bitumen and laterite. Asphalt normally constitutes the surfacing, while aggregates and binders (cement or bitumen) are normally used for the road base. Laterite, which is abundantly available in Nigeria, is normally used for the subbase. In some cases, lateritic gravel is used as substitute for aggregates for the road base. A lot of research work has been carried out on the suitability of laterite as a road construction material in the tropics (Adedimila and Oti, 1987; Gidigasu, 1991). The following road construction materials i.e. bitumen, aggregates, asphalt and laterite, are briefly examined below within the Nigerian context.

2.4.1 Bitumen

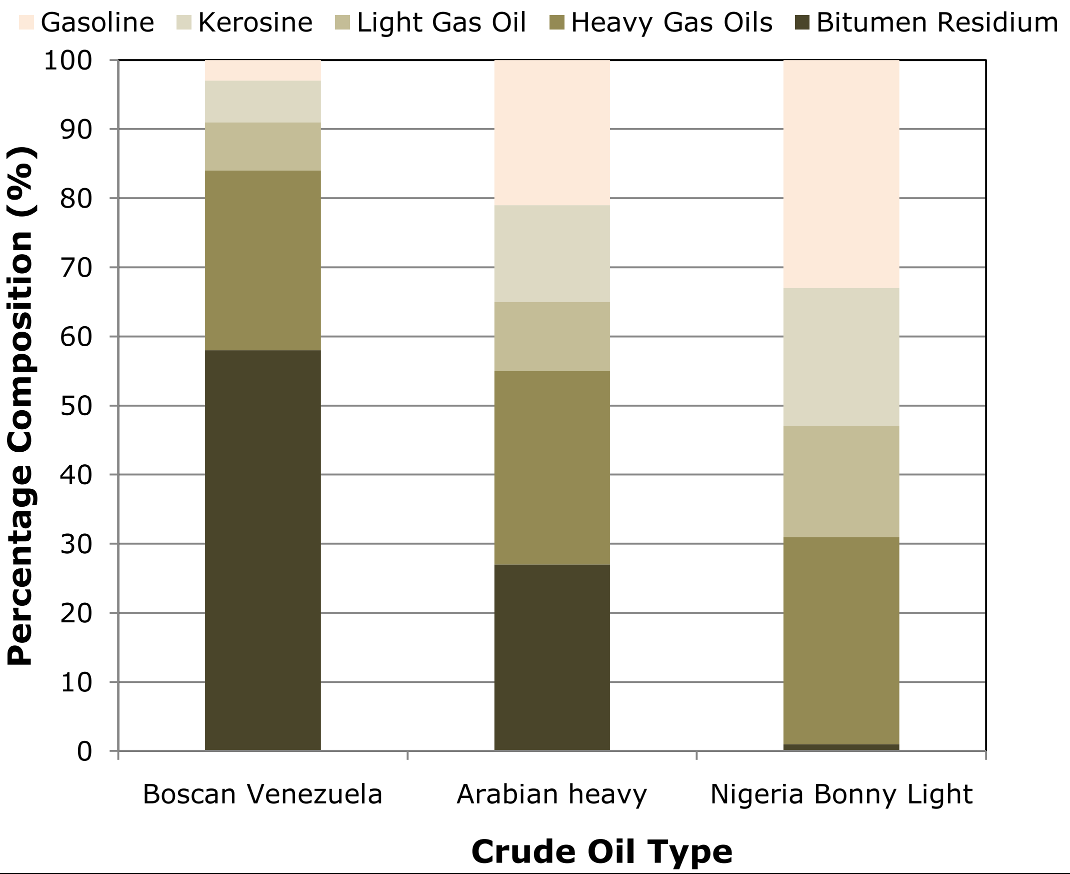
Bitumen is a mixture of organic liquids that are highly viscous, black, sticky, entirely soluble in carbon disulphide, and composed primarily of highly condensed polycyclic aromatic hydrocarbons (Bonner, 2001). While it occurs naturally on its own, it can also be obtained as a by-product of crude oil. Bitumen properties are greatly influenced by the characteristics of its components, particularly the structure of asphaltene and asphaltene/resin ratio of the bitumen sample. These in turn depend mainly on the nature of the crude oil. The suitability of a crude oil for bitumen manufacture is governed by the high weight percent of carbon residue, high specific gravity, sulphur content of more than 5 per cent by weight, low wax content and high asphaltene content. Unfortunately, the Nigerian crude oil does not meet these requirements considering among other things the American Petroleum Institute (API) gravity values and the sulphur content of a typical Nigerian crude oil residue as illustrated in Table 2.4 (Corbett and Urban, 2005). Worse still, Nigeria which presently boasts a proven 42 billion barrel of bitumen reserve has for logistical reasons not been able to tap this vast resource (Adebiyi et al, 2005). Thus in order to meet demands for road construction, Nigeria imports bitumen presently chiefly from Venezuela (Duke, 2010; and Ministry of Mines and Steel development Nigeria, 2010). The imported bitumen is normally straight run of penetration grade 40/50 or 60/70 or 85/100 (FMWH, 1997).

2.4.2 Aggregates

Raw materials for aggregates are in abundance in Nigeria. While the river sand is suitable for fine aggregates, natural gravels and crushed granitic stones are readily available to meet the demands for coarse aggregates in road construction. The quarry dusts which are the by-products of the processes involved in the production of crushed stones would easily pass for fillers and even fine aggregates as well. The bulk of the work now lies in quality control. Generally, for coarse aggregates (4.75mm and larger), the aggregate crushing value (ACV) should not be more than 30% and also the Los Angeles Abrasion Value should not be more than 40% (FMWH, 1997).

Table 2.4: Composition of Various Petroleum Crude Oils (Corbett and Urban, 2005)

Physical Properties	Crude Type		
	Boscan Venezuela*	Arabian heavy	Nigeria Bonny Light
API degrees	10.1	28.2	38.1
Specific Gravity	0.999	0.886	0.834
% Sulphur	6.4	2.8	0.2



**A good crude oil source for the production of bitumen*

2.4.3 Asphalt

Asphalt either as a surface course or binder course, forms the final layer in flexible pavement structure. It is expensive to produce and lay, and since its durability and serviceability depends on; the proportions and properties of the components of the mixture, the temperature of mixing and laying, and as well the intensity and mode of compaction, it is therefore imperative that rigid control be maintained in testing throughout the period of production. Generally, the major work here is that of quality control. When embarking on a road construction project in Nigeria, it is common practice to appoint a Materials Engineer, who ensures that the quality of all the above mentioned materials conform to the requirements of the specifications. He also has the mandate to ensure that such materials are laid and compacted just as stated in the specifications. It is common practice to use granite aggregates for the production of asphalt in Nigeria since they are abundantly available (FMWH, 1997).

2.4.4 Laterite

Gidigasu (1991) described laterite as a surface formation in hot and wet tropical areas which is enriched in iron and aluminium, and develops by intensive and long lasting weathering of the underlying parent rock. Since Nigeria is located in the tropical belt of the world, this material is abundantly available there, and pavement engineers have found it very useful in the construction of highway pavements when properly characterised. This confirms the TRRL (1993) findings that a lot of naturally occurring granular materials in the tropics can be used successfully as road base in flexible pavement construction. It is thus, essentially a material for the road base and mainly subbase layers. When firm enough, bituminous layers could be placed directly on lateritic layer. Meanwhile, TRRL (1993), Adeyemi and Oyeyemi (2000), and Gidigasu (1991) have extensively discussed factors and properties to be considered and checked respectively when laterite is to be used in road construction.

2.5 Some Common Tests for Mix Design and Quality Control

There are a number of tests normally conducted on highway materials essentially for mix design and quality control. These tests are discussed in the literature extensively (Read and Whiteoak, 2003; and Fwa, 2003). Also standards have been produced for guidance on how to conduct such tests. Table 2.5 summarises the required tests for mix design and quality control of such highway materials.

Table 2.5: Highway Materials, Relevant Tests and Standards for Quality Control and Performance Monitoring

Material	Quality Control and Performance Tests
Asphalt	Determination of Bulk Specific Gravity and Density of Mixtures, Indirect Tensile Strength (BS EN 12697-23:2003), Indirect Tensile Stiffness Modulus (DD 213:1993), Indirect Tensile Fatigue Test, Repeated Load Axial Test-Confined and Unconfined (BS DD 226:1996), Water Sensitivity Test (BS EN 12697-12:2003), Compaction by Gyratory Compactor (BS EN 12697-31:2007)
Aggregates	Aggregate Crushing Value Determination, Aggregate Gradation (BS 63), Aggregate Angularity Test, Flat and Elongated Particle Test, Clay Content Determination, Soundness Test, Determination of Atterberg Limits (Plastic Limit, Liquid Limit, Shrinkage Limit, Plasticity Index) (BS 812)
Bitumen	Softening Point Test (BS EN 1427:2000), Penetration Test (BS EN 1426:2000), Flash Point Test, Ductility Test, Solubility Test, Thin Film Oven, Viscosity (BS EN 13302:2003), Dynamic Shear Rheometer Test (BS EN 14770:2005), Bending Beam Rheometer Test, Pressure Ageing Vessel Test, Direct Tension Test, Rolling Thin Film Oven Test (BS EN 12607-1:2007)
Laterite	CBR Test, Gradation Test, Determination of Atterberg Limits (Plastic Limit, Liquid Limit, Shrinkage Limit, Plasticity Index), Soil Density, (BS 1377: Parts 1 - 9)

It is worth noting however that the Marshall test (for the determination of stability and flow) is still common in Nigeria just as in most developing countries for asphalt job mix design. The recent developments in the industry are shown in the table. Of particular interest among the tests listed for asphalt are the Indirect Tensile Stiffness Modulus (ITSM) test, Indirect Tensile Fatigue Test (ITFT), and Repeated Load Axial Test-unconfined and confined (RLAT and VRLAT). These three tests can be conducted in the Nottingham Asphalt Tester (NAT) which is becoming increasingly popular in the industry (Wu, 2009) for its versatility (useful for examining the fundamental properties of asphalt mixtures), inexpensiveness and simplicity of use.

2.6 Common Flexible Pavement Defects and their Causes

In his work, Ibrahim (1998) opined that a pavement is considered to have failed when the deformation of its components is sufficiently large to cause either an intolerably uneven riding surface or cracking of the material resulting from repeated stress over a prolonged period of time. He further identified the causes of permanent deformation as densification of materials and accumulation of shear deformation due to repeated action of shear and tensile stresses. Adedimila (1986) in an earlier work submitted that generally, pavement failure can be associated with materials or mixture failure which Holmes (1990) associated with, inadequate design or specification at the outset, failure to predict traffic loadings accurately, and failure to consider naturally occurring substrata or drainage conditions. Whereas, Gidigasu (1991) in his work reported that results of studies on failure of pavements have revealed that basic natural crushed rock paving aggregates undergo physical, chemical, physico-chemical and mineralogical changes and some pavement failures have been attributed solely to increase in secondary mineral content of rock aggregates due to further degradation.

Sterling and Zamhari (1997) argued that the causes of road failure in the tropics are cracks in pavements and ageing which are results of the embrittlement of the pavement as a result of the loss of lighter oils in the pavement binder. Dachlan et al (1997) on the other hand later found out that roads in Indonesia normally in the past suffered premature failure by cracking, but more recently they failed prematurely by permanent deformation. Adeyemi and Oyeyemi (2000) submitted that failure of flexible pavement is a common phenomenon in the tropical world and that permanent deformation is adjudged as the most common failure type.

Eyo (2005) from his experience in Nigeria, noted that unprecedented traffic load among other causes accelerates the failure of a pavement long before its appointed life span. Das (2008) in a more recent work stated that fatigue, rutting, and low temperature cracking are generally considered as the important modes of failure of a bituminous pavement structure. This position is perfectly in agreement with an earlier observation of Brown and Brunton (1986) on the mechanism of pavement failure - dominated by fatigue and rutting.

It would thus appear that most researchers are of the opinion that permanent deformation (caused by poor mix design, weak road foundation and excessive axle loads) and fatigue (caused principally by the repeated bending action of the pavement and ageing) among other factors are undeniably the major causes responsible for the deplorable state of flexible pavements. Each or a combination of these two factors eventually result in most surface and structural road defects.

Mamlouk (2006), PTCA (2005), Petts (1994) and Research and Development Division Highways Department (1992), identify some of these defects as bleeding, cracks, ruts and depressions, edge subsidence and rutting, edge damage, joint sealant defects, spalling, local aggregate loss/ravelling, potholes and shoving, aggregate polishing. Figure 2.2a shows fatigue and rutting in a pavement cross section while Figure 2.2b depicts some of the resulting defects.

2.7 Flexible Pavement Restoration Practices in some Developing Countries

In West Africa, road rehabilitation practices follow the typical conventional old methods. In Ghana for example, many hitherto surface-dressed major roads due for rehabilitation are being or have already been resurfaced with asphaltic overlays as part of a major effort to upgrade some of the country’s roads. For medium-trafficked roads, however, the Ghana Highway Authority (GHA) is adopting surface seal by double surface-dressing as an alternative to cut down the cost of construction and rehabilitation (Oduroh and Tuffor, 1996). These practices are not in any way different from what obtain in Benin, Togo, Gambia, Liberia, Ivory Coast, Nigeria, indeed in all the sixteen West African countries.

In some extreme cases, where a road has failed completely, the failed asphaltic pavements are normally scarified. The subgrade is reworked and now overlaid with fresh asphalt pavement. However, the scarified materials are dumped in nearby bushes (see Figures 2.3a and b). It is worth noting that in some cases where an asphalt pavement has experienced structural failure, though still

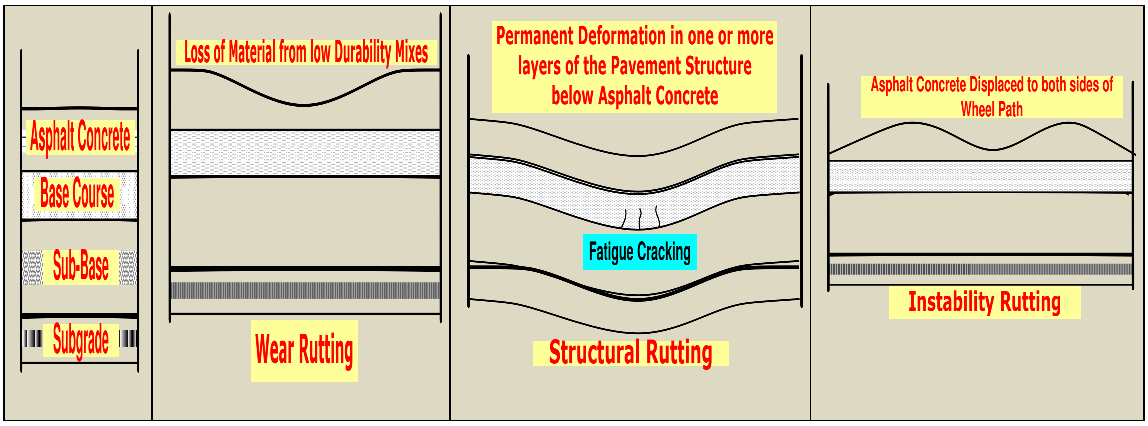
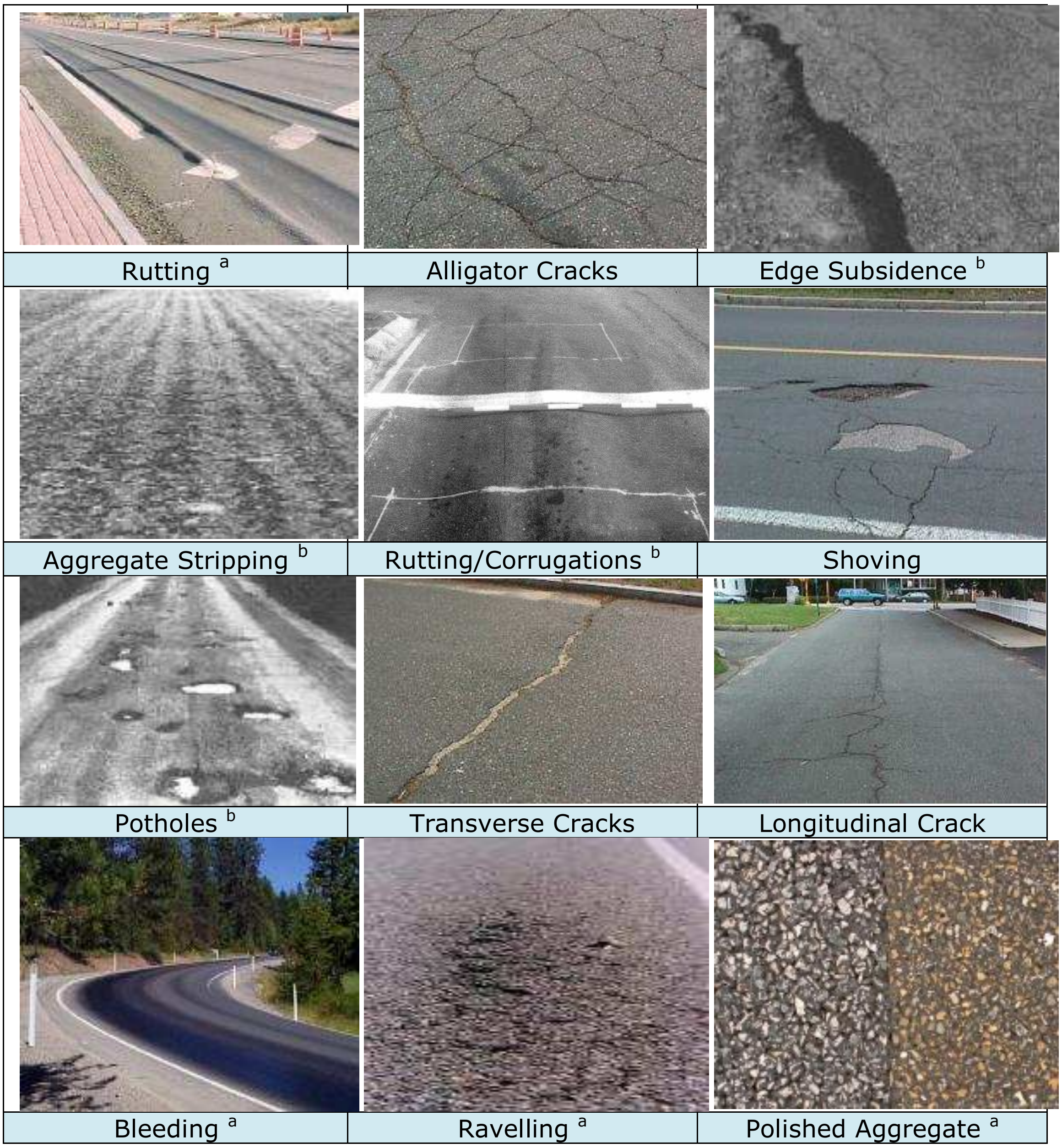


Figure 2.2a: Cross Section of Road Pavement Showing Failure Mechanisms due to Fatigue Cracking and Rutting (White et al, 2002)



a- PTCA (2005)
b- Petts (1994)

Figure 2.2b: Some Common Flexible Pavement Defects

Figure 2.3a: Ripped Asphalt Improperly Disposed Close to Farmland along Igede - Aramoko Road in Ekiti State, Nigeria

Figure 2.3b: Indiscriminate Dumping of Scarified Failed Pavement Materials in Nigeria (Eyo, 2005)

passable, an overlay is normally applied to strengthen such pavement. Eyo (2005) reported that stabilizers (liquid and powder), geotextiles and surfacing materials are presently being trialled to ascertain their suitability for road pavement rehabilitation in Nigeria.

2.8 Trends in Road Restoration Practices in Developed Countries

The present trend in road restoration practice in most developed countries of the world is recycling, and it is being driven chiefly by the resolve of the governments of these countries at enforcing the concept of sustainability in all human endeavours. This concept was introduced in Rio de Janeiro in 1992 during the United Nations Conference on Environment and Development (Roberts, 1997). Part of the recommendations made requires limiting wastes to landfills and also preserving the natural habitat as much as possible in all endeavours. Of course the road sector is not excluded from this, and as such one of the best ways of achieving this is by the reuse and recycling of construction wastes. To achieve this goal, ARRA (2001) reported that many countries have already enacted legislation which requires that certain percentages of material, particularly the ones used in roadway construction and rehabilitation must be recycled or include recycled materials. Shortage of funds needed for road management and the energy crisis among others have also been responsible for this present trend. ARRA (2001) reported that asphalt pavement is the most commonly recycled material in North America. Japan, Australia, UK and other European countries too are following this path (Merrill et al, 2004; Woodside et al, 2000; and Austroads, 2000).

2.9 State of Roads in Nigeria and other Developing Nations, and Reasons for Failure

The performance of a road cannot in any way be isolated from its history. The activities that characterized the pre-construction, construction and post construction stages will have a large bearing on the performance of such a road. Thus, roads have their individual identities and as such their performances are characteristically functions of their histories. No two roads are the same in nature. Therefore, the assessment of the performance of any road should always be premised on its history.

Normally, when a road is being designed to last for *N* years, and all things being equal during the construction, the assumption of the highway engineer is that such a road will be adequately maintained throughout its life time. This is also done knowing full well that the deterioration mode of flexible pavements could be accelerated by prevailing climatic conditions in the tropics. For example, the deterioration of bitumen is hastened when subjected to high temperatures in the presence of oxygen. This will lead to oxidation, loss of volatiles and quick ageing of asphalt. The combination of oxidation and loss of volatile effects leads to the hardening of bitumen, and consequential reduction in penetration, an increase in softening point and an increase in penetration index. These in turn make bitumen brittle at low temperatures and prone to failure by cracking (Bonner, 2001).

There are quite a number of flexible pavements in Nigeria that have and are still satisfying their intended design roles functionally and structurally. Good example of such is the Abuja municipal roads. Their outstanding performance is a consequence of the proper attention that they received during the various stages of pre-construction, construction and post construction. However, it is sad to note that the situation is different in other sections of the Nigerian road network system. Nnanna (2003) observed that most of the roads especially in the southern part of Nigeria are in very poor condition, and require complete rehabilitation. FERMA (2007) in a more recent report confirmed that most roads in Nigeria are distressed. And as such the entire highway network in the country might face a total collapse if urgent steps are not taken to rehabilitate, repair or reconstruct them at the appropriate time. It was recently estimated that fixing the federal roads alone in Nigeria will cost the sum of $20.5 billion (FERMA, 2010a), a sum just a little below the Federal Government’s annual budget.

Nigerian roads have remained poor for a number of reasons which are traceable to activities that characterized the construction and post construction stages of such roads. These mainly are in two categories: (a) Poor constructional practice and supervision, and (b) Lack of maintenance. The factors responsible are numerous, varied and diverse in nature. These have been discussed extensively elsewhere (Idowu, 2000; and Nnanna, 2003). The present condition of the Nigerian road network is as shown in Table 2.6 and Figure 2.4a.

While commenting generally on the road network in Africa, UNECA (2007), reported that, given the challenges of globalization, Africa is lagging significantly behind in the development of regional trade, particularly because of the lack of reliable and adequate transport. The existing transport facilities are completely outward-looking with the result that the transport infrastructure and services have been little developed and the physical network poorly integrated. UNECA (2009) further reported that presently, only 22.5% of the road network in Africa is paved (see Table 2.7), and that the entire network is poorly maintained.

Table 2.6: Road Network of Nigeria broken down to Federal, State and Local/Rural (Idowu, 2000)

Type of Road	Federal Road	State Road	Local Govt. Roads	Total
Paved Roads	26,500	10,400	-	36,900
Unpaved roads	5,600	20,100	-	25,700
Urban Roads	-	-	21,900	21,900
Main rural Roads	-	-	72,800	72,800
Village Access Roads	-	-	35,900	35,900
Total	**32,100**	**30,500**	**130,600**	**193,200**
Percent	**17%**	**16%**	**67%**	**100%**

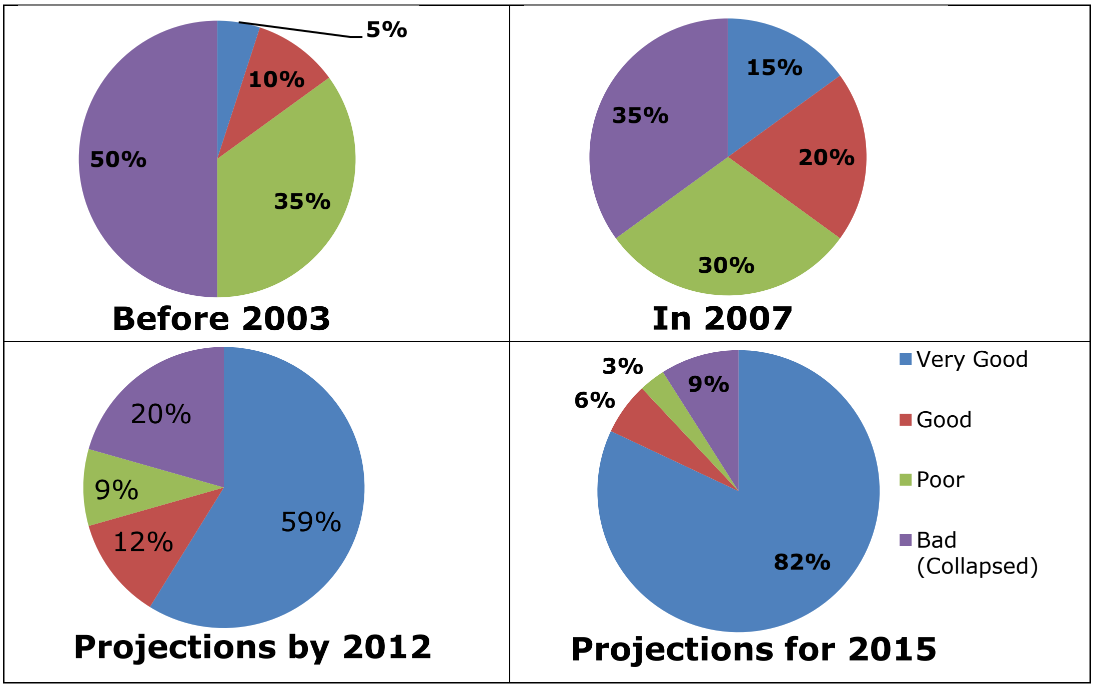


Figure 2.4a: Current and Projected Conditions (Due to Intervention) of the Nigeria Road Network (FERMA, 2010)

Table 2.7: Subregional Distribution of the Road Network and Share of Paved Road in Africa (UNECA, 2009)

Region	Land Area (Km^2)	Road Length (Km)	Road Density ($Km/100\ Km^2$)	Paved Roads (%)	Distribution /10,000 Inhabitants
Central	3,021,180	186,471	6.3	1.0	49.5
Eastern	6,755,902	527,502	8.4	10.0	18.4
North	9,301,385	400,520	4.4	49.0	21.0
Southern	6,005,240	728,834	12.3	27.0	56.3
West	5,112,060	580,066	11.5	13.0	21.5
Total	**30,195,767**	**2,423,393**	**8.3**	**22.7**	**26.0**

2.10 The Need for Sustainable Road Development

By timely and proper maintenance, highway engineers can extend a pavement's usefulness. Eventually, however, even the best-maintained pavement will begin to disintegrate and will need to be rehabilitated. The traditional approach to pavement rehabilitation has been to either reconstruct it with all new material or patch and overlay it with a new wearing surface, which is expensive, time consuming, wasteful and causes considerable inconvenience to the road user. This is clearly evident in the extension cum maintenance works going on along Lagos – Ibadan expressway in Nigeria. Apart from this, it leads to the depletion of the natural resource base essential for future development, increased hydrocarbon pollution (Parry, 2009), the need to locate tipping sites for waste road stone, and transport required to remove waste material and bring in new material can add greatly to traffic congestion, reduction of the life cycle of existing pavements and safety problems.

Additionally, a number of the processes involved in the conventional/traditional methods do not satisfactorily comply with the provisions for conservation and management of resources for development defined in Agenda 21 of the UN resolutions during its Earth Summit meeting held on Sustainable Development in Rio de Janeiro, Brazil, in 1992. For example, the provisions chiefly require among others, protection of the atmosphere, integrated approach to the planning and management of land resources, combating deforestation, managing fragile ecosystems: combating desertification and drought, managing fragile ecosystems: sustainable mountain development, promoting sustainable agriculture and rural development, conservation of biological diversity, environmentally sound management of biotechnology, environmentally sound management of solid wastes and sewage-related issues (United Nations, 1992).

These disadvantages of the traditional/conventional methods of pavement rehabilitation, and the increasing construction costs have caused highway engineers to search for new methods of rehabilitating highway pavements. However, for sustainable road development to be possible, road activities will have to be redesigned to reuse raw materials and construction materials many times over. This will include salvaging construction materials such as concrete and asphalt from roads, the reuse of metals and other natural and synthetic materials. Waste recycling and reuse will have to become a way of life in both developed and developing nations. Interestingly, the developed nations are beginning to do this but the reverse is the case in developing nations such as Nigeria. One of the most interesting examples of resource recovery is the use of recycled products as construction materials (Parry, 2009). The recycling concept is not particularly new to the highway field. Various methods, some quite successful, have been practiced in a limited manner since about 1900s (ARRA, 2001).

It is estimated that over ₦2.5 trillion (£11.4 billion) is spent annually on roads in developing countries, much of this on road rehabilitation (Smith, 1998). The Federal Government of Nigeria presently spends about ₦ 4.44 billion (£20 million) annually on direct labour allocation for minor road maintenance excluding rehabilitation/major maintenance and development of new roads. The most expensive element of the road pavement is the asphalt (bitumen + aggregate mixture) which forms the surfacing and often forms the principal load bearing layers of the road.

The cost of asphalt is typically more than 3 times the cost of unbound aggregate, but when roads are rehabilitated, the asphalt is almost always removed and discarded as a waste product. To worsen the situation, in Nigeria, the disposal is done indiscriminately (see Figures 2.3a and b) since environmental laws are not being fully enforced. Very large volumes of good quality aggregate and asphalt are therefore lost and new materials must be quarried/ procured to replace the discarded materials. The use of thick asphalt layers through overlays is (see Figure 2.4b) increasing to keep pace with expanding networks and ever greater traffic loadings. Unfortunately after several overlays have been done, the free passage of high vehicles under bridges is threatened as the head rooms under bridges keep reducing each time an overlay is applied. Thus, the need to introduce construction techniques and tools which facilitate the reuse of old/waste road materials in Nigeria and other developing countries cannot be overemphasized as they play important roles in achieving sustainable road development.

2.11 The Pavement Recycling Option for Road Restoration

The Asphalt Institute (1978) defines pavement recycling as the reworking of in-place surface and base material while PIARC (2003) defined the recycling of road materials as the reuse of existing road materials in road construction, with or without changing the characteristics of the materials. In a broader sense, recycling is reducing reclaimed materials from the road to a suitable size for processing, blending the reclaimed materials with virgin ones and relaying the materials as a base, binder or surface course. However, the ready availability of cheap aggregates and binders did not make these methods very attractive to

Figure 2.4b: Very Thick Asphalt Layer Due to Several Overlays in Ibadan, Nigeria

highway engineers and it required the 'energy crisis' which followed the 1973, Arab-Israeli conflict to re-awaken interest in this rehabilitation method (PIARC 2001; Karlsson and Isacsson, 2006). Whereas the concept of asphalt pavement recycling is relatively new in Nigeria, it has been successfully practiced since the mid 20th century in the US and Europe (Eyo, 2004). Adopting this method of road rehabilitation in Nigeria will not only engender sustainable development and large cost savings, but will serve as one of Nigeria's major local contributions to the realisation of Agenda 21; more so, any road in need of reconstruction or new overlay is a candidate for recycling (Karlsson and Isacsson, 2006).

Recycling roads provides a lot of benefits. The first and most obvious benefit is the saving in the amount of fresh materials required. This saving is wide-reaching. Less material equals less quarrying (energy reduction as well as reduced environmental impact), equals less fuel used, equals less transport (reduced damage to road network and noise pollution). In the same vein, experience has shown that the in situ recycling process is up to 50% faster than conventional reconstruction (Sinclair and Valentine, 1999). This brings benefits through reduced site supervision costs. Also, a reduced construction period minimizes traffic delays caused by road works, which contributes to the economic benefit of the process. Reusing the existing road and footway materials leads to a reduction in the waste to landfill. This produces environmental and economic benefits. Thus, this method of road reconstruction meets the requirements and provisions of Sustainable Development as stated in Principles 1 to 27 of Agenda 21 of the UN resolutions of the Earth Summit.

2.12 Methods of Flexible Pavement Recycling

Different classifications of asphalt pavement recycling have been advanced in literature. PIARC (2003) maintained that classifications of the main types of recycling can be made according to:

- The place where mixing is carried out (whether *in situ* or *ex situ i.e.* in-plant),
- The temperature of the process (cold or hot recycling),
- The characteristics of the material to be recycled,
- The binder type (cement, bitumen emulsion, foamed bitumen, bitumen).

ARRA (2001) on the other hand categorised recycling into:

- Cold Planing (CP)

- Hot Recycling (HR)
- Hot In Place Recycling (HIR)
- Cold Recycling (CR), and
- Full Depth Reclamation (FDR)

ARRA (2001) further classified Hot In-Place Recycling into- Surface Recycling (Resurfacing), Remixing and Repaving. Cold Recycling was further classified into Cold In-Place Recycling (CIR) and Cold Central Plant Recycling (CCPR), while Full Depth Reclamation was classified into, Pulverization, Mechanical Stabilisation, Bituminous Stabilisation and Chemical Stabilisation. Figure 2.5 illustrates these methods of asphalt pavement recycling as stated by Karlsson and Isacsson (2006). However, since this work only focuses on cold recycling ex situ, i.e. in-plant, subsequently, discussions shall only reflect this interest.

Information on all these classifications is well detailed in literature (ARRA, 2001; PIARC, 2003; and Karlsson and Isacsson, 2006).

2.13 Cold Recycling *Ex Situ* (In-Plant)

ARRA (2001) in describing this method of recycling stated that it is a process in which the asphalt recycling takes place in a central location using a stationary cold mix plant. The stationary plant could be a specifically designed plant or a cold in-place recycling train minus the milling machine, set up in a stationary configuration. As the name dictates, this method of pavement rehabilitation is usually carried out cold and without heating the reused material during the recycling process to produce a rehabilitated pavement (ARRA, 2001). O'Flaherty, (1988) opined that this method is normally used in order to conserve materials and energy, and to improve pavement surface condition.

The author explained further that, in addition, however it can be used to improve the load carrying capacity of the pavement. The reclaimed asphalt pavement used normally is obtained from cold planning or by ripping, removing and crushing operations. The milled or pulverized material is stockpiled, then processed to obtain an appropriate grading, and finally mixed in a plant to produce new treated bituminous mixture which could be used immediately or stockpiled to be used later. When it is to be used immediately, the recycled material is transported to the site, where it is mechanically laid and compacted. PIARC (2003) stated that mixers can be either continuous or batch wise.

In describing in detail a typical cold recycling plant, ARRA (2001), explained that the plant usually consists of a number of cold feed bins for the RAP and new

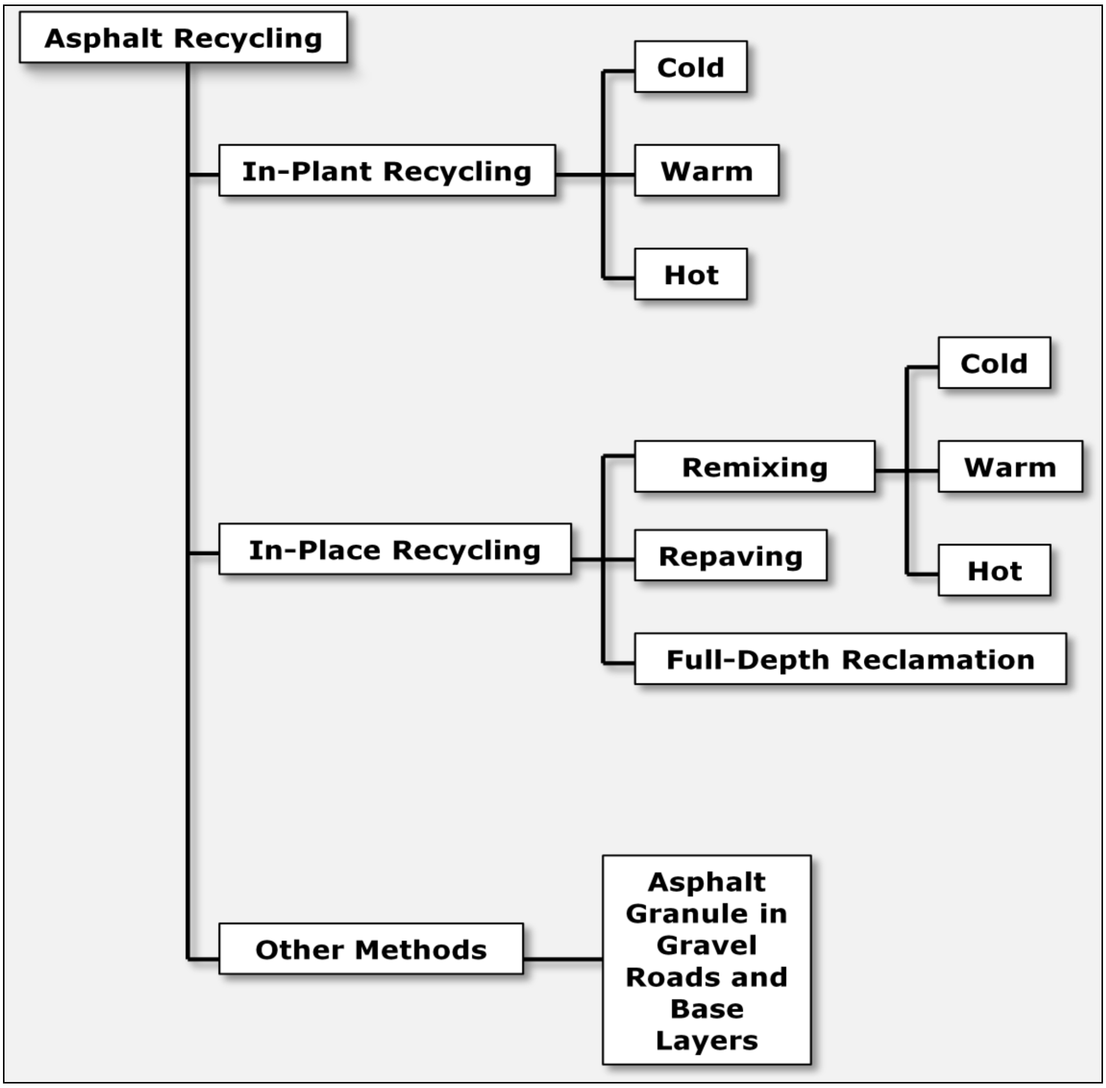


Figure 2.5: Methods of Asphalt Recycling (Karlsson and Isacsson, 2006)

aggregate, a belt scale, a computer controlled liquid recycling additive system, a pugmill, a hopper for temporary storage and loading of haul trucks or a conveyor/belt stacker if the cold mix is being stockpiled. Figures 2.6 and 2.7 illustrate the cold recycling plant set ups while Figure 2.8 illustrates the normal procedure to follow for cold recycling *ex situ*. The recycled materials are taken to site, placed and compacted using the conventional construction machineries normally used in the making of HMA pavements. ARRA (2001) further stated that the cold laid mix is usually overlain with a layer of HMA, although for some very low traffic roadways, a single or double seal coat is sometimes used.

Apart from the main advantage that direct control over the quality of materials being recycled is achievable through cold recycling *ex situ,* ARRA (2001) lists the general advantages of cold recycling as:

- Conservation of non-renewable resources
- Energy conservation compared to other reconstruction methods
- Eliminates the disposal problems inherent in conventional methods
- Surface irregularities and cracks are interrupted and filled
- Rutting, potholes and ravelling are eliminated
- Base and subgrade materials are not disturbed
- Pavement cross-slope and profile can be improved
- Problems with existing aggregate gradation and/or asphalt binder can be corrected with proper selection of new granular materials and stabilizing additives
- Significant structural treatment and improved ride quality
- Economic savings are realised

ARRA (2001) further stated that bitumen emulsion among other recycling agents could be used in cold recycling, but that however, the densification of cold mixes requires higher compactive effort than conventional HMA.

2.14 Pavement Recyclability - Factors to be Considered

Karlsson and Isacsson (2006) stated that any road in need of reconstruction or new overlay is a candidate for recycling. This assertion is just too general, though they later opined that the re-use of asphalt means that the complexity of the maintenance operation increases, as the reclaimed material is more difficult to characterise compared to new material. However, on a more technical point, PIARC (2003) opined that, the criteria for ascertaining the feasibility of recycling a

Figure 2.6: Cold Recycled Mix Plant (Wirtgen, 2005)

Figure 2.7: Cold Recycled Mix Placement (Wirtgen, 2005)

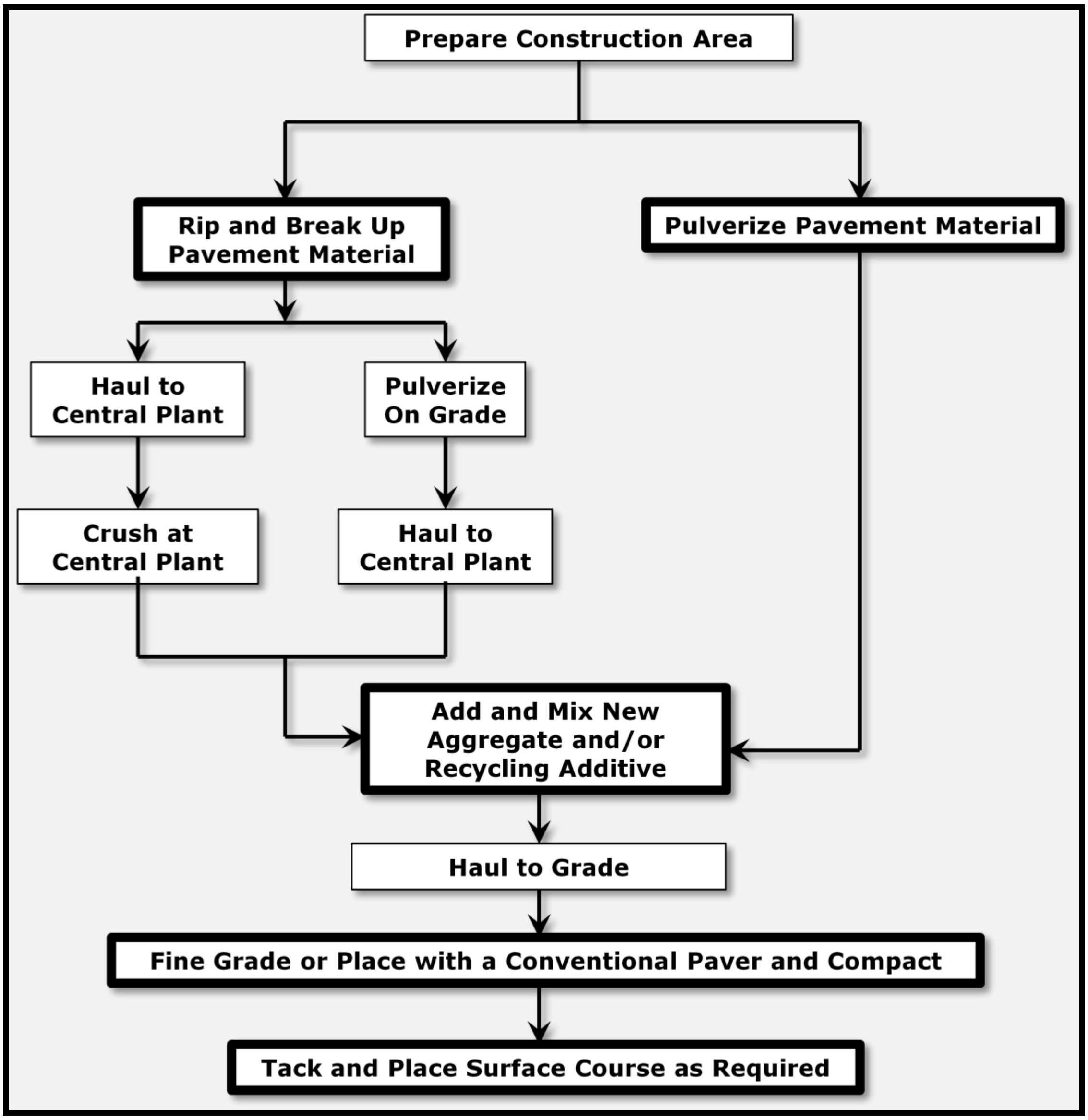


Figure 2.8: Process involved in Cold Recycling *Ex Situ* (ARRA, 2001)

pavement is to identify whether its deterioration comes mainly from the poor quality of the pavement itself (insufficient thickness, granular layers contaminated with clay, de-bonded bituminous layers, etc) or from problems related to the subgrade. They further submitted that the recycling feasibility should be established from knowledge of the structure of the pavement and of the characteristics of the materials present in it.

Thus for this reason, it is necessary to examine the road, to determine the characteristics of the materials of the pavement and to collect data on the traffic and climate. ARRA (2001) while elaborating on this gave general guideline for the preliminary selection of roads that are candidates for recycling or reclamation methods for the rehabilitation of asphalt pavements. Table 2.8 details the guidelines. However, ARRA (2001) cautioned that all of the candidate rehabilitation techniques have disadvantages and advantages since not all rehabilitation techniques are equally suited to treat the various pavement distress types. Also, the ability of a rehabilitation technique to correct pavement distress is dependent on the type of pavement distress, as well as the extent of the distress and its severity. A suggested guide by ARRA is shown in Figure 2.9. The local aggregate quality, amount/type of traffic, and climatic conditions are also important factors which need to be considered.

ARRA (2001) further stated that, though economic analysis which includes life cycle costs must be done before a decision is made on the method to be adopted, the following factors must be generally considered:

- The type and severity of the existing distresses
- Age/condition of the existing pavement materials and their potential for recycling
- Expected design life and performance requirements of the rehabilitation
- Traffic growth
- Structural capacity of existing roadway
- Environmental conditions
- Acceptable future maintenance activities
- Geometric, drainage, underground and surface utilities
- Traffic accommodation and safety
- Construction limitations
- Project location and size
- Contractor availability and experience
- Impacts on adjacent businesses and public
- Available budget
- Good engineering judgement

Table 2.8: Guidelines for the Preliminary Selection of Candidate Recycling or Reclamation Methods for Rehabilitation of Asphalt Pavements (ARRA, 2001)

Pavement Distress Mode	Candidate Rehabilitation Techniques							
	Cold Planing	Hot In-place Recycling	Cold In-place Recycling	Thin HMA	Thick HMA	Full Depth Recycling	Combination Treatments	Reconstruction
Raveling	3	1	2	1	3		1	
Potholes		2	1	3	2	3	1	
Bleeding	2	2	2		3	2	1	
Skid Resistance	2	2	2	1	3	3	1	
Shoulder Drop Off	2							1
Rutting	2	2	1		3	3	1	3
Corrugations	2	2	1		3	3	1	3
Shoving	2	2	1		3	3	1	3
Fatigue Cracking			1		1	1	1	1
Edge Cracking			3		3	1	1	1
Slippage Cracking		2	1		3	2	1	2
Block Cracking			1		3	1	1	2
Longitudinal Cracking		2	1		3	2	1	2
Transverse Cracking		2	1		3	1	1	2
Reflection Cracking		2	1		3	1	1	2
Discontinuity Cracking			1	3	3	1	1	2
Swells	3	2	2	3	1	2	1	3
Bumps	3	2	2	3	1	2	1	3
Sags	3	2	2	3	1	2	1	3
Depressions	3	2	2	3	1	2	1	3
Ride Quality	3	1	1	3	1		1	
Strength		3	2		1	1	1	1

Most Appropriate → **Least Appropriate**

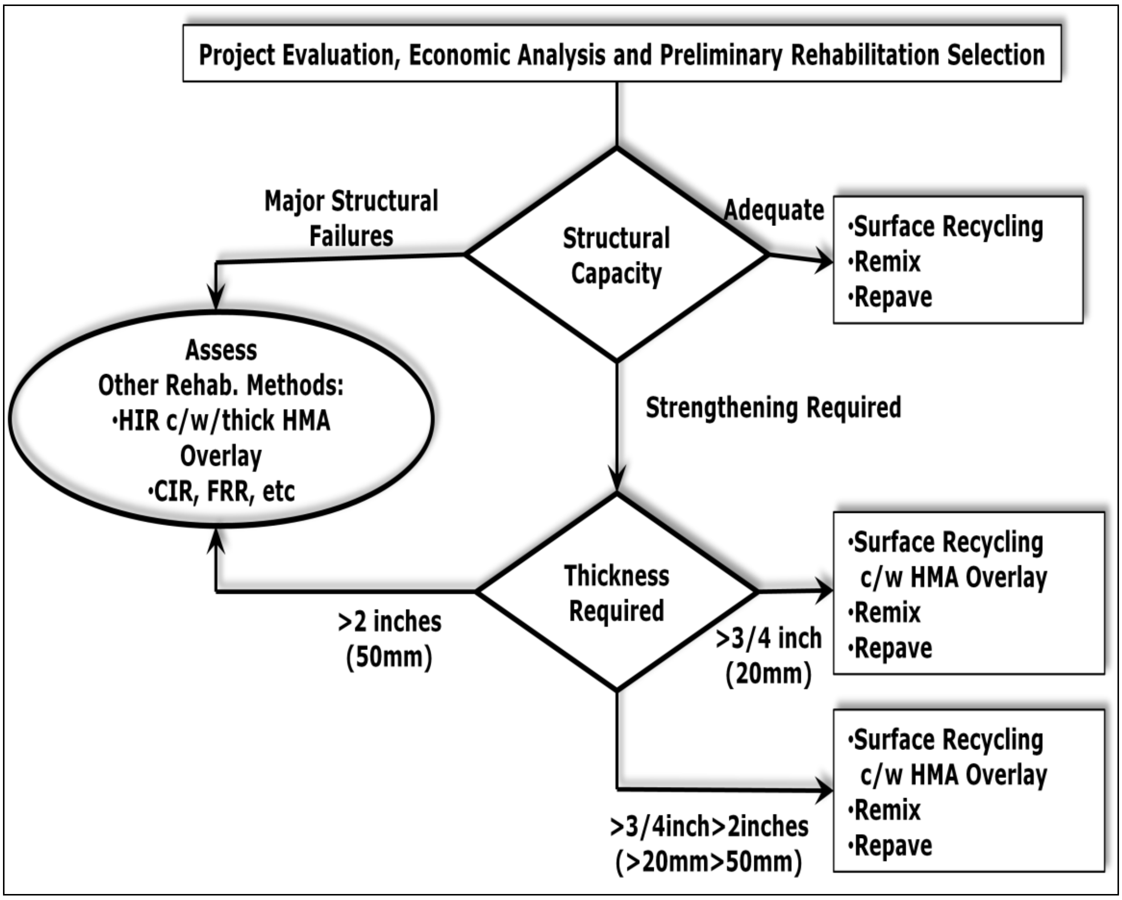


Figure 2.9: A Guide for Choosing Recycling Type to Employ in Addressing Pavement Problems (ARRA, 2001)

2.15 Recycling Agents

A number of materials with the purpose of altering the properties of old or aged binders during the recycling of asphalt pavements have been advanced. Karlsson and Iscasson (2006) stated that some of the materials go by names such as, reclaiming, recycling, modifying or softening agents (additives), recycling modifiers, rejuvenators but also fluxing, extender or aromatic oils. They further classified rejuvenators into two general groups:

- Softening agents- examples of such are, asphalt flux oil, lube stock, lubricating or crankcase oil, and slurry oils.
- Rejuvenating agents- examples of such are lube extracts and oil extenders.

They also reported that bitumen, bitumen emulsions, cut backs, foamed bitumen and a number of other proprietary additives have also been used as rejuvenators.

Rejuvenators generally are expected to restore the reclaimed binder characteristics to a consistency level appropriate for construction purposes and pavement performance, and, at the same time, optimise the chemical characteristics with regard to durability. A rejuvenator should also provide sufficient additional binder to coat any new aggregate that is added to the reclaimed mixture and satisfy mixture design requirements. Chen et al (2007) in buttressing this affirmed that the softening agent lowers viscosity of the aged bitumen while the rejuvenator restores the physical and chemical properties of the aged bitumen. FHWA (1997) stated that the types of recycling agents used include emulsified recycling agents, softer grade asphalt cements and cutback asphalts and that however, the most commonly used recycling agents for completely cold recycling processes are emulsified asphalt cements or emulsified recycling agents. This is because the emulsions are liquid at ambient temperatures and have the capacity for being dispersed throughout the mix and do not cause major air pollution problems. At ambient temperature, the softening effect of the recycling agent is a time and temperature dependent physico-chemical process. Schiavi et al (2003) reported that rejuvenators are solutions or emulsions of bituminous binders that are used to restore the properties of softness and low viscosity to hardened (aged) bitumen and that they do this by restoring the balance between maltenes, which are the volatile part of the bitumen lost with time, and the asphaltenes, which are produced and precipitated as a consequence of the ageing. Epps et al (1980) informed that rejuvenators should easily be dispersed in the old binder, and as well be uniform from batch to batch, and practical to use, for example preventing flashing, smoking and health risks.

On the same note, PIARC (2003) identified cement, lime and cement, and cement and emulsion or foamed bitumen in addition to those materials earlier mentioned as materials which have been found very useful in quite a number of recycling projects.

In this research work bitumen emulsion will be used as the recycling agent. This has been chosen since cutback which is similar in operation to emulsion, is commonly used in Nigeria. Therefore switching to emulsions should not pose much difficulty compared to foamed bitumen and furthermore existing equipments for cutbacks can be easily adapted to suit emulsions. Hence further discussions will focus on bitumen emulsion.

2.16 Bitumen Emulsion

The Asphalt Institute and Asphalt Emulsion Manufacturers Association (1997) and Read and Whiteoak (2003) reported that there are three ways to make highly viscous bitumen into a low-viscosity liquid:

- heat it,
- dissolve it in a petroleum solvent
- emulsify it

Thanaya (2003) while discussing same topic opined that, in addition to these three methods, foaming the highly viscous bitumen is another method for lowering the viscosity of such bitumen. While other methods of lowering the viscosity of bitumen are well detailed in literature, the focus of this research work is on bitumen emulsion. The Asphalt Institute and Asphalt Emulsion Manufacturers Association (1997) reported that bitumen emulsions are an effective method of preventive and corrective maintenance of existing pavements by virtue of energy conservation (normally applied at ambient temperature), very low or absence of atmospheric pollution and reduced hazards, that they offer (James, 2006). Also, the fact that bitumen emulsions are suitable replacements for cutbacks that the Nigerian road sector is used to deem them appropriate for this research work.

James (2006) and Akzo Nobel (2008) defined an emulsion as the dispersion of small droplets of one liquid in another. Needham (1996) stated that in contrast to solutions, the two liquids are coexistent rather than mutually mixed. Akzo Nobel

(2008) puts it better by stating that emulsions are formed by any two immiscible liquids, but in most emulsions one of the phases is water. Thus in the case of bitumen emulsion, these are bitumen, which is a liquid with a very high viscosity, and water, though emulsifying agents are normally added to facilitate stability.

The Asphalt Institute and Asphalt Emulsion Manufacturers Association (1997) stated that when bitumen is milled into microscopic particles and dispersed in water with a chemical emulsifier, it becomes a bitumen emulsion as seen in Figure 2.10. Akzo Nobel (2008) and James (2006) classified emulsion into three broad groups: oil-in-water (O/W), water-in-oil and multiple phase emulsions. Oil-in-water emulsions are those in which the continuous phase is water and the dispersed (droplet) phase is an oily water-insoluble liquid. On the other hand, water-in-oil (W/O) emulsions are those in which the continuous phase is oil and the dispersed phase is water. Water-in-oil emulsions are sometimes called inverted emulsions. Akzo Nobel (2008) further stated that multiple phase emulsions can be formed in which the dispersed droplets themselves contain smaller droplets of a third phase, usually the same liquid as the continuous phase. Figure 2.11 depicts the three broad groups of emulsions. Bitumen emulsions are normally of the oil-in-water type.

James (2006) and the Asphalt Institute and Asphalt Emulsion Manufacturers Association (1997) reported that bitumen emulsions were first developed in the early 1900s, and that it was in the 1920s when emulsions came into general use in pavement applications. Specifically, Read and Whiteoak (2003) reported that in 1906, the first patent covering the application of bituminous dispersions in water for road building was taken out. Their early use was in spray applications and as dust palliatives. However James (2006) and the Asphalt Institute and Asphalt Emulsion Manufacturers Association (1997) further confirmed that the growth in the use of bitumen emulsion was relatively slow, limited by the type of emulsions available and a lack of knowledge as to how they should be used. None-the-less, the continuing development of new types and grades, coupled with improved construction equipment and practices, now give a broad range of choices. For example, Read and Whiteoak (2003) reported that since the early 1950s, cationic emulsions have become increasingly popular because of their affinity to many solid surfaces. This is an important property in road construction because good adhesion of bitumen to different types of mineral aggregate is essential. Hence, virtually any roadway requirement can be met with emulsions; furthermore

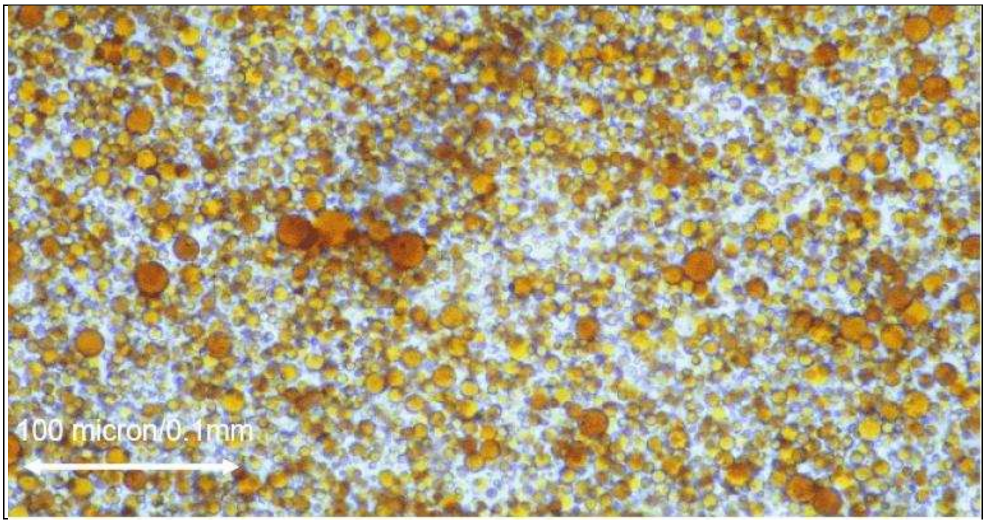
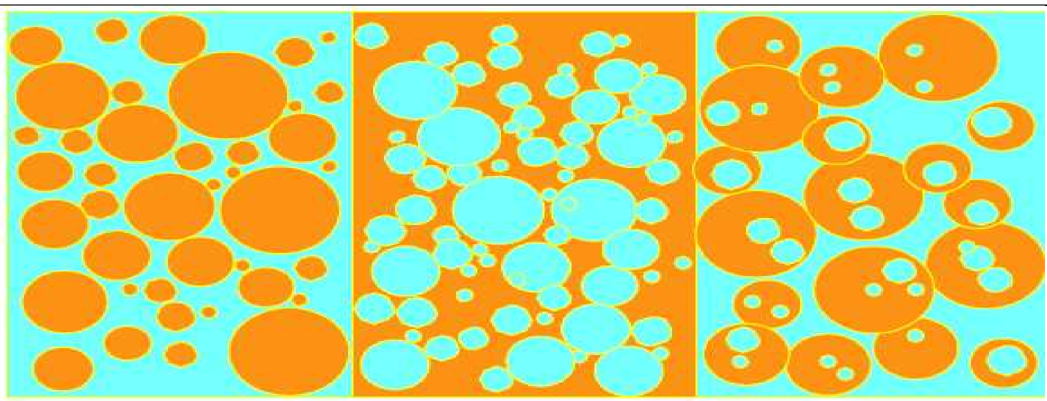


Figure 2.10: Photomicrograph of a Bitumen Emulsion (James, 2006)



oil-in-water emulsion	water-in-oil emulsion	multiple emulsion

Figure 2.11: The Three Broad Groups of Emulsion (James, 2006; and Akzo Nobel, 2008)

judicious selection and use can yield significant economic and environmental benefits.

Bitumen emulsions have been found useful in many road applications. Table 2.9 details some of these applications.

2.17 Manufacture of Bitumen Emulsions

James (2006) and the Asphalt Institute and Asphalt Emulsion Manufacturers Association (1997) reported that the three main constituents used for the manufacture of bitumen emulsions are: bitumen, water, and emulsifier. Bitumen is the main basic ingredient of bitumen emulsion and, in most cases it makes up from 50 to 75% of the emulsion. Although hardness of base bitumen may vary, most emulsions are made with bitumens in the 60-250 penetration range. On occasion, climatic conditions may require harder or softer base bitumen. In any case, chemical compatibility of the emulsifying agent with the bitumen is essential for production of a stable emulsion (Asphalt Institute and Asphalt Emulsion Manufacturers Association, 1997).

Water is the second ingredient used in the manufacture of bitumen emulsions and it makes up from 25% to 60% of a bitumen emulsion (James, 2006). The Asphalt Institute and Asphalt Emulsion Manufacturers Association (1997) stated that the contribution of water to the desired properties of the finished product cannot be minimised. Impurities in water can lead to the production of an unstable bitumen emulsion and thus, the water to be used must be carefully selected.

The third ingredient is the emulsifying agent and which makes up from 0.1% to 2.5% of the bitumen emulsion. The property of the bitumen emulsion produced normally depends on the chemical used as the emulsifier. James (2006) listed some minor components such as, calcium and sodium chloride (normally included as 0.1% to 0.2% of the bitumen emulsion to reduce the osmosis of water into the bitumen and minimize the changes in viscosity), adhesion promoters (to enhance adhesion properties of bitumen emulsion), solvent (to improve emulsification, reduce settlement, improve curing rate at low temperatures or to provide the right binder viscosity) and latex (to improve properties such as cohesion, resistance to cracking at low temperatures and resistance to flow at high temperatures).

Bitumen emulsions are usually manufactured using a colloid mill, although other dispersion devices are possible (Akzo Nobel, 2008). In the colloid mill energy is

Table 2.9: Typical Uses of Bitumen Emulsion (Asphalt Institute and Asphalt Emulsion Manufacturers Association, 1997)

Type of Construction	ASTM D977 AASHTO M208									ASTM D2397 AASHTO M140					
	RS-1	RS-2	HFRS-2	MS-1, HFMS-1	MS-2, HFMS-2	MS-2h, HFMS-2h	HFMS-2s	SS-1	SS-1h	CRS-1	CRS-2	CMS-2	CMS-2h	CSS-1	CSS-1h
Asphalt-Aggregate Mixtures:															
Plant Mix (Hot or Warm)						X[A]									
Plant Mix (Cold)															
Open-Graded Aggregate					X	X						X	X		
Dense-Graded Aggregate							X	X	X					X	X
Sand							X	X	X					X	X
Mixed-in-Place															
Open-Graded Aggregate					X	X						X	X		
Dense-Graded Aggregate							X	X	X					X	X
Sand							X	X	X					X	X
Sandy Soil							X	X	X					X	X
Asphalt-Aggregate Applications															
Single and Multiple Surface Treatments	X	X	X							X	X				
Sand Seal	X	X	X	X						X	X				
Slurry Seal							X	X	X					X	X
Micro Surfacing															X[E]
Sandwich Seal		X	X								X				
Cape Seal		X									X				
Asphalt Application:															
Fog Seal				X[B]				X[C]	X[C]					X[C]	X[C]
Prime Coat					X[D]			X[D]	X[D]					X[D]	X[D]
Tack Coat				X[B]				X[C]	X[C]					X[C]	X[C]
Dust Palliative								X[C]	X[C]					X[C]	X[C]
Mulch Treatment								X[C]	X[C]					X[C]	X[C]
Crack Filler								X	X					X	X
Maintenance mix:															
Immediate Use							X					X	X		
Stockpile							X								

A Grades other than HFMS-2h may be used where experience has shown that they give satisfactory performance
B Diluted with water by the manufacturer
C Diluted with Water
D Mixed-in prime only
E Polymer must be added during or prior to emulsification

applied to the system by passing the mixture of hot bitumen and water phase between a rotating disc, cone or flywheel and a stator. The rotor as well as stator may be grooved or have teeth in order to create a turbulent flow. Bitumen emulsion can be produced either in a batch or an in-line process plant. The batch process involves at least two process steps- water phase (soap) preparation and the actual emulsion production. The water phase is prepared in a tank into which heated water, emulsifier and other emulsion chemicals are metered and the solution properly mixed. In the emulsion production process, bitumen and the readymade water phase are dosed to the colloid mill. If solvent is to be added to the bitumen, then a batch tank is needed for bitumen as well, or the solvent must be dosed in-line. Figure 2.12 illustrates a typical batch emulsion plant while Figure 2.13 is a picture of an emulsion plant.

Since an entire scientific field has been devoted to the study of emulsification, it is of no use discussing beyond this point since this work is not focusing on the production of bitumen emulsion. Meanwhile, Akzo Nobel (2008), Baumgardner (2006), Thanaya (2003), Ibrahim (1998), Asphalt Institute and Asphalt Emulsion Manufacturers Association (1997) and Needham (1996) have extensively discussed the manufacturing of bitumen emulsion among other related issues.

2.20 Classification of Bitumen Emulsions

The emulsifier is a surface-active agent or a surfactant (James, 2006; Asphalt Institute and Asphalt Emulsion Manufacturers Association, 1997; and Akzo Nobel, 2008). The emulsifier keeps the bitumen droplets in stable suspension and controls the breaking time. It is also the determining factor in the classification of the emulsion. The Asphalt Institute and Asphalt Emulsion Manufacturers Association (1997) stated that surfactants are water soluble substances whose presence in solution markedly changes the properties of the solvent and the surfaces they contact. They are categorized by the way they dissociate or ionize in water. Structurally they possess a molecular balance of a long lipophilic (oil loving), hydrocarbon tail and a polar, hydrophilic (water loving) head (see Figure 2.14). Surfactants are absorbed at the interface between liquids and gases or liquid and solid phases. The surfactant molecule or ion acts as a bridge between two phases (James, 2006).

While the Asphalt Institute and Asphalt Emulsion Manufacturers Association (1997) classified bitumen emulsions as anionic, cationic, or non-ionic only, Read and Whiteoak (2003) added a fourth group referred to as clay-stabilised

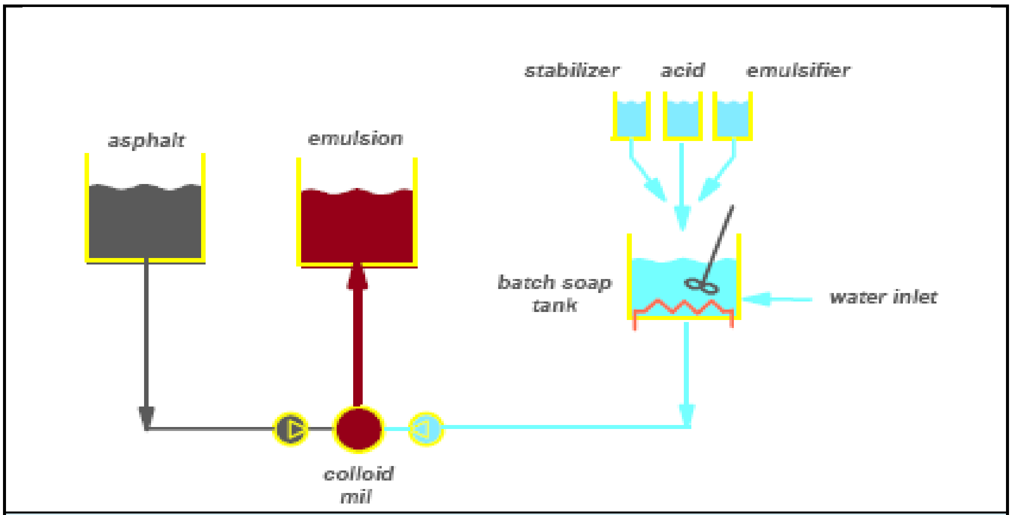


Figure 2.12: Batch Emulsion Plant (James, 2006)

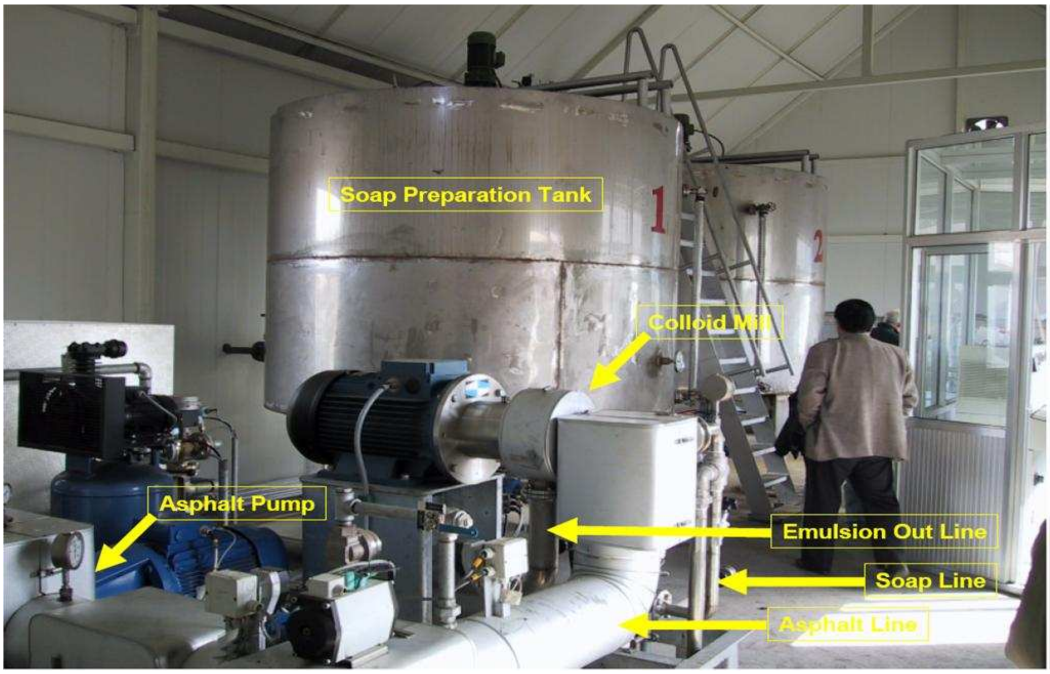


Figure 2.13: Picture of a Bitumen Emulsion Plant (James, 2006)

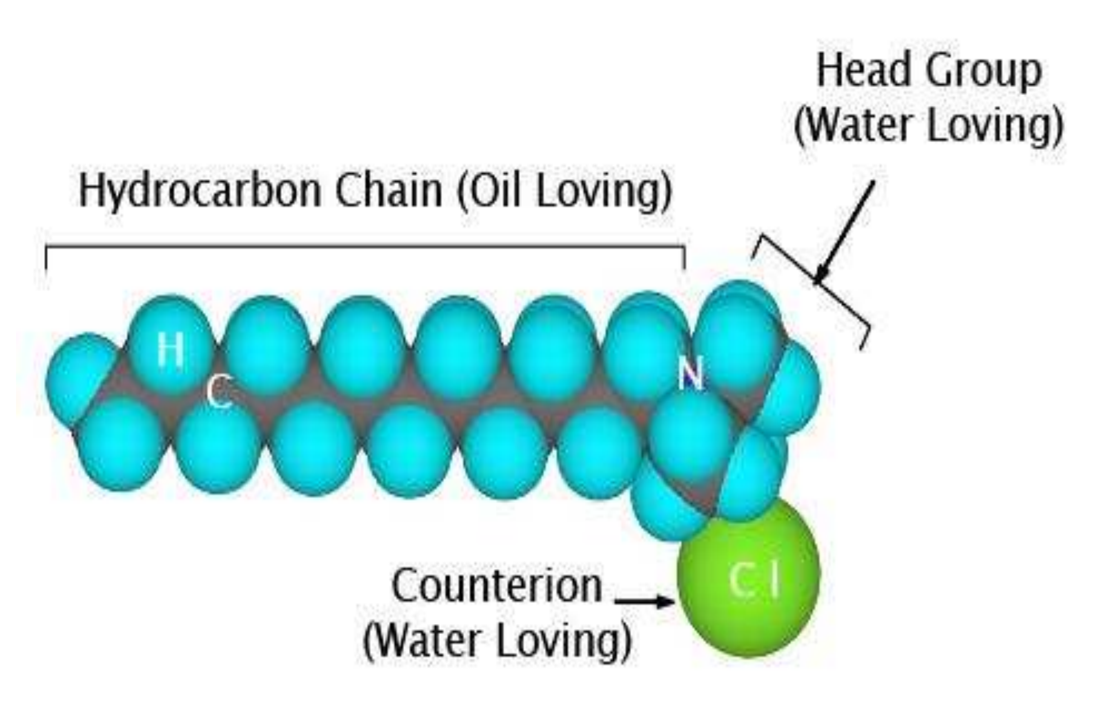


Figure 2.14: Structure of Surfactant (James, 2006)

emulsions. The most commonly used of these in road construction are the anionic and the cationic emulsions (James, 2006; Needham, 1996; Thanaya, 2003; and Read and Whiteoak, 2003).

Meanwhile, the Asphalt Institute and Asphalt Emulsion Manufacturers Association (1997) reported that the most common anionic emulsifiers are fatty acids, which are wood-product derivatives such as tall oils, rosins, and lignins. Anionic emulsifiers are saponified (turned into soap) by reacting with sodium hydroxide or potassium hydroxide. Most cationic emulsifiers are fatty amines (e.g. diamines, imidazolines, and amidoamines). The amines are converted into soap by reacting with acid, usually hydrochloric. Another type of emulsifying agent, fatty quaternary ammonium salts, is used to produce cationic emulsions. The Asphalt Institute and Asphalt Emulsion Manufacturers Association (1997) further stated that, basically there are three types of surfactants that are classified according to their dissociation characteristics in water and these are:

- Anionic Surfactants: $CH_3(CH_2)_n COO^-Na^+$ (where the electrovalent and polar hydrocarbon group is part of the negatively charged ion, when the compound ionizes)
- Non-ionic Surfactants: $CH_3(CH_2)_n$ COO $(CH_2\ CH_2O)_x$ H (Where the hydrophilic group is covalent and polar, and which dissolves without ionization)
- Cationic Surfactants: $CH_3(CH_2)_n\ NH_3^+Cl^-$ (where the electrovalent and polar hydrocarbon group is part of the positively charged ion when the compound ionizes)

The emulsifier is the single most important component in any asphalt emulsion formulation. To be an affective emulsifier for bitumen, a surfactant must be water soluble and possess a proper balance between the hydrophilic and lipophilic properties.

The Asphalt Institute and Asphalt Emulsion Manufacturers Association (1997) further classified emulsions on the basis of how quickly the asphalt droplets will coalesce i.e. revert to asphalt cement: Rapid Setting (RS), Medium Setting (MS), Slow Setting (SS) and Quick Setting (QS). The tendency to coalesce is closely related to the speed with which an emulsion will become unstable and break after contacting the surface of an aggregate. An RS emulsion has little or no ability to mix with an aggregate, an MS emulsion is expected to mix with coarse but not fine aggregate, SS and QS emulsions are designed to mix with the fine aggregate, with the QS expected to break more quickly than the SS. Emulsions are further classified by a series of numbers. The letter "C" in front of the emulsion type denotes cationic, absence of "C" implies anionic. The numbers in the classification indicate the relative viscosity of the emulsion. The "h" that follows certain grades simply means that harder base bitumen is used, while "s" means softer base bitumen e.g. MS-1 and CMS-2s, MS-2h. In this case the "2" means more viscous than "1".The "HF" preceding some of the anionic grades indicates high float, as measured by the float test. High-float emulsions have a gel quality imparted by the addition of certain chemicals that permit a thicker asphalt film on the aggregate particles and prevent drain off of asphalt from the aggregate. These grades are used primarily for cold and warm plant mixes, seal coats and road mixes. Table 2.9 details these classifications.

2.19 Factors that Affect the Choice and Performance of Bitumen Emulsions

The Asphalt Institute and Asphalt Emulsion Manufacturers Association (1997) identified some of the factors that affect the selection of bitumen emulsion as:

- Climatic conditions anticipated during construction
- Aggregate type, gradation and availability
- Construction equipment availability
- Geographical location
- Traffic control
- Environmental considerations

In the same vein, The Asphalt Institute and Asphalt Emulsion Manufacturers Association (1997) stated that the following factors among others affect the performance of a bitumen emulsion:

- Chemical properties of the base bitumen
- Hardness and quantity of the base bitumen
- Bitumen particle size in the emulsion
- Type and concentration of the emulsifying agent
- Manufacturing conditions such as temperature, pressure and shear
- Ionic charge on the emulsion particles
- Order of addition of the ingredients
- Type of equipment used in manufacturing the emulsion
- Properties of the emulsifying agent
- Addition of chemical modifiers or polymers
- Water quality (hardness).

They however opined there is no good substitute for a laboratory evaluation of the emulsion and the aggregate to be used.

2.20 Tests on Emulsion

There a number of tests normally carried out to check whether emulsions meet set specifications. Such tests are carried out on the emulsion or the residue. Some of these common tests and the respective specifications/requirements are detailed in Table 2.10. These tests and others are properly discussed by the

Asphalt Institute and Asphalt Emulsion Manufacturers Association (1997). Most of the requirements stated in the table apply to temperate climates.

2.21 Curing and Breaking of Bitumen Emulsion

For bitumen emulsion to perform its ultimate function as a binder, the Asphalt Institute and Asphalt Emulsion Manufacturers Association (1997) stated that the water in the emulsion must separate from the bitumen phase and evaporate. This separation is referred to as breaking or setting and is normally characterised by a change in colour from brown to black. They further state that the specific type and concentration of emulsifying agent primarily controls the rate of breaking. Meanwhile (Thom, 2009) stated that emulsion needs the aggregate to be wet too in order to produce reasonable binder coating.

Curing involves the development of the mechanical properties of the bitumen (Asphalt Institute and Asphalt Emulsion Manufacturers Association, 1997). For this to happen, the water must substantially evaporate, and the bitumen emulsion particles have to coalesce and bond to the aggregate. Thom (2009) however opined that some of the water is trapped. Figure 2.15 illustrates the processes involved in the breaking of a typical emulsion.

The Asphalt Institute and Asphalt Emulsion Manufacturers Association (1997) listed some factors which affect breaking and curing thus:

- water absorption
- aggregate moisture
- weather conditions
- mechanical forces
- surface area
- surface chemistry
- emulsion and aggregate temperature
- type and amount of emulsifier

2.22 Recycling in the Tropics?

There are a lot of success stories which have confirmed that asphalt pavement recycling generally is a tool and technique for sustainable road development and that it has come to stay in the developed countries. However, asphalt pavement recycling is not yet being practiced fully as a method of road rehabilitation/reclamation in most tropical nations located in Africa. PIARC (2008) reported that only South Africa, which is located in the South of Sub Saharan

Table 2.10: Testing and Specification for Bitumen Emulsions and Emulsion Residues (James, 2006)

Tests	CRS-2	CMS-2	CSS-1	HFRS-2	HFMS-2	SS-1
On Emulsions						
Viscosity @ 25ºC			20-100		100+	20-100
Viscosity @ 50ºC	100-400	50-450		75-400		
Storage Stability 24hours	<1	<1	<1	<1	<1	<1
Demulsibility	60+			60+		
Coating Ability		Good/Fair			Good/Fair	
Particle Charge Test	positive	positive	positive			
Sieve Test	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
Cement Mixing Test			<2			<2
Distillation Residue	65+	65+	57+	63+	65+	57+
Oil Distillate	<3	<12				
On Residue						
Penetration @ 25ºC	100-250	100-250	100-250	100-200	100-200	100-200
Ductility at 25ºC	40+	40+	40+	40+	40+	40+
Solubility	97.5	97.5	97.5	97.5	97.5	97.5
Float Test				1200+	1200+	

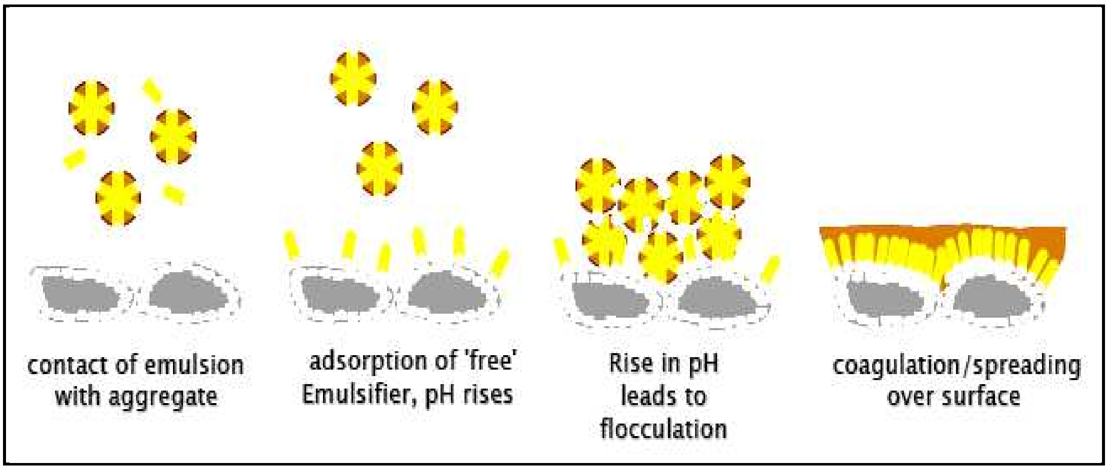


Figure 2.15: Possible Stages in the Breaking of a Cationic Emulsion (James, 2006; and Akzo Nobel, 2008)

Africa, has been practising recycling for many years. So far, the only other evidence of such practice is in Nigeria. Eyo (2005), reported that the Federal Road Maintenance Agency, Nigeria, (FERMA) recycled failed asphalt and applied the reclaimed pavement in the patching of potholes in the Northern part of the country. PW Nigeria Limited also carried out some repairs on the runway at Muritala Mohammed International Airport in Lagos, by milling about 75,000m^2 of asphalt wearing course, pulverising it and laying it as the road base. This was then overlaid with asphalt (PW Nigeria Ltd, 2007). Although there is no evidence in literature about the details of the performances of pavements produced from these few recycling practices which really, are not on a large scale, they are pointers to the fact that pavement recycling is possible in West Africa, and elsewhere in the tropics, once the proper pavement material characterization and mix design has been done. Eyo (2004) and Uket (2007) reported that the Federal Government in Nigeria is concluding plans to introduce the recycling of asphalt for the strengthening of road pavements in Nigeria. Thus the prospects for recycling in Nigeria and other tropical countries are very high. The essence of this present research is to develop a guideline for the production of cold recycled mixes using bitumen emulsion for tropical conditions with hot climate.

While reasons for adopting cold recycling are stated in Chapter 1 of this report, one of the reasons for choosing bitumen emulsion as the recycling agent for this research work, is the fact that most practising engineers and road builders in such countries (developing countries) are used to cutbacks (the use of which is now outlawed in most developed countries because of safety and environmental concerns) which operate in a similar fashion to bitumen emulsion, furthermore, the facilities for applying cutbacks could be used for bitumen emulsion. It is hoped that such a material would not pose any problems with regard to handling when introduced in road works in developing countries.

2.23 Summary

The following conclusions have been drawn based on the literature search conducted in this chapter and some information given earlier in Chapter 1 of this thesis:

- The road Network in Nigeria and Sub-Saharan Africa generally is in a deplorable state due to long neglect as a result of lack of funds for maintenance and rehabilitation. The situation can only get worse under the present global economic recession.

- The few road restorations being embarked upon in such places are at the moment not sustainable and very expensive.
- There is urgent need to introduce alternative methods and materials in order to significantly cut down on costs (stretch funds) and also meet the 2015 target of achieving 82% ‘very good’ condition for the entire road network in Nigeria for example.
- With only an improvement of 10% achieved over a period of 4 years (2003 – 2007) using the conventional means of road restoration, this implies that the target is not feasible unless extra measures such as the use of alternative sustainable methods and materials are introduced where applicable.
- Such methods and materials have been applied successfully in America and Europe where the climate is temperate (cold).
- One out of the methods that appeal and which can be applied without spending huge amounts on machinery is cold recycling RAPs with bitumen emulsion with a potential cut in costs in the region of 40-60%.
- Cold recycling *ex situ* using bitumen emulsion should not pose any problems since cutbacks which are similar in operation to emulsions have been used in the industry for a long time and thus are easily adaptable. Such would also ensure good quality control.
- The development of stiffness that meets minimum requirements for cold mixes early in life should not be a problem since the hot climate that exists in Nigeria and other Sub-Saharan Countries would aid quick evaporation of moisture in such mixes. Moisture hitherto has been identified by researchers as a major inhibiting factor responsible for slow stiffness development of cold mixes in temperate climates.
- There is a need to develop guidelines that will assist in making provisions for the use of such alternative materials in the highway design manual of Nigeria which is lacking at the moment.
- Such guidelines can only be developed by formulating and assessing the mechanical performances of mixes made with RAPs typical of road pavements in such hot climates. Such assessment should include fatigue and permanent deformation, having been identified as the major mechanisms by which flexible pavements fail. It is believed that when CBEMs are fully cured, their responses should be close to those of HMA.

3 A Review on Laboratory Investigations on Cold Mixes

3.1 Overview

Cold mixes with focus on the use of reclaimed asphalt pavements (RAPs) with bitumen emulsions have been considered as appropriate for the scenario being considered i.e. construction/rehabilitation of bituminous roads in developing countries with hot tropical climates for the reasons mentioned earlier in this thesis. The major benefits of such materials and methods are that they assist in stretching funds available for roads, since they have been acclaimed among others to be economical (old materials are reused), energy and eco-efficient, when used for road construction or upgrading. Presently, developing countries of the world are finding it difficult to adequately fund road construction, rehabilitation and upgrading (UNECA, 2007). This study will thus revolve round RAPs and bitumen emulsion. While bitumen emulsion could easily be sourced locally, sourcing the heavily aged RAPs needed for this research work might pose a problem.

Since RAPs have been extensively used in this work, it was thought proper to start this chapter by discussing the causes of hardening in bituminous materials and the consequences of such. This discussion is later extended to laboratory ageing of bituminous materials and cold mixes generally. Researchers have been able to successfully produce RAPs in the laboratory which are typical of those found in developed countries with temperate climates (Bell, 1989; and Scholz, 1995). Meanwhile bitumen emulsion was properly discussed in the preceding chapter.

3.2 Ageing in Bitumen and Bituminous Materials Defined

In a review carried out by Airey (2003) on ageing in bitumen and bituminous mixtures, he described the term ageing as hardening which is primarily associated with the loss of volatile components and oxidation of the bitumen during asphalt mixture construction (short-term ageing) and progressive oxidation of the in-place material in the field (long-term ageing). Scholz (1995) in his work, defined bitumen ageing as hardening in bitumen as a result of compositional changes in the bitumen. Srivastava and Rooijen (2000) considered that the phenomenon in which the rheological properties of bitumen change with time (i.e. bitumen becomes harder and more elastic) is referred to as ageing. Notwithstanding the associated causes, ageing in bitumen could be defined simply as hardening due to changes in molecular structure and/or chemistry of bitumen with a consequential effect on its rheology.

3.3 The Cause, Nature and Process of Ageing in Bitumen and Bituminous Materials

Activities tailored towards understanding ageing in bitumen have a relatively long history. Welborn (1979) recorded that since the 1930s, research has continued to develop an understanding of the factors contributing to short term and long term ageing. Scholz (1995) reported that many researchers have investigated age hardening of bitumens and bituminous mixtures and have provided significant advances toward a better understanding of the mechanisms of age hardening. In an extensive investigation, Traxler (1963) listed 15 factors (see Table 3.1) that could aid ageing in bitumen and which consequently lead to a change in its chemical, rheological and adhesion characteristics. Mastrofini and Scarsella (1999) on the same issue opined that, colloidal structure and stability of asphaltenes in residues and bitumens are directly connected with ageing. Srivastava and Rooijen (2000) averred that some aggregates act as catalysts for the oxidation reactions, while others have inhibitive effects and that during service life, oxidation depends, apart from climatic conditions and the ageing resistance of the bitumen, mainly on the amount of air voids in the asphalt and the bitumen film thickness.

Lu and Isacsson (2002) opined that bitumen ageing mechanisms in pavements are two in nature with the main being irreversible, and normally characterised by chemical changes of the binder which in turn have an impact on the rheological properties of the bitumen. They identified oxidation, loss of volatile components and exudation as the factors responsible for the mechanism. On the other hand, they stated that the second mechanism is a reversible process which is called physical hardening (steric hardening). Karlsson (2002) from his experience found out that the degree of ageing depends on temperature, air void content of the mixture and chemical composition of the binder. Airey (2003) advanced that molecular structuring over time (steric hardening) and actinic light (primarily ultraviolet radiation, particularly in desert conditions) may also contribute to ageing. Judycki and Jaskula (2003) in line with Airey’s findings declared in

Table 3.1: Factors which Affect Chemical Rheological and Adhesion Characteristics of Bitumen (Traxler, 1963)

Factors	Influenced by					Occurs	
	Time	Heat	Oxygen	Sun-light	Beta & Gamma Rays	At Surface	In Mass
Oxidation (in dark)	√	√	√			√	√
Photo oxidation (direct light)	√	√	√	√		√	
Volatilisation	√	√				√	√
Photo oxidation (reflected light)	√	√	√	√		√	
Photo chemical (direct light)	√	√		√		√	
Photo chemical (reflected light)	√	√		√		√	
Polymerisation	√	√		√		√	√
Development of internal structure (Ageing, Thixotropy)	√					√	√
Exudation of Oil (Syneresis)	√	√				√	
Changes by nuclear energy	√	√			√	√	√
Action by water	√	√	√	√		√	
Absorption by solid	√	√				√	√
Absorption of components at solid surface	√	√				√	
Chemical reactions or catalytic effects at interface	√	√				√	√
Microbiological deterioration	√	√	√			√	√

their work that short term ageing of bitumen occurs during bitumen storage, mix production and laying; while long term ageing occurs during the service life of the asphalt pavement. Mouillet et al (2003) simply stated that ageing in conventional bitumen is a complex process. In a more recent work, Chen et al (2007) corroborated the findings of earlier researchers by reporting that the degree of ageing in bitumen depends on temperature, air void content of the mixture and the chemical composition of the binder, and that during bitumen oxidative ageing, the saturates remain the same while solubilising aromatics decrease in quantity, because the aromatics react with oxygen to produce asphaltene, which causes the asphaltene content to increase. Also, Shen et al (2007) in agreement with Scholz's findings, using gel permeation chromatography (GPC) showed that compositional changes of the blends of aged binders are reflected well by the chromatographic profiles measured by the GPC. Meanwhile, Carswell et al (2008) in a general review commented that durability of asphalt pavements is a major issue and that it is a fairly complex problem because it involves a number of parameters, with binder ageing and moisture damage considered as prominent.

From the forgoing, it could be conveniently summarised that ageing in bitumen and bituminous mixtures is a complex phenomenon that ultimately manifests as hardening of the binder. Airey (2003) and Bell (1989) from their extensive literature reviews, both agreed that the majority of researchers considering ageing of bitumen and bituminous mixtures have come to the conclusion that, (a) the loss of oily components by volatility or absorption, (b) changes in composition by reaction with atmospheric oxygen and (c) molecular structuring that produces thixotropic effects are the major factors responsible for ageing. Thus, most researchers have limited their investigations to these three factors and indeed recent research works have been towing this line as will be discovered in the subsequent sections of this report.

3.4 Consequences of Ageing in Bitumen and Bituminous Mixtures

Short term ageing of bituminous mixtures i.e. during mixing, transportation, laying and compaction, is most desired by pavement engineers for the beneficial role of facilitating the necessary gain in stiffness required for pavements to support traffic after construction. However, when the ageing becomes excessive, it could lead to a failure of the pavement functionally and structurally. Scholz (1995) reported that excessive age hardening can result in a brittle bitumen- one with significantly reduced flow capabilities- which contributes to various forms of cracking in the bituminous mixture. In an earlier work, Kim and Burati (1993) discovered that ageing of bitumen either in the field or in the laboratory results in drastic changes in molecular size that leads to changes in the consistency of the bitumen. Airey (1997) from his findings stated the consequences of bitumen ageing as, increase in complex modulus with corresponding decrease in phase angle (i.e. showing an increase in elastic behaviour of bitumen). In considering pavements in the tropics, Smith and Jones (1998) observed that ageing causes premature top down cracking in pavements in the tropics. In buttressing this fact further, Mastrofini and Scarsella (1999) in their work agreed that bitumen ageing is a very complex process resulting in hardening of bitumens and embrittlement, both in application and in service, which contributes greatly to the deterioration of paving applications.

Meanwhile, Soenen et al (2000) in their studies on polymer modified bitumen confirmed that, due to ageing, the bitumen increases its stiffness and elasticity, and that the interaction between aged polymer and aged bitumen is different from the un-aged state. Srivastava and Rooijen (2000) on another note argued that hot temperature induces ageing which results in permanent deformation and surface cracking of asphalt mixtures. Lu and Isacsson (2002) opined that bitumen ageing is one of the principal factors causing the deterioration of asphalt pavements. They reported that ageing influences bitumen chemistry and rheology significantly and that chemical changes such as, formation of carbonyl compounds and sulfoxides, transformation of generic fractions (asphaltenes, resins, saturates and aromatics), and increases in amount of large molecules, molecular weight and polydispersity take place during ageing (see Figure 3.1 and Table 3.2). Karlsson (2002) stated that ageing in principle has negligible effect on the rate of diffusion in aged binder since the diffusion only takes place in the maltene phase which is relatively unaffected by ageing. On the other hand, Widyatmoko (2002) reported that the capacity for healing reduces more rapidly after ageing for semi-blown binders. Airey (2003) submitted that ageing of the bituminous binder is manifested as an increase in its stiffness (or viscosity see Figure 3.2) which consequently results in the stiffening of the mixture. Section 5, Volume 7 of the 1999 version of the UK Design Manual for Roads and Bridges, opined that conventional binders, as thin films on the aggregate particles, age in the presence of air leading to fretting and ravelling (loss of aggregate in the surface), cracking, and finally to failure. It was further stated that the rate of change in the binder depends on the voids in the mixture.

Bearsley et al (2004) in their studies reported that the structure and fluorescence of the asphaltene phase does not appear to alter radically upon oxidative ageing.

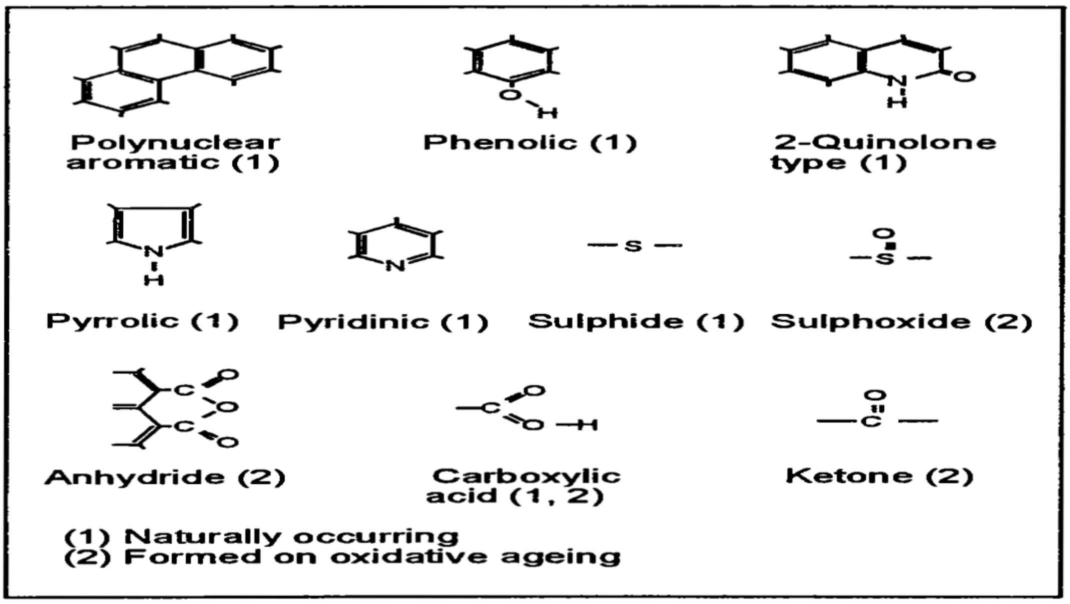


Figure 3.1: Chemical Functionalities in Bitumen Molecules Normally Present or Formed on Oxidative Ageing (Petersen, 1984)

Table 3.2: Chemical Functional Groups Formed in Bitumens During Oxidative Ageing (Petersen et al, 1974)

Bitumen	Concentration (moles per litre)				Average Hardening Index[b]
	Ketone	Anhydride	Carboxylic Acid[a]	Sulphoxide	
B-2959	0.50	0.014	0.008	0.30	38.0
B-3036	0.55	0.015	0.005	0.29	27.0
B-3051	0.58	0.020	0.009	0.29	132.0
B-3602	0.77	0.043	0.005	0.18	30.0

Note: Column oxidation, 130ºC, 24hours, 15µm film
[a] Naturally occurring acids have been subtracted from reported value.
[b] Ratio of viscosity after oxidative ageing to viscosity before oxidative ageing.

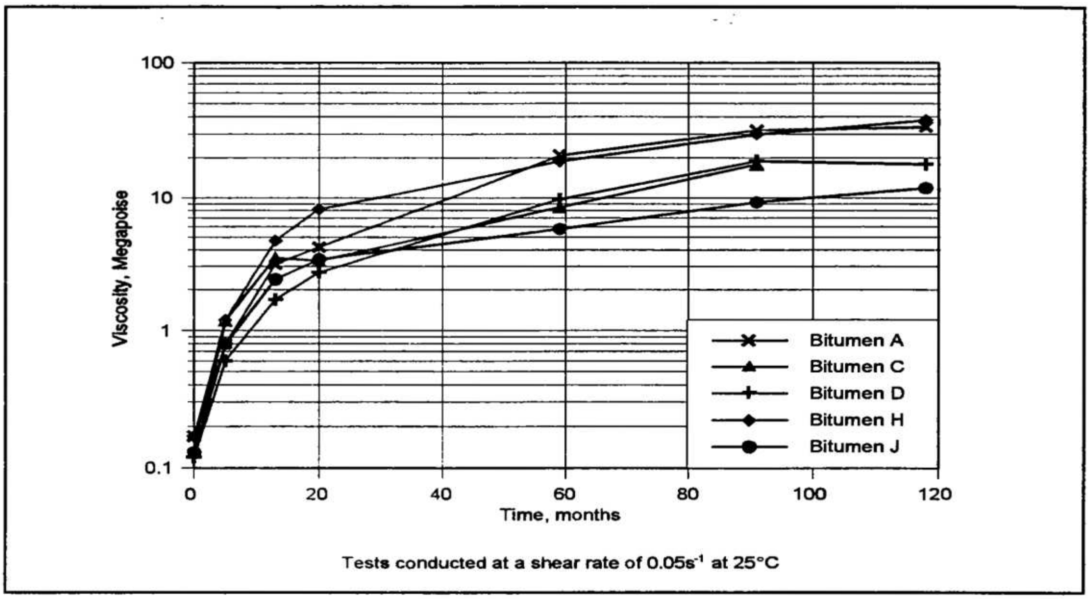


Figure 3.2: Viscosity Change of several Bitumens During Service in Pavements (Zube and Skog, 1969)

Walubita et al (2005) reported that, ageing reduced HMAC mixture fatigue resistance and its ability to heal and that, ageing plays a significant role in HMAC mixture fatigue performance and subsequently advised that it should be incorporated in fatigue design and analysis. Airey et al (2005) are of the opinion that, age hardening can have two effects, either increasing the load bearing capacity and permanent deformation resistance of the pavement by producing a stiffer material or reducing pavement flexibility resulting in the formation of cracks with the possibility of total failure. Lately, Romera et al (2006) argued that asphaltene contents increase with ageing and that aged bitumen needs a high mixing temperature (>200°C) to behave like a fluid material able to wet, adhere and envelop aggregate.

It is thus obvious from the account of previous researchers that, all things being equal, ageing in bituminous mixtures could play a beneficial role most especially when it is short term in nature as it assists the asphalt layer in a flexible pavement to support the designed traffic loads, since it moderately stiffens the asphalt layer. On the contrary, when the asphalt layer is aged severely or excessively, it consequently leads to significant hardening, which imposes some undesirable attributes on the asphalt layer in which its ability to flex gets reduced, and thus the pavement assumes a position in which it is at the risk of imminent failure by cracking since the fatigue life has reduced drastically. Ageing too in this regard, could cause permanent deformation, ravelling and fretting among others. All these eventually result in surface defects which ultimately lead to the total failure of the road if un-attended to.

3.5 Laboratory Protocols that Simulate Field Ageing in Bituminous Road Pavements

The need for laboratory simulations generally to mimic the behaviour of engineering structures while in service cannot be overemphasized. Laboratory simulations serve as avenues through which the performances of such structures under a near to service/field condition are evaluated before being finally put in place so that such structures would eventually serve their intended design functions. Moreover, a pavement which is the structure being considered here, involves a huge investment, thus every effort at the disposal of road builders must be harnessed to ensure that such roads perform their roles efficiently and effectively and also meet the design life at moderate costs. To this end, lots of simulations have been carried out and are still ongoing to ascertain the behaviour/performance of asphalt pavements while in service. One such is the effect of ageing on the performances of bitumen and bituminous mixtures. Mastrofini and Scarsella (1999) observed that quite a number of researchers have tried to simulate in the laboratory the ageing that takes place in service on asphalt pavements in order to foresee bitumen behaviour during application and service life. Notable among these are the works carried out by Bell (1989) which was sponsored under the Strategic Highway Research Program (SHRP) of the National Research Council in the USA, and Scholz (1995) which was carried out under the BITUTEST programme in the UK. In his work, Bell (1989) summarised the various methods that have been used to simulate field ageing in binders and bituminous mixtures. Similarly, Airey (2003) did an appraisal on the different laboratory methods that have been advanced by researchers for simulating ageing that occurs in the field in asphaltic pavements both in the short and long term. Indeed, these works showed that there are a lot of methods being used. Some of these methods are summarised in Tables 3.3 and 3.4.

Ever since the works of Bell and Airey, a number of simulations have been conducted by researchers in this regard. For example, Mouillet et al (2003) developed a new ageing protocol in which ageing of sample was done by the simulated oxidation of Polymer Modified Bitumens (PmBs) in the ageing cell, by heating the sample at 130°C for 2 hours under synthetic air (80% N_2, 20% 0_2). Before this, the cell was heated under neutral gas from 25°C to 130°C at 111°C/min heating rate. They eventually concluded that the method used for simulating ageing in their work would be useful for other kinds of organic materials for which a precise ageing knowledge is essential.

In another elaborate work, Hachiya et al (2003) discovered that asphalt concretes seem to become hard and brittle owing to both heat ageing and exposure to natural conditions. They also confirmed that ageing of asphalt in natural conditions varies with depth below the surface. They noted that asphalt near the surface hardens to a greater extent and that flexural strength changes fairly little with ageing time, strain at failure decreases clearly with ageing time and asphalt concrete mixture increases in stiffness with ageing time. They further remarked that the change in composition of bitumen due to ageing is a decrease in aromatics and an increase in asphaltenes and that however, heat ageing merely causes hardening. They also found out that samples from oxidative ageing of asphalt concrete had flexural strength and stiffness that increased with the duration of accelerated oxygen ageing. These authors opined that as asphalt cannot be aged in this procedure as well as in outdoor exposure, a

Table 3.3: Bitumen Ageing Methods (Airey, 2003)

Test method	Temperature (°C)	Duration	Sample size (g)	Film thickness	Extra features
Thin film oven test (TFOT) (Lewis and Welborn, 1940)—ASTM D1754, EN 12607-2	163	5 h	50	3.2 mm	-
Modified thin film oven test (MTFOT) (Edler *et al.*, 1985)	163	24 h	-	100 μm	-
Rolling thin film oven test (RTFOT) (Hveem *et al.*, 1963)—AASHTO T240, ASTM D2872, EN12607-1	163	75 m	35	1.25 mm	Air flow—4000 ml/min
Extended rolling thin film oven test (ERTFOT) (Edler *et al.*, 1985)	163	8 h	35	1.25 mm	Air flow—4000 ml/min
Nitrogen rolling thin film oven test (NRTFOT) (Parmeggiani, 2000)	163	75 m	35	1.25 mm	N_2 flow—4000 ml/min
Rotating Flask Test (RFT)—DIN 52016, EN12607-3	165	150 m	100	-	Flask rotation—20 rpm
Shell microfilm test (Griffin *et al.*, 1955)	107	2 h	-	5 μm	-
Modified Shell microfilm test (Hveem *et al.*, 1963)	99	24 h	-	20 μm	-
Modified Shell microfilm test (Traxler, 1961; Halstead and Zenewitz, 1961)	107	2 h	-	15 μm	-
Rolling microfilm oven test (RMFOT) (Schmidt and Santucci, 1969)	99	24 h	0.5	20 μm	Benzene solvent
Modified RMFOT (Schmidt, 1973)	99	48 h	0.5	20 μm	1.04 mm ϕ opening
Tilt-oven durability test (TODT) (Kemp and Prodoehl, 1981)	113	168 h	35	1.25 mm	-
Alternative TODT (McHattie, 1983)	115	100 h	35	1.25 mm	-
Thin film accelerated ageing test (TFAAT) (Petersen, 1989)	130 or 113	24 or 72 h	4	160 μm	3 mm ϕ opening
Modified rolling thin film oven test (RTFOTM) (Bahia *et al.*, 1998)	163	75 m	35	1.25 mm	Steel rods
Iowa durability test (IDT) (Lee, 1973)	65	1000 h	TFOT residue—50	3.2 mm	2.07 MPa—pure oxygen
Pressure oxidation bomb (POB) (Edler *et al.*, 1985)	65	96 h	ERTFOT residue	30 μm	2.07 MPa—pure oxygen
Accelerated ageing test device/Rotating cylinder ageing test (RCAT) (Verhasselt and Choquet, 1991)	70–110	144 h	500	2 mm	4–5 l/h—pure oxygen
Pressure ageing vessel (PAV) (Christensen and Anderson, 1992)	90–110	20 h	RTFOT or TFOT residue—50	3.2 mm	2.07 MPa—air
High pressure ageing test (HiPAT) (Hayton *et al.*, 1999)	85	65 h	RTFOT residue—50	3.2 mm	2.07 MPa—air

Table 3.4: Bituminous Mixture Ageing Methods (Airey, 2003)

Test method	Temp (^{0}C)	Duration	Sample Size/Condition	Extra Features
Production Ageing (Von Quintas, 1988)	135	8,16,24,36h	Loose material	-
SHRP short-term oven ageing (STOA)	135	4h	Loose material	-
Bitutest protocol (Scholz, 1995)	135	2h	Loose material	-
Ottawa sand mixtures (Paul and Welborn, 1952)	163	Various periods	50 x 50mm^2 cylinders	-
Plancher et al (1976)	150	5h	250 x 40mm^{2}Φ	-
Ottawa sand mixtures (Kemp and Prodoehl, 1981)	60	120h	-	-
Hugo and Kennedy (1985)	100	4 or 7 days	-	80% RH
Long-term ageing (Von Quintas, 1988)	60	2 days	Compacted Specimens	-
	107	3 days		
SHRP long –term oven ageing (LTOA)	85	5 days	Compacted Specimens	-
Bitutest protocol (Scholz, 1995)	85	5 days	Compacted Specimens	-
Kumar and Goetz (1977)	60	1, 2, 4, 6, 10 days	Compacted Specimens	Ait at 0.5mm of water
Long-term ageing (Von Quintas, 1988)	60	5 to 10 days	Compacted Specimens	0.7MPa -air
Oregon mixtures (Kim et al, 1986)	60	0,1 ,2, 3, 5 days	Compacted Specimens	0.7MPa -air
SHRP low pressure oxidation (LPO)	60 or 85	5 days	Compacted Specimens	Oxygen-1.9l/min
Khalid and Walsh (2000)	60	Up to 25 days	Compacted Specimens	Air-3l/min
PAV mixtures (Korsgaard, 1996)	100	72h	Compacted Specimens	20.7MPa -air

further modification such as a combination of heat ageing and oxygen should be adopted to simulate actual ageing. The work ended by concluding that ageing rate of asphalt concrete depends upon various factors such as asphalt type, asphalt content, aggregate, and environment. They also remarked that ageing is more pronounced in the top 5mm layer of the pavement than the other parts of the pavement, thus ageing decreases with depth, and that even if heat ageing could be used to evaluate hardening of asphalt concretes in terms of mechanical properties, it cannot be used to evaluate the changes in physical and chemical properties of asphalt.

On the same note, Walubita et al (2005) used three ageing protocols in their study. The exposure conditions lasted for 0, 3, and 6months at 60°C, simulating up to approximately 12 years of Texas field Hot Mix Asphaltic Concrete (HMAC) at the critical pavement service temperature with no explanation on what critical meant. All loose HMAC mixtures were subjected to the standard AASHTO PP24hr short-oven ageing process at 135°C prior to 60°C ageing. After HMAC mixture testing, aged binders were extracted for testing to characterize the binder's chemical and physical properties. They found out that the mixture tensile test conducted at 20°C indicated that as HMAC ages, it becomes more brittle, thus breaking under tensile loading at a lower strain level.

Indeed the results of these simulations are instructive. Bell et al (1994) reported that two days of long term oven ageing at 85°C is representative of pavement up to 5 years old depending on climate. Four days of oven ageing at 85°C appears to be representative of field ageing of about 15yrs in a Wet-No Freeze zone and about 7yrs in a Dry Freeze zone. Oven ageing at 100°C for one, two and four days causes similar changes in modulus to 85°C ageing for two, four and eight days, but damages the samples in the process. Oven ageing at 85°C is thus considered to be more reliable though nothing was mentioned about the nature of the damage caused by oven ageing at 100°C. In line with these assertions, Hinds (2007) in his analysis, and also articulated by Maguire (2008), stated that, using the SHRP long term oven ageing technique reflects an effect equivalent to 15years in a wet no freeze climate. Kulash (1994) had earlier reported that, in the Pressure Ageing Vessel, bitumen aged at 2.07MPa air for 20 hours at 100°C simulates a 5year aged level in a pavement in the temperate region, and that the forced draft oven is able to simulate field ageing (short and long term). Airey et al (2005) observed that the Saturation Ageing Tensile Stiffness Test (SATS) was able to reproduce the 60% reduction in stiffness modulus found for the acidic aggregate HMB mixture in the field.

It could be summarised from the foregoing that, ageing tests are basically two i.e. those carried out on binders and those carried out on bituminous mixtures. Airey (2003) listed ageing tests for bituminous binders thus:

- Extended Heating Procedures;
  - Thin Film Oven Test (TFOT),
  - Rolling Thin Film Oven Test (RTFOT),
  - Rotating Flask Test (DIN 52016),
  - Shell Microfilm Test,
  - Rolling Microfilm Oven Test (RMOT),
  - Tilt-oven Durability Test,
  - Thin Film Accelerated Ageing Test,
  - Modified Rolling Thin Film Oven Test,
- Oxidative (Air Blowing) Procedures;
  - Iowa Durability Test,
  - Pressure oxidation Bomb,
  - Accelerated Ageing test Device/Rotating Cylinder Ageing Test (RCAT),
  - Pressure Ageing Vessel (PAV),
  - High Pressure Ageing Test (HiPAT),
- Ultraviolet and Infrared Light Treatments;
- Microwave Ageing;
- Steric Hardening (no tests that address this phenomenon at the moment).

While ageing tests for asphalt mixtures could be:

- extended heating procedures,
- oxidation tests,
- ultraviolet/infrared treatment and,
- steric hardening.

Airey (2003) observed that the TFOT and RTFOT are the most commonly used short term ageing tests for simulating the hardening that occurs during asphalt mixture production. Currently, the most commonly used binder tests to simulate long-term ageing are the PAV and RCAT. In terms of long term ageing, no one test seems to be satisfactory for all cases and the RCAT method, based on a kinematic approach to ageing is probably the most acceptable. On the same note, Airey (2003) recorded that the most promising methods for short term ageing of asphalt mixtures are extended heating of the loose material and extended mixing and that the most promising methods for long term ageing of mixtures include extended oven ageing, such as the SHRP long-term oven ageing method, pressure oxidation, using low pressure oxidation as well as pressurised procedures, and ultraviolet and infrared light treatments. Meanwhile tests on mixtures could be conducted on either a compacted sample or a loose sample.

Bell (1989) on a comparative note, observed that research mostly has been focusing on bitumen, and that there has been little research on the ageing of asphalt mixtures, and to date, there is no standard test. Airey (2003) reasoned in line with this assertion when he stated that extended heating procedures show the most promise for short term ageing, and pressure oxidation and/or extended heating the most promise for long term ageing. He further advised that the condition for choosing whatever method used is that it should simulate conditions in the field and emphasize oxidation. He also remarked that the forced draft oven promises to be useful in extended oven ageing over the conventional ovens and however cautioned that, for mixtures, an elevated temperature level used in tests could cause specimen disruption. He also expressed a concern/need for the evaluation of the integrity of the compacted samples at high temperatures since it has been purportedly envisaged that the internal structure of a sample may be disrupted at temperatures such as 140°C and that in extreme cases the samples may slump. It must be noted that all these works have been carried out to simulate what obtains in the temperate regions of the world.

In conclusion, this review showed that temperature is an important factor for simulating accelerated ageing of asphalt mixtures in the laboratory. Also physical hardening of residual binder in the mix as a consequence of ageing could affect both the chemistry and rheology of bituminous binders. The review also showed that extended ageing of either compacted or loose asphalt mixtures in the forced draft oven are considered suitable to simulate both short and long term ageing in the laboratory. Ageing of compacted specimens is considered appropriate for cases where samples are to undergo further testing for mechanical properties such as stiffness modulus. Ageing of loose samples in particular is deemed appropriate for cases where aged materials are to be used as RAP in reconstituted asphalt mix for studies on recycled asphalt mixtures. Although ageing of compacted specimens is not used in the study discussed in this thesis, the softening point of the binder in the asphalt mix can be used as a guide for choosing ageing temperature to forestall specimen disruption. Ageing loose mixtures is a more practical means for achieving uniformly aged asphalt mix in a short laboratory time. This method also enables easy reconstitution of aged asphalt into new asphalt mix when compared to aged compacted specimens. It has the advantage that ageing can be conducted at high temperature, though there is the concern that excessive and high ageing temperatures can potentially damage the binder in the mix such that the physico-chemical and rheological properties of the binder are significantly different from what is obtained on the road pavement.

3.6 Cold Mixes

Generally, there are basically three types of asphalt mixes used in road construction. These are the Hot Mix (which is the conventional), the Cold Mix, and the Warm Mix. This work focuses on cold mixes for the obvious reasons stated earlier in Chapter 1 of this report. The term cold is used in the sense that virtually all the operations involved in the production of the material are carried out at ambient temperature. Needham (1996) stated that cold mix is manufactured at ambient temperatures, although some processes can use the emulsion warmed to around 60°C. Meanwhile, the materials being used for the preparation of cold mixes are very similar to those in hot mix, and as was stated in Chapter 2 of this write up, the major difference is that the bituminous binders used in cold mixes are liquefied and applied at relatively low temperatures compared to that of hot mix. Achieving these would mean that the binder is either emulsified or foamed. Since the use of water is normally involved in these two processes, hydraulic binders too are usually applied to facilitate among other things, the rapid evaporation of water in the mix. Thanaya (2003) listed the most commonly used types of Cold Bituminous Mixtures as:

- cold lay Macadam (cutbacks),
- grave emulsions (developed in France),
- foamed bituminous mixtures,
- CBEMs (Emulsified Asphalt Materials – USA) (Ibrahim and Thom, 1997).

Many researchers in this field are of the opinion that, in terms of energy savings, cold mixtures appear to be much more efficient than hot mixtures (Needham, 1996; Ibrahim, 1998; Thanaya, 2003; and Thom, 2008). Needham (1996) stated that, cold mix can be manufactured to cater for a range of different applicational regimes and that they are used mainly for base course and sometimes for binder or wearing course. They are applied through a number of methods ranging from hand application, through graders, finishers or pavers to self contained mixing and laying plants. Compaction regimes for cold mix are quite varied at present as different companies advocate and utilize different techniques. However, the preferred method seems to be steel rolling, followed by very heavy pneumatic tyred roller and finally finishing with steel. Meanwhile, PIARC (2003) stated that since cold mixes are difficult to compact, a continuous grading is suggested and that gap gradations and stone matrix are generally unsuitable for cold recycling. The grading envelopes used for the design of grave emulsion can be taken as a reference when using emulsion as the recycling agent in cold recycling.

In cold mixes, and particularly in the case of recycling, in addition to the specific area, it is necessary to take into account the capability of absorption of water and the chemical nature of the mineral surface. Needham (1996) opined that cold mix wearing course can be used on all but the most highly trafficked roads. Mixtures can be produced in specialised mixing plant, motor pavers or simple concrete mixers in which case the mixture can be produced on site. Also in line with Needham's assertions, but now considering the use of RAP, Carswell et al (2008) in an overview of the design guide and specification that is now the common practice for recycling in the UK, reported that cold recycling using a plant mixed process (*ex situ*) allows for screening and crushing of aggregates, prior to mixing with binder(s) in plant located nearby and the laying of materials in one or more layers using a paver. It also gives room for the introduction of alternative aggregates from sources other than the existing pavement. They stated further that, provided that the cold recycled materials can achieve the desired performance, the potential use of cold recycling is not limited. However, each site needs to be evaluated for the most appropriate maintenance selected in terms of:

- location,
- proximity of suitable location for setting up ex situ plant,
- proximity of source(s) of alternative materials, if required,
- type(s) and severity of deterioration,
- extent of deterioration,
- location of services within the pavement construction,
- condition of drainage,
- edge detail and verge condition.

Despite of all these advantages, cold mixes are still generally classified as inferior to hot mixtures with respect to performance, although the engineering equivalence and practical difficulties in adopting cold mixtures formulations have not yet been clearly defined (Biczysko, 1996). Carswell (2004) remarked that, looking at cold mixes, for example, despite the advantages for environment and health, they are still perceived as asphalt 2nd Class, used for reinforcement, re-profiling or for micro-surfacing, only where unavoidable, and only on secondary roads. Leech (1994) reported that, cold bituminous emulsions mixes are not so common in the UK because of the cold climatic conditions, more over in the UK there are sufficient HMA plants available and less remote areas- a serious challenge for CBEMs (Ibrahim and Thom, 1997; Khalid and Eta, 1997).

This position must have been premised on the fact that such a cold climatic condition would normally inhibit the rapid evaporation of water in the mix. Thanaya (2003) maintained that, cold mixes did not receive attention until 1992 in the UK. To this end, Zoorob and Thanaya (2002) remarked that Cold bituminous emulsion mixtures (CBEMs) are more universally accepted for low to medium traffic conditions, for works in remote areas and for small scale jobs such as reinstatement work. These notwithstanding, cold mixes have been used successfully in France, South Africa, Sweden and USA among other countries since the 1970s to meet various needs on the road (Thanaya, 2003; and Needham, 1996) and thus cold mixes hold a lot of promise. Carswell (2004) in buttressing this opined that recent experiences with cold mixes have been showing that they are far beyond the state of being assumed as asphalt 2nd class. He further stated that, an understanding of these mixes is improving, and it is sure that they are finding their ways to the high class, and thus would help gain a higher market share all over Europe. Zoorob and Thanaya (2002) in their work further strengthened this opinion in which they found out that, CBEMs with added cement at full curing can be comparable to conventional hot bituminous mixtures in terms of indirect tensile stiffness modulus.

In retrospect, it is worth noting that most of the previous works have been considering only virgin materials for cold mixes with the exception of Zoorob and Thanaya (2002) and Thanaya (2003) that looked at the incorporation of waste materials such as PFA, Red Porphyry Sand, Synthetic Aggregates, Steel Slag and Crumb Rubber into CBEMs. Most researchers only mentioned the possibility of using RAP in passing (Needham, 1996; Ibrahim, 1998; and Thanaya, 2003), and where they are used they are just regarded as black rocks without any regard to the properties of the residual binder in the RAP. The only related work to the author's knowledge is still ongoing at the University of Iowa, USA, which concerns cold-in place recycling typical of a temperate region (Lee, 2007). This present work focuses on cold bituminous emulsion mixtures with RAP and virgin materials as aggregates in a circumstance surrounded by tropical condition and as well *ex situ* recycling.

3.7 Design of Cold Mixes

The importance of adequate design to all engineering structures cannot be overestimated. Without any prejudice, it is a commonly acknowledged fact that much advancement has been made in the design of hot bituminous mixtures compared to cold bituminous mixtures. While definite universal guidelines/standards have been established and are still being improved for the design of hot bituminous mixtures, Thanaya, (2007), Thanaya (2003), Zoorob and Thanaya (2002), Ibrahim (1998), Asphalt Institute and Asphalt Emulsion Manufacturers Association, 1997 (1997), FHWA (1997) and Needham (1996) among other researchers stated that there is no widely accepted mixture design method or structural design methodology for either virgin emulsion aggregate mixtures or cold recycled materials. This is one of the reasons why most pavement engineers feel more able to specify hot bituminous mixtures than cold bituminous mixtures.

This notwithstanding, FHWA (1997) affirmed that guidelines have been developed by several agencies, based on laboratory tests, empirical formulae or past experience with identical projects. It further stated that generally, the RAP particles are treated like black rock or aggregates in cold-recycled mix design and that however, the most commonly used recycling agent for complete cold recycling processes are emulsified asphalt cements (bitumen). This is because the emulsions are liquid at ambient temperatures and have the capacity for being dispersed throughout the mix and do not cause major air pollution problems, though quite a number of researchers have also found foamed bitumen useful in cold recycling (Jitareekul, 2009; Sunarjono, 2008; Jitareekul et al, 2007; Kim et al, 2007; Kim and Lee, 2006; Loizos and Papavasiliou, 2006; Long and Theyse, 2004; Romanoschi et al, 2003; PIARC, 2003; Jenkins, 2000; and GEOPAVE, 1993). Needham (1996), Asphalt Institute and Asphalt Emulsion Manufacturers Association, (1997), and Montepara and Giuliani (2002) remarked that the Marshall or Hveem design methods or modified versions are nearly always used on such occasions. Montepara and Giuliani (2002) opined that these design methods essentially investigate the best dosing of the bituminous emulsion and the total content of liquids (optimum fluid content), and that however, to study the same mixtures with added cement, it is necessary to consider other parameters, such as the water/cement ratio and the monitoring of the time to obtain certain mechanical properties. Needham (1996) opined that, if laboratory tests are not possible, empirical formulae for the addition level of emulsion to densely graded mixtures (see Table 3.5 for typical aggregates for dense-graded emulsion mixtures) are provided as a substitute and that the formula for the

Table 3.5: Aggregates for Dense-Graded Emulsion Mixtures (Asphalt Institute and Asphalt Emulsion Manufacturers Association, 1997)

Sieve Size	Semi-Processed Crusher, Pit or Bank Run	Processed Dense-Graded Asphalt Mixtures Percent Passing by Weight				
50.0mm (2 in.)	-	100	-	-	-	-
37.5mm (1-1/2 in.)	100	90-100	100	-	-	-
25.0mm (1 in.)	80-90	-	90-100	100	-	-
19.0mm (3/4 in.)	-	60-80	-	90-100	100	-
12.5mm (1/2 in.)	-	-	60-80	-	90-100	100
9.5mm (3/8 in.)	-	-	-	60-80	-	90-100
4.75mm (No. 4)	25-85	20-55	25-60	35-65	45-70	60-80
2.36mm (No. 8)	-	10-40	15-45	20-50	25-55	35-65
1.18mm (No. 16)	-	-	-	-	-	-
600µm (No. 30)	-	-	-	-	-	-
300µm (No. 50)	-	2-16	3-18	3-20	5-20	6-25
150µm (No. 100)	-	-	-	-	-	-
75µm (No. 200)	3-15	0-5	1-7	2-8	2-9	2-10
Sand Equivalent, Percent	30 min.	35 min.	35 min.	35 min.	35 min.	35 min.
Los Angeles Abrasion @ 500 Revolutions	-	40 max	40 max	40 max	40 max	40 max
Percent Crushed Faces	-	65 min.	65 min.	65 min.	65 min.	65 min.

initial residual binder should be as shown below in equation 3.1:

$$P = (0.05A + 0.1B + 0.5\,C) \text{ x } (0.7) \tag{3.1}$$

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

Thanaya (2007) having reviewed and experimented with some guidelines for the design of cold mix in his work proposed that cold mix design should consider:

1 Determination of aggregate gradation, which should be continuous due to its interlock property, giving strength to the material during early life strength of cold mixes. This can be obtained using Cooper's equation:

$$P = \frac{(100-F)(d^n - 0.075^n)}{D^n - 0.075^n} + F \tag{3.2}$$

Where P is the percentage material passing sieve size d (mm), D is maximum aggregate size (mm), F is the percentage filler, and n, an exponential value that dictates the concavity of the gradation line.

2 Estimation of initial residual bitumen content and initial emulsion content in line with equation (3.1) above.

3 Coating test or binder compatibility test.

4 Determination of compaction level to meet porosity target;
   a. Storage time for the loose mixture and condition- sealed and unsealed,
   b. Compaction by applying an initially judged compaction effort
   c. Curing for dry density determination,
   d. Determination of specific gravity of the mix and porosity value after obtaining dry density data,
   e. Adjustment of compaction effort and determination of compaction effort to meet the porosity target. Suggested porosity target is 5-10%. The following useful formulae were also suggested:

$$SG_{mix} = \frac{100}{\frac{\%CA}{SGCA} + \frac{\%FA}{SGFA} + \frac{\%F}{SGF} + \frac{\%Binder}{SGBinder}} \tag{3.3}$$

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

By weight of total mix:

$$\text{Bulk Density} = \frac{\text{\%Weight in air}}{\text{WeightSSD} - \text{Weight in water}} \qquad (3.4)$$

Weight saturated surface dry (SSD) is obtained by towel drying the samples after weighing in water, until no air bubble occurs.

$$\text{Porosity}(\text{P} - \%) = \left(1 - \frac{\text{Bulk Density}}{\text{SG}_{\text{mix}}}\right) \text{x } 100\% \qquad (3.5)$$

5 Variation of residual bitumen content.
6 Determination of optimum residual bitumen content.
7 Calculation of bitumen film thickness at optimum residual bitumen content. The bitumen film thickness (BFT) can be calculated using the formula:

$$\text{BFT} = \frac{\text{\%Binder}}{100 - \text{\%Binder}} \text{x} \frac{1}{\text{SGBinder}} \text{x} \frac{\text{\%F}}{\text{ASA}} \qquad (3.6)$$

Where ASA is the aggregate surface area and requires surface area factor as given Table 3.6, the calculation of the aggregate surface area (ASA) is obtained as shown Table 3.7. The ASA is calculated by multiplying the total percent passing each sieve size by the appropriate SAF and adding together. The minimum BFT to be targeted is 8 micron.
8 Determination of retained stability of the mixture at optimum residual bitumen content only, according to the design curing procedure.
9 Determination of the ultimate strength at full curing of the samples at optimum residual bitumen content only.
10 Incorporation of about 1-2% cement by mass of aggregates to improve the performance of the cold mix.

Also, a general flow chart for the design of mixtures as suggested by Jenkins (2000) is shown in Figure 3.3.

Although Thanaya (2007) concluded that this design method was found to be simpler than those reviewed, his work was conducted in the light of the conditions

Table 3.6: Surface Area Factor (SAF) (Thanaya, 2007)

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
300μm (No. 50)	6.14
150μm (No. 100)	12.29
75μm (No. 200)	32.77

Table 3.7: Calculation of Aggregate Surface Area (ASA) (Thanaya, 2007)

Sieve		ASA Calculation		
Inch/No.	mm*	Estimated Total Pass (%)**	SAF	ASA (m^2/kg)
		a	**b**	**c = a x b**
3/4″	19.0	100	0.41	0.4100
3/8″	9.5	-		
No. 4	4.75	58	0.41	0.2378
No. 8	2.36	41.5	0.82	0.3403
No. 16	1.18	28.8	1.64	0.4723
No. 30	600μm	19.6	2.87	0.5625
No. 50	300μm	12.7	6.14	0.7798
No. 100	150μm	7.7	12.29	0.9463
No. 200	75μm	4	32.77	1.3108
ASA (sum)				**5.0598**

* In line with particle size/sieve in Table 3.7
**Estimated based on the mixture's aggregate grading curve from Cooper's formula curve i.e. equation 3.1

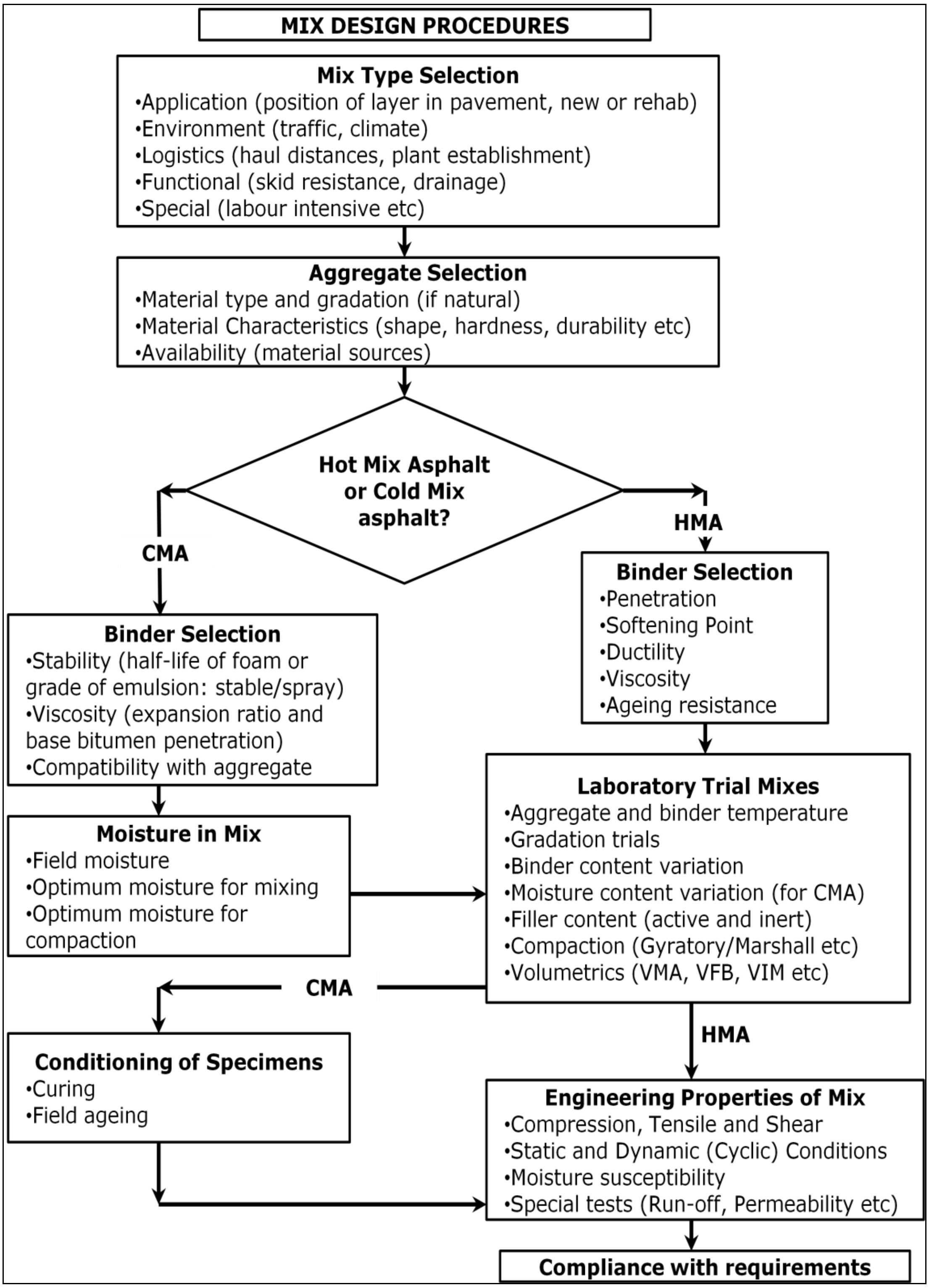


Figure 3.3: A General Procedure for Mix Design (Jenkins, 2000)

that obtain in temperate regions. In the same vein, while he used cationic bitumen emulsions along with limestone, RAP and granite were never experimented with in his work.

3.8 Curing Protocols

Unlike hot mixes, adequate and proper curing is a requirement for cold mixes (Thom, 2009). Jenkins (2000) defined curing of cold bituminous mixes, whether emulsion or foam as the process whereby the mixed and compacted material discharges water through evaporation, particle charge repulsion or pore-pressure induced flow paths. A reduction in moisture content leads to an increase in strength of the mix (both tensile and compressive). Roberts et al (1984) found a significant increase in tensile strength as curing temperature was increased from 23ºC to 60ºC. Bocci et al (2002) discovered that as curing time and curing temperature increase, the performances of the mixtures considerably improve; curing for 14 days at 20°C is almost equivalent to 7 days at 40°C. Thanaya, (2007), Leech (1994) and Santucci (1977) reported that full curing can occur between 2-24mths in the field. Table 3.8 details some curing methods while Figure 3.4 shows the effect of different curing conditions and compaction on the compressive strength of reference grave emulsion.

Jenkins (2000) consequently opined that the temperature of curing cannot be ruled out as unimportant to mix preparation as temperature and moisture are dependent variables with temperature influencing the rate of moisture loss. Meanwhile, Jenkins (2000) had earlier submitted that too little moisture impedes dispersion of the foam (emulsion), workability and compaction of the mix and too much moisture increases the curing time and reduces the density and strength of the compacted mix and that the moisture contents of mixes that are oven cured in an unsealed state are generally between 0% and 1.5% and always less than 4%, which is seldom representative of field conditions. In addition, the influences of curing temperature on changes in the binder condition have not been analysed in the literature, which is unfortunate considering the high surface area of the binder and higher void contents in foamed mixes. The challenge is to select the appropriate curing conditions in the laboratory, ensuring adequate shear strength in early life and selecting the correct stiffness for the structural design life. In addition, compaction due to traffic requires consideration. In a related view, Serfass et al (2003) believed that evaluating cured cold mixes in the laboratory is clearly necessary, but reproducing exact field curing conditions is too complicated and, above all, time consuming, and thus an accelerated curing method is

Table 3.8: Some Curing Methods for Foamed Bitumen (Jenkins, 2000)

Curing Method	Equivalent Field Cure	Reference
3 days @ 60°C + 3 days @ 24°C	Unspecified	Bowering (1970)
3 days @ 60°C	Construction period + early field life	Bowering and Martin (1970)
3 days @ 60°C	Between 23 and 200 days from Vane Shear Tests	Acott (1980)
1 day in mould	Short term	Ruckel et al (1982)
1 day in mould + 3 days @ 40°C	Between 7 and 14 days (intermediate)	Ruckel et al (1982)
1 day in mould + 3 days @ 40°C	30 days (Long term)	Ruckel et al (1982)
1 day @ 38°C	7 days	Asphalt Institute
10 days in air + 50 hours @ 60°C	Unspecified	Van Wijk and Wood (1983)
3 days @ ambient temp. + 4 days vacuum desiccation	Unspecified	Little el al (1983)
3 days @ 23°C	Unspecified	Roberts et al (1984)
3 days @ 60°C	Unspecified	Lancaster et al (1994)
3 days @ 60°C	1 year	Maccarrone et al (1994)

Note: 1. Specimens are cured in an unsealed state in the oven, unless otherwise stated.

2. Brennen et al (1981) developed the procedure to first cure the foamed specimens in the mould for 24hrs during the most fragile period.

3. Vacuum desiccation methods are in line with the Asphalt Institute design procedure (PCD-1) and require further investigation.

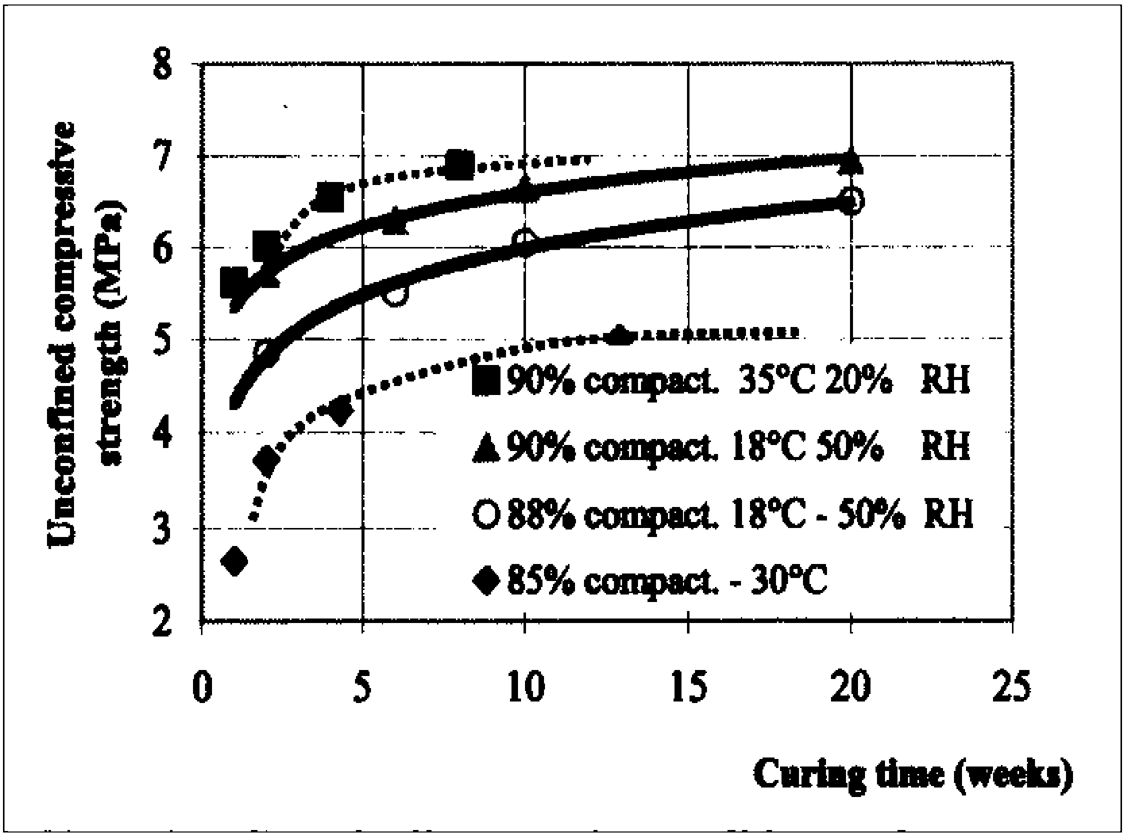


Figure 3.4: Effect of Different Curing Conditions and Compaction on Compressive Strength of Reference Grave Emulsion (Serfass et al, 2003)

necessary. The requirements are that:

- The curing procedure should be as short as possible,
- It must produce materials in a state as close as possible to their in-place mature state,
- It must not cause significant ageing of the bituminous binder,
- The laboratory equipment should not be too sophisticated.

Serfass et al (2003) further discovered in their study that the moisture content of small specimens decreases very quickly, whatever the temperature. For large specimens it takes longer. Also, too high a temperature (e.g. 50°C) tends to produce too quick a sample drying, which can cause cracking of large samples. They advised that, it should be borne in mind that, in the field, cold mixes rarely become totally dry. The moisture content has often been found to be between 0.5 and 1.5% in road bases in temperate climates. They established a relationship between the degree of compaction and stiffness modulus and thus stated that modulus values must therefore always be related to the degree of compaction of the mix tested. Therefore in order to obtain cold mixtures in a mature state, they proposed that such samples should be cured for 14 days at 35°C-20% Relative Humidity (RH). This procedure does not cause deterioration to the specimens. 35°C was chosen because it is realistic, as it prevails for long periods in the actual pavements. It was further proposed that curing over 14 days at 18°C – 50% RH would be appropriate for mix in a fresh state i.e. a few weeks after laying (18°C chosen because that is a typical average daily temperature for temperate climates).

3.9 Some Compaction Methods for Cold Mixes

Compaction is another important factor that is pivotal to the efficient performance of cold mixtures. In buttressing this, Harun and Morosiuk (1995) remarked that the full benefit from the use of bituminous materials in road construction can only be achieved if a satisfactory degree of compaction is attained during construction. They further opined that if shear failure is to be prevented then secondary compaction of the material after construction must be limited to a safe value and that an alternative to the use of additives is a modification of the mix design and compaction method to produce a more mechanically stable material, again making it more resistant to deformation at high temperatures. They thus proposed that design of mixes should be done using refusal density.

While Thanaya (2007) suggested that compaction should be done to target a porosity within the range 5-10% generally for cold mixes which thus implies heavy compaction effort, Jenkins (2000) opined that the distribution of binder within a foamed mix differs from that of HMA and the inclusion of the water phase sets these two mixes apart, in so doing introducing differences in compactability, and that a laboratory compaction technique that not only achieves the void content expected in the field, but also emulates the particle orientation after rolling is to be sought. He further advised that both the volumetrics and the engineering properties of the mix require consideration in the selection of an appropriate compaction technique and more reliable links between laboratory and field compaction are required. PIARC (2003) advised that, for the laboratory study of mix design for recycling, the total fluid content to insure adequate compaction of the mix must be determined. AEMA and AI (1997) remarked that for gradations containing appreciable fines, aeration or drying prior to compaction may be required. FHWA (1997) suggested that the rate at which the reaction between the recycling agent and the aged asphalt occurs is a function of the properties of the recycling agent and the aged asphalt cement, and the mechanical effects of the physical processes such as mixing, compaction, traffic and climatic conditions.

Thanaya (2007) further opined that the porosity of CBEMs can be reduced to meet a pre-selected target simply by increasing the compaction effort and that the compaction effort is a significant variable that needs to be determined depending on the target porosity, mixture type, storage conditions and storage time prior to compaction. Similarly, Kim et al (2007) categorically listed compaction method as one of the factors to be considered in design.

All these assertions of previous researchers indicate that compaction method employed in the laboratory is very important in the design of mixtures. However, the situation becomes more confusing as Needham (1996) stated that the field compaction regimes for cold mix are quite varied at present as different companies advocate and utilize different techniques and that however, the preferred method seems to be steel rolling, followed by very heavy pneumatic tyred roller and finally finishing with steel. To ascertain which compaction method is the best, researchers have made every effort to produce compaction methods in the laboratory that simulate what obtains in the field. Meanwhile, a lot of methods for the compaction of such mixtures in the laboratory have been developed. Needham (1996) listed them as the Marshall Hammer method, The

Percentage Refusal Density Apparatus, the Static Load Press, the Roller Compactor and the Gyratory Compactor. Ibrahim (1998) also used the Vibrating Compactor in his work but stated that Roller Compactors have not been used in the study of cold mixes. Some of these methods are detailed in Table 3.9.

Obviously, not all these methods have been found to be successful/applicable in all conditions and thus efforts have been made to identify the most suitable method which among other things mimics the compaction on the road, simple to operate and also affordable for different operations and as well materials. In the light of this, various studies have been conducted to ascertain best methods for compacting mixtures and as well to understand the effect of compaction on mixtures. Kim et al (2007) discovered from their studies that additional foamed asphalt helped mixtures compact better under gyratory compaction, but not under Marshall Hammer compaction. Smith and Jones (1998) in an earlier work reported that the 75-blow Marshall compaction underestimates the effect of secondary compaction under traffic and many of these surfacings suffer severe structural instability leading to plastic deformation. In line with this, Kim and Lee (2006) observed that 75 blows from the Marshall Hammer did not provide sufficient compaction to simulate initial compaction after construction. They opined that although the gyratory compaction method produced a higher density value than the Marshall Compaction method, the samples made by the gyratory compactor exhibited lower resilient modulus values than the ones prepared by the Marshall compactor.

In another similar work, Ulmgren (2003) observed that with an increased angle of compaction a predetermined degree of compaction is reached with fewer rotations. Figure 3.5 shows this relationship. On another note, Thanaya (2007a) stated that in order to achieve an air void content of between 5-10%, compaction was done using 240 gyros- extra heavy compaction (medium compaction is carried out at 80 gyros which is considered to be the equivalent of 50 blows using the Marshall hammer, while heavy compaction is carried out at 120 gyros, equivalent to 75 blows using the Marshall hammer). He concluded that the application of a heavier compaction level is inevitable in cold mixes, as the emulsion set and hence the mixes stiffen during compaction. Serfass et al (2003) in their work established a relationship between the degree of compaction and stiffness modulus. Figure 3.6 shows this relationship. Thanaya (2003) in his work observed that, compaction of CBEMs in two layers aided escape of moisture faster which eventually reduced the curing time.

Recently, Lee and Kim (2006) examined the compaction characteristics of RAP

Table 3.9: Summary of Laboratory Compaction Techniques used for Foamed Mix Design (Jenkins, 2000)

Compaction Method	Settings/Temperature	Remarks	Reference
Kneading Compactor	Ambient temperature	-	Shackel (1974)
Kneading Compactor	Ambient temperature	-	Bowering & Martin (1976)
Gyratory Compactor	Angle = 1° Ram pressure = 1.38 MPa	Optimum Bitumen Content = f(Degree of comp)	Tia and Wood (1982)
Texas Gyratory Compactor	25°C	-	Little et al (1983)
Gyratory	20 rev. with Ram pressure =1.38 MPa	12% higher density than 75 blows Marshall	Brennen et al (1983)
Gyratory Compactor	150 cycles, Angle = 2° Ram pressure = 0.24 MPa for 100mmΦ 150 cycles, Angle = 3° Ram pressure = 0.54 MPa for 150mmΦ	-	Maccarrone et al (1984)
PCG (French Gyratory Compactor)	200 cycles at French standard settings	LCPC carousel: PCG 200 gyrations ≡ 85% Solid density	Brosseaud et al (1997)

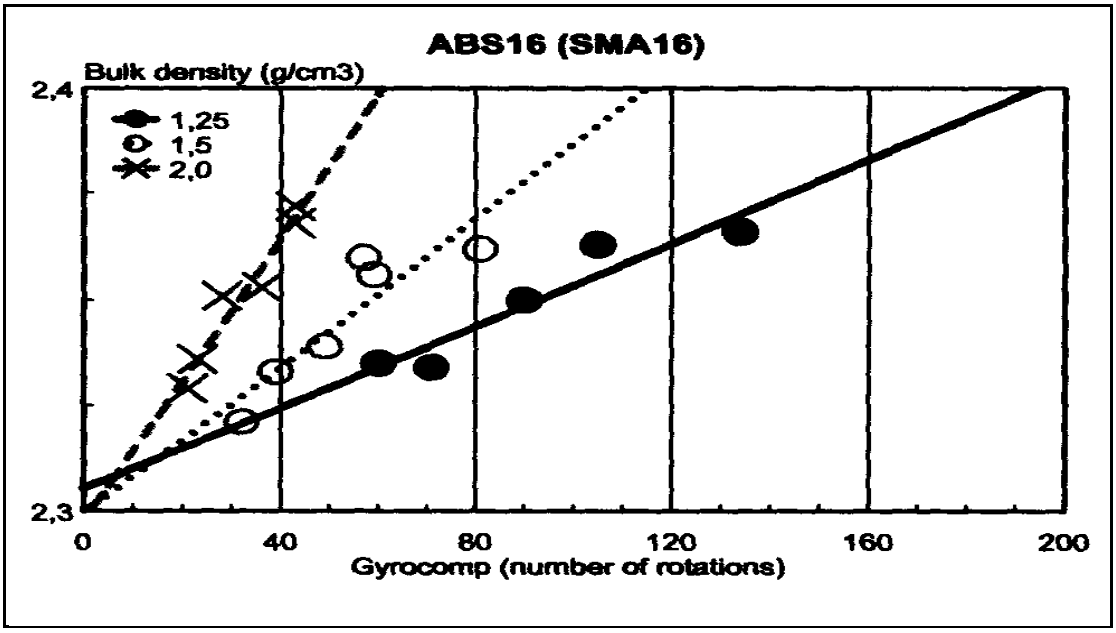


Figure 3.5: Relationship between Bulk Density and Number of Rotations (note, with an increased compaction angle i.e. 1.25 to 2°, the number of rotations needed to reach a given density is fewer, ABS is the Swedish equivalent of SMA) (Ulmgren, 2003)

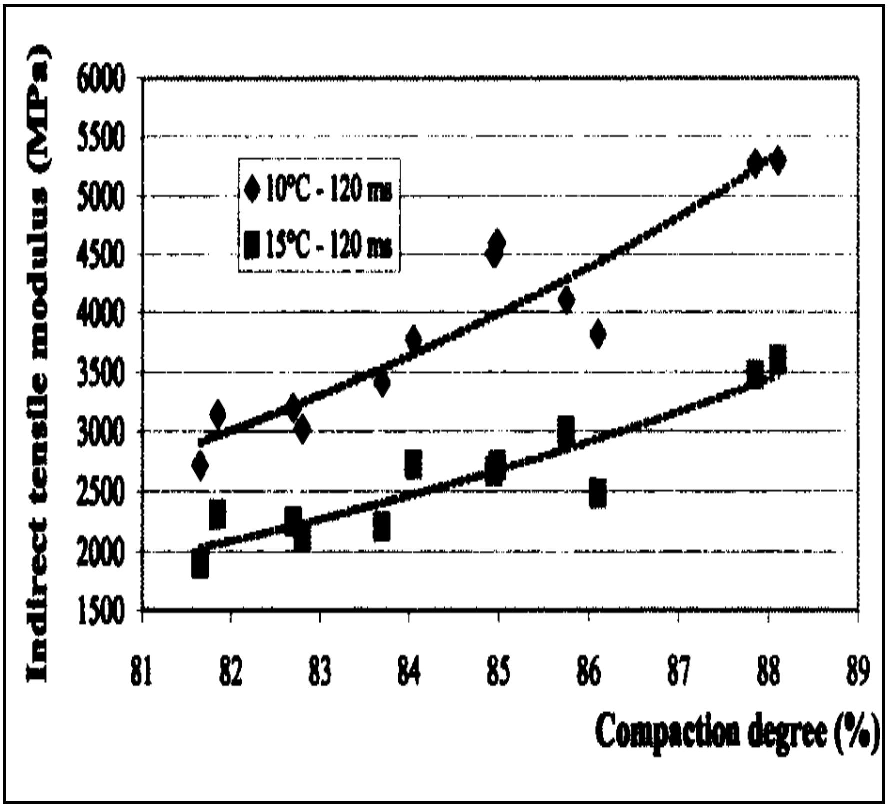


Figure 3.6: Influence of Compaction on Stiffness (Serfass et al, 2003)

material at different temperatures and moisture contents, RAP materials were compacted at temperatures of 25°C, 40°C and 55°C and moisture contents of zero and 4% using the gyratory compactor without adding any additional bitumen. The three temperatures were selected to represent the pavement surface temperatures in the field for early fall, late spring, and peak summer conditions respectively. Two samples were compacted up to 200 gyrations for each of six cases of three RAP temperatures and two moisture contents. The bulk specific gravities of compacted RAP samples with and without water were measured and plotted against the number of gyrations. The compacted samples with 4% water exhibited the higher bulk specific gravity than those without water and the higher RAP temperature also produced the higher bulk specific gravity.

Also, Lee and Kim (2007) investigated the compaction characteristics of RAP materials, and as a reference point, RAP materials were compacted using a gyratory compactor without adding water or foamed asphalt. They found a significant increase in bulk specific gravity by adding foamed asphalt. Moisture content was fixed at 4%. Specimens compacted by gyratory compactor at 30 gyrations or by Marshall Hammer at 75 blows were cured at 40°C for three days or 60°C for two days. The indirect tensile strength of gyratory compacted and vacuum saturated specimens was more sensitive to foamed asphalt contents than that of the Marshall hammer compacted vacuum saturated specimens. Brennan et al (2007) concluded in their work on compaction that, the voids content of a cold mixture can be up to 9% greater than that for a comparable hot mixture.

Indeed the development/adoption of compaction method in the laboratory which mimics what obtains in the field is essential for coming up with efficient designs most especially for cold mixes. As discussed earlier, it would be observed that a lot has been done on ascertaining the best method to adopt for simulating in the laboratory the compaction that takes place in the field. Most researchers have been favourably disposed to the gyratory compactor, which was developed under the SHRP programme in the USA. However, as most researchers rightly caution, both the volumetrics and the engineering properties of the mix require consideration in the selection of an appropriate compaction technique and more reliable links between laboratory and field compaction are required.

3.10 Performance Characteristics of Cold Mixes

Quite a number of investigations have been carried out to examine the performance of cold mixes. Through the outcomes of some of these investigations, the major problems of cold mixes have been identified, and procedures for mitigating them have been proposed. Meanwhile, performances of cold mixes are normally examined with reference to hot mixes. In a recent work, Thanaya (2007) observed that CBEMs compared well in mechanical properties with hot mixes as detailed in Table 3.10. On a similar note, Robinson (1997) using cores examined in the UK in his work as reported by Thanaya (2003), revealed that mixtures develop stiffness gradually, e.g. the ITSM value of a 6mm Dense Macadam (200pen base emulsion) met the specification requirements of 600MPa after 10mths and the value increased to almost 800MPa after 24mths. Thanaya (2007) further noted that considering the creep slopes of the cold mixes studied in the work, it was confirmed that cold mixes are suitable for low to medium trafficked roads and that the addition of 1-2% cement (see Tables 3.11a and b) by mass of aggregates into cold asphalt emulsion mixes significantly improves the overall mechanical properties of the CBEMs.

Brown and Needham (2000) in buttressing this submitted that adding OPC to bitumen emulsion mixtures (cold mix) has some beneficial effects. They revealed that without OPC, cold mix failed at less than 1000cycles in the unconfined mode of Repeated Load Axial Tests (RLAT), but suggested that it could do better if the triaxial mode is used. They further found out that OPC has no beneficial effect on the performance of hot mix and that cold mix with OPC offered better resistance to permanent deformation than the hot mix. Also, OPC acts as a secondary binder and the addition of OPC caused a reduction in fatigue life above 200 microstrains. They however, opined that a pavement structure might not in real life be subjected to as high a microstrain level, thus it could be said that OPC addition clearly extends the fatigue life of a pavement. Thanaya (2007) later confirmed this in his work (the developed relationship is as shown in Figure 3.7) though Ibrahim (1998) opined that there is no consensus for dealing with fatigue in cold mixes.

For example, Khalid (2003) observed that CBEMs continue to be underutilised in the UK in comparison with other European countries despite their significant environmental and economic potential. He opined that the main reasons behind this under-use are believed to be the high cost of emulsion binders and complexity involved in the design and performance assessment of CBEMs. More so, pavement engineers would always want to put in place structures that would be able to perform their intended design roles immediately after construction. Serfass et al (2003) submitted that cold mixes are evolutive materials (see Figure 3.8), especially in their early life and that their peculiar behaviour results from the

Table 3.10: Properties of the CBEMS Compared to Hot Mixes (Thanaya, 2007)

Mixture type	Compaction effort	Porosity (%)	ITSM (MPa)
CBEM	2 x heavy compaction	9.7	2275 (full curing)
CBEM + 1% cement*	2 x heavy compaction	9.4	3378(full curing)
CBEM + 2% cement*	2 x heavy compaction	9.2	4970(full curing)
100pen Hot Mix	Medium compaction	4.7	2150
100pen Hot Mix	Heavy compaction	3.4	2520
*by mass of aggregates			

Table 3.11a: The Beneficial Effect of Adding Cement to CBEMs (Thanaya, 2007)

No	Type of mix, with Nynas emulsion	ITSM after 1 month (MPa)	ITSM after 2 months (MPa)
1	WC 1-RPS without cement	752.37	816.04
2	WC 1-RPS + 2% OPC	1691.1	2084*
3	WC 1-RPS + 2% natural cement	1456.95	1691
4	WC 1-RPS + 2% rapid setting cement	1769.45	2258*
*Had achieved target			

Table 3.11b: Beneficial Effect of Adding Cement to CBEMs (Thanaya, 2007)

No	Type of mix	Porosity (%)	ITSM (MPa)
I	**CM with Nynas emulsion**		
I.1	CM without cement	9.2*	1595
I.2	CM + 2% OPC	8.7*	2581*
	CM + 2% rapid setting cement	8.5*	2593*
II	**CM with TotalfinaElf emulsion**		
II.1	CM without cement	8.2*	1346
II.2	CM + 2% OPC	7.8*	2327
	CM + 2% rapid setting cement	7.9*	2696
***Meets target**			

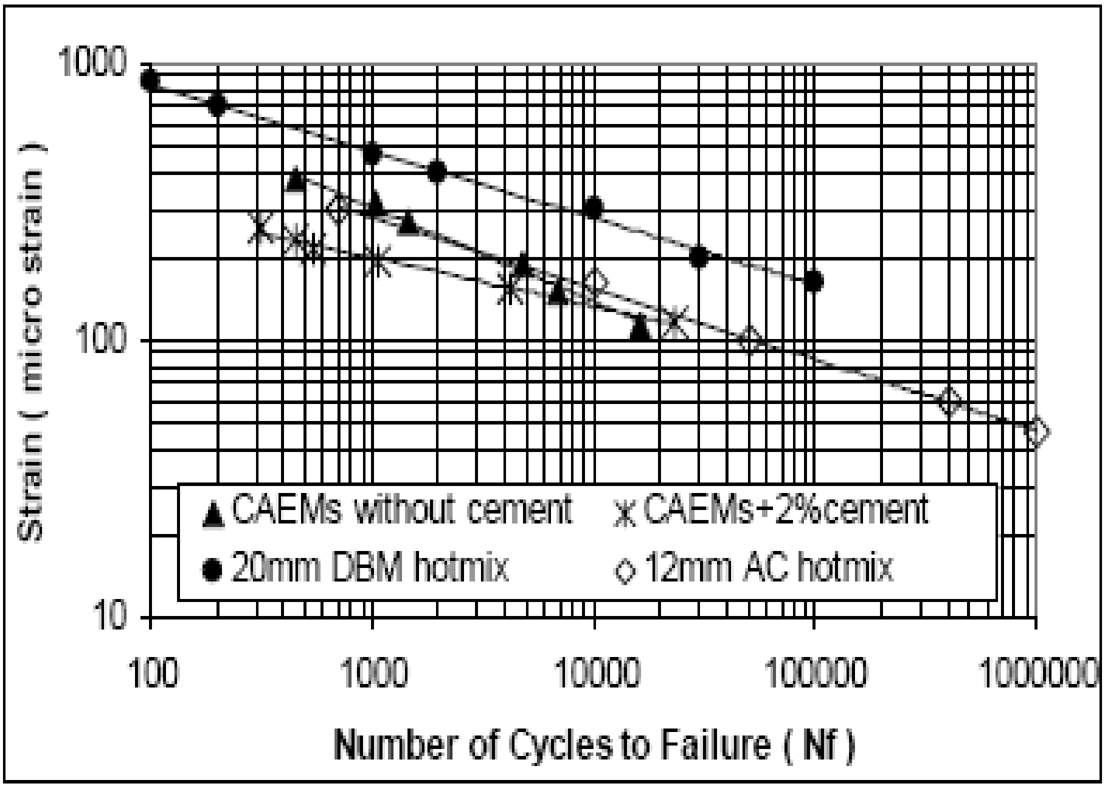


Figure 3.7: CBEMs Fatigue Performance Compared to Hot Mixes (Thanaya, 2007)

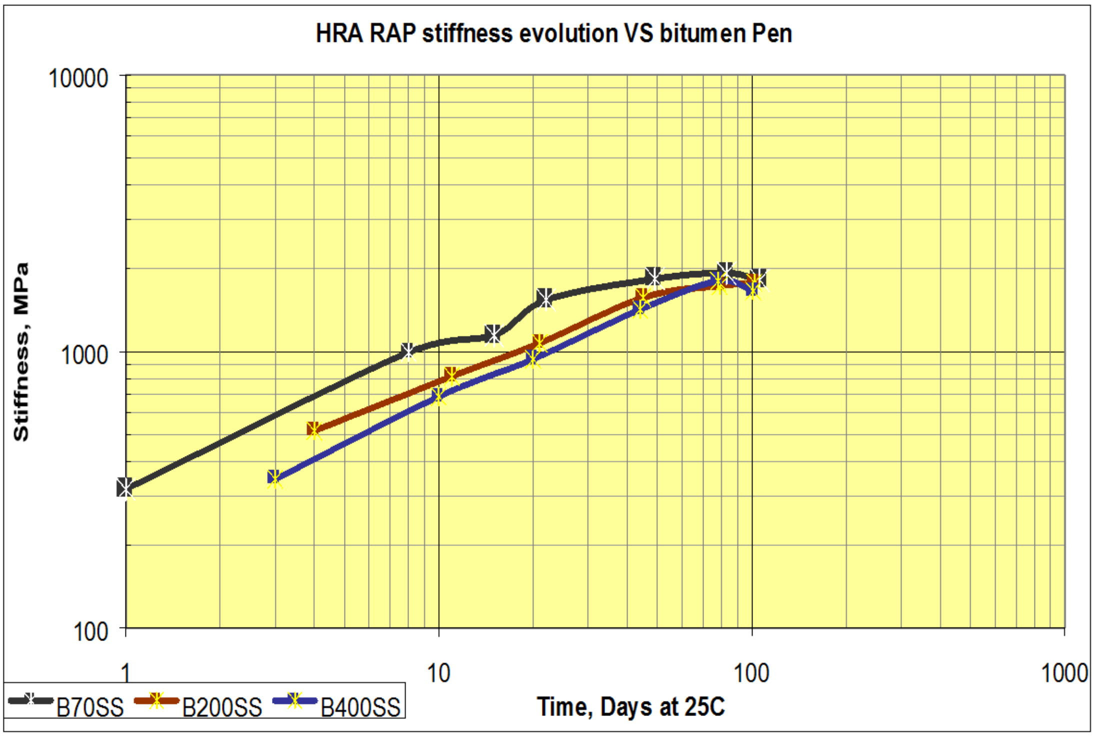


Figure 3.8: Influence of Bitumen Grade on Cold RAP Mix Stiffness versus Time Showing the Evolutive Behaviour of Cold Mixes (Lesueur et al, 2005; and Walter et al, 2008)

combination of several factors, presence of water, aggregate emulsion reactivity, binder film coalescence and cohesion build up. In the field, cold mixes reach their mature level of properties only after a period of time. In temperate climates and under medium traffic, at least one complete cycle of seasons is necessary for the mix to attain a stable condition. The curing time may be longer if the climate is cooler and more humid, or shorter if the traffic is lighter. This problem has been successfully addressed by applying hydraulic binders to the mix. However, when his option is used, care must be taken in the choice of hydraulic binder since they can significantly shorten the opening time for construction (depending on the type of cement used).

However, the major problems of cold mixes which would normally require the exercise of caution during the design and construction stages have been identified too. For example, Zoorob and Thanaya (2002) stated that besides being highly sensitive to ambient conditions during the mixture laying stage, the three most common problems encountered when using cold mixes for road layers are:

- High mixture porosity,
- Low early life strengths,
- Long curing times especially in cold and damp environments.

They indentified other problems such as poor aggregate coating, binder stripping and low binder film thickness which have now been overcome to some extent with rapid improvements in emulsion manufacturing technology and site production techniques.

Thanaya (2003) established that this binder coating problem is particularly peculiar to the coarser aggregates and that the performance of the mixtures is highly dependent on the availability of a superior emulsion formulation. Jove and Bock (2003) observed that the weak aspect of cold recycling is poor cohesion behaviour of the final mixture. Thanaya (2003) discovered that CBEMs can have some problems with binder drainage during storage as a result of low binder viscosity. Binder stripping can also occur due to weak adhesion and in general, the compacted mixtures contain high void contents.

On a similar note, Bocci et al (2002) established in their work that RAP particle size distribution has little influence on the compressive strength of cold recycled bituminous concrete. They also found out that the presence of lumps in RAP has a negative effect on the performance of the mixtures. Epps (1990) and Roberts et al (1984 & 1991) considered that cold mix recycling is only used to form a base course for low –to- medium traffic volume highways, because cold mixtures are not structurally as strong as hot mixtures and cold recycled mixes do not have adequate resistance to either abrasion by traffic or moisture induced damage.

Staple (1997) contrary to the experience of Robinson (Thanaya, 2003) was not favourably disposed to the use of emulsion due to the fact that cold emulsion mixes could not meet UK standards even after 18months of being laid as the stiffness moduli were far too low. He observed that cold emulsion macadam had significantly lower elastic modulus than that required. However nothing was stated about materials used and the factors that guided selection. He further opined in this work that the high air void content of the mix could have been responsible for the general low stiffness. Room is generally given for some degree of permeability and contact with air to allow for the curing or break of an emulsion. Staple (1997) was still doubtful about the applicability of cold mixes in road construction.

It may be observed from this review that there are a lot of challenges to overcome if cold mix, most especially cold recycled mix is to be fully integrated into road construction. Obviously, all of these observations have been made in the light of the performances of such mixes under temperate conditions. In simulating what obtains under a hot climate (temperature), one challenge is to determine which laboratory test would be the most appropriate for characterising the mechanical responses of recycled emulsion cold mixtures- since Ibrahim (1998) opined that such materials exhibit composite responses i.e. during the preparation of the mixes and laying, they behave as unbound granular materials, while after construction and having gained appreciable strength they start acting as bound materials. Other challenges in this regard are mentioned in the following section.

3.11 Interaction between Aged and Virgin Binders in Cold Recycled Mixes

While a lot has been done by researchers to establish the relationship between aged binders and recycling agents in hot recycled mixes, not much has been done on cold recycled mixes. Quite a number of researchers have established beyond reasonable doubt that in hot recycled mixtures, there is an interaction between aged binders and recycling agents (Lee et al, 1983; Noureldin and Wood, 1987; Soleymani et al, 2000; McDaniel et al, 2000; Karlsson and Isacsson, 2003; Schiavi et al, 2003; Romera et al, 2006; Artamendi and Khalid 2006; Chen et al, 2007; and Shen et al 2007).

On the other hand, FHWA (1997) stated that at ambient temperature, the softening effect of the recycling agent is a time and temperature dependent physico-chemical process and thus, the rate at which the reaction between the recycling agent and the aged asphalt cement (bitumen) occurs is a function of the properties of the recycling agent and the aged asphalt cement, and the mechanical effects of the physical processes such as mixing, compaction, traffic and climatic conditions. The relative contributions of the recycling agent and the aged asphalt binder are not fully understood at this time and thus, FHWA (1997) assumed that for cold recycled mixes, RAP only acts as black rock- which implies that the aged binder in the RAP does not interact with the recycling agent.

The only work among the reviewed reports which shed light on the interaction between aged binders in cold mixes and recycling agents was carried out very recently under the SCORE project in which Lesueur et al (2005) and Walter et al (2008), using the DSR (see Figure 3.9) and measurements of the mechanical properties of binders (aged and rejuvenating agents) and cold recycled mixtures respectively, established that ultimately there is an interaction between aged binders and the recycling agents. They studied the effect of new bitumen grade on complex modulus versus time and discovered that all samples seem to converge after 100days which indicate that, due largely to diffusion, the 400 and 200 Pen aged bitumen evolve toward a harder grade, between 70 and 100 penetration. However, the 70 penetration binder provided the highest modulus values both initially and for the next 100 days.

On the maturation time, they observed that cold diffusion is a slow process, the improvement between 1 week and 1 month was negligible which tends to indicate that most of the diffusion at ambient temperature takes place during the first 7 to 10 days. They however concluded that, DSR measurements are limited for practical reasons to temperatures above 40°C and reducing the number of concentration measurements is highly desirable given the large experimental times involved. They showed in their studies that it is possible to follow the diffusion phenomenon between new and old bitumen by measuring the evolution of stiffness of cold RAP mix versus time as the stiffness of a mix is directly related to the binder consistency and curing state of the emulsion binder. They also recommended the use of rejuvenating oil for RAP containing more than 4.5% residual bitumen and also hard grade bitumen emulsion when stiffness is the key performance criterion.

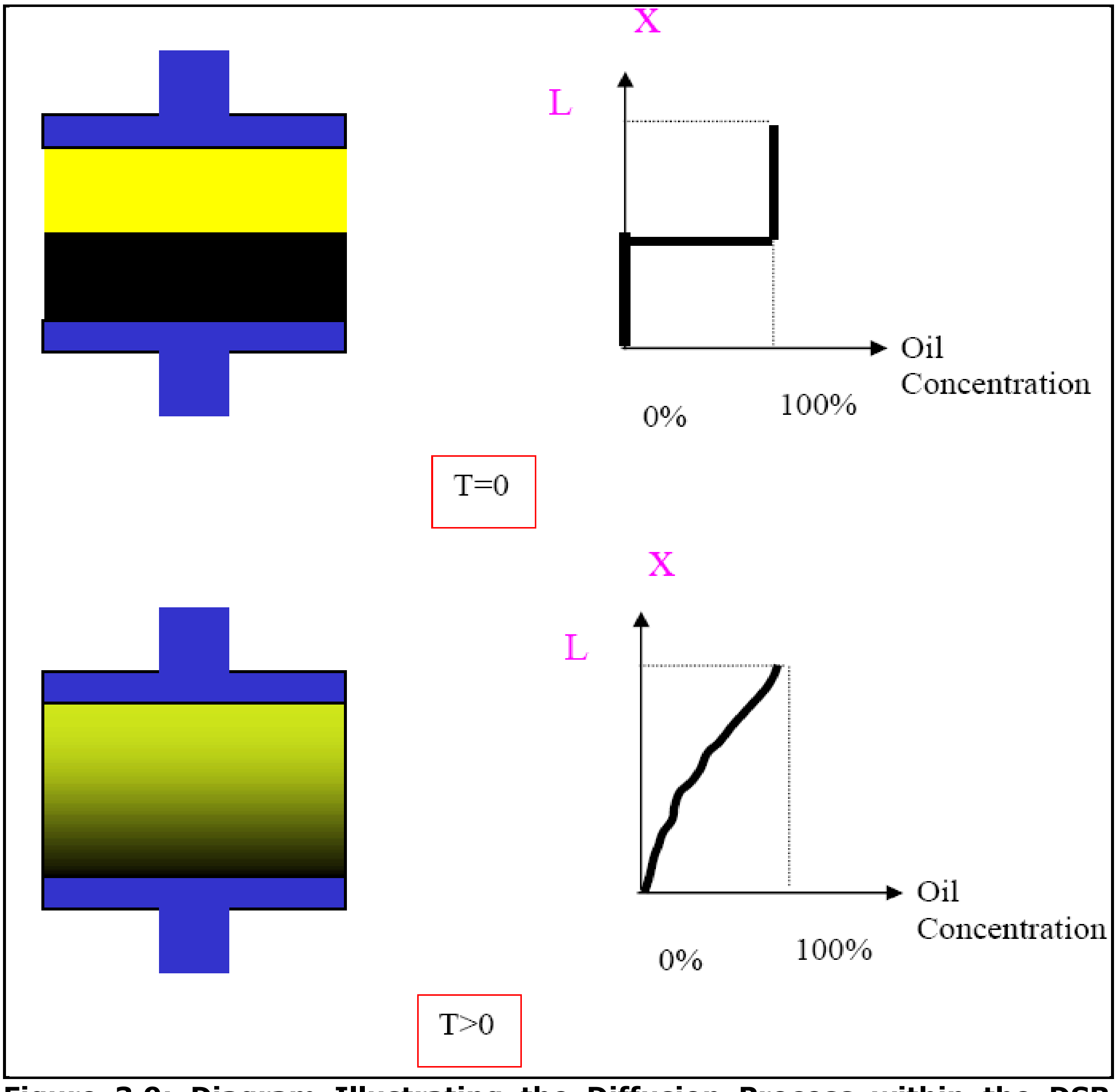


Figure 3.9: Diagram Illustrating the Diffusion Process within the DSR (Lesueur et al, 2005; and Walter et al, 2008)

Meanwhile, Karlsson and Isacsson (2003) opined that the major factors which facilitate an interaction between old and new binders in recycled mixtures are:

- mechanical mixing,
- compatibility (i.e. solubility parameter and molecular weight distributions) and
- diffusion

They further opined that the effectiveness of mechanical mixing is influenced by many factors i.e. temperature, binder viscosity, mixing time and type of mixture, and that compatibility between binders is a requirement for creating a homogeneous binder and is mainly dependent on the nature and distribution of intermolecular associations, which in turn determine the structural stability of binder. Also, they remarked that diffusion is a function of: temperature, intermolecular forces, structural rigidity of the diffusing molecules (restrictions on bending and twisting), microscopic structure of a relatively stationary phase if any, size and shape of the diffusing molecules or agglomerations as well as the viscosity of the medium in which the diffusion takes place. In conclusion, they submitted that the possibility of diffusion may not hold for low temperatures or severely aged binders and binders showing different behaviour under the influence of ageing.

In summary, it is obvious that more studies should be undertaken to ascertain the interaction between aged and virgin (rejuvenating) binders in cold recycled mixtures as information regarding this is still ambiguous. Though it has been established in the SCORE project that an interaction exists in such circumstances, the fact that the process involved is a time and temperature dependent physico-chemical one, and coupled with the fact that pavement engineers would normally desire that pavements start performing their roles efficiently shortly after being laid make it incumbent on researchers to put in place clear cut guidelines for the use of RAP in cold mixtures, which are based on a better understanding of the behaviour/interaction of the binders embedded in such cold recycled mixtures. For example, the degree of severity of ageing of binders in RAPs to be used in cold mixtures should be set in order to ultimately get good mixture performance almost immediately after construction.

3.12 Cold Mixes and Structural Design

Structural design is not completely new in cold mix design although there is no universally accepted method. This aspect of design is very important for determining (by optimisation) the layer thicknesses that should give the structural support required for the design life of a pavement. Thom (2009) suggested two general approaches for pavements incorporating cold mixture:

- treat it as hot-mix ,
- treat it as a very superior granular material.

Similarly, some methods which are basically empirical in approach have been advanced. These have been reported by Jitareekul (2009) as, the Catalogue Method, the CBR Design Method, the Asphalt Institute Method, AASHTO Method, and a UK method developed by Milton and Earland (1999).

In recent times also, the use of modelling tools involving multi layer linear elastic analysis which are commonly used for optimising layer thicknesses in pavements containing HMA have also been found useful for pavements which incorporate cold layers (Jitareekul, 2009; Ebels, 2008; and Twagira, 2010). These are implemented based on the results of performance tests conducted on mixtures. BISAR 3.0 and KENLAYER among other modelling tools have been chiefly used in this regard. Other methods which have been suggested are finite element modelling due to the complex nature of cold mixtures (Ebels, 2008; and Twagira, 2010), though this could be time consuming.

However, as much as such modelling tools are good for layer thickness optimisation, they must be executed with caution since cold mixtures have been found to be stress dependent i.e. non-linear in response (Ibrahim, 1998; Sunarjono, 2008; Ebels, 2008; and Twagira 2010) compared to HMA, whereas most such tools assume linear elastic behaviour.

3.13 Summary

The following are the conclusions from the review conducted in this chapter:

- Ageing of bitumen whether in the short or long term is inevitable in bituminous road pavements.
- Ageing in bituminous materials is evidenced by physical hardening which could either be advantageous or otherwise in a road pavement.
- Laboratory protocols (accelerated) for simulating ageing of bituminous road pavements in temperate climates have been developed. Such protocols are useful for predicting performance in service of bituminous road pavements.

- There is a need to develop similar laboratory protocols for road pavements in hot tropical climates.
- Cold mixes are cost effective, energy efficient and environmentally friendly, although inhibiting factors such as low early life stiffness and strength development, high air void contents and presence of moisture in the mix are still preventing such mixtures from being fully embraced in some developed countries.
- Most of the studies reported in literature on cold mixes have focused on temperate climates, though with good results in most cases.
- There is lack of evidence in literature of similar studies conducted in hot tropical climates and therefore there is a need to urgently look at this area.
- There is no universally accepted method for cold mix designs (job formula) although most of the methods which have been advanced in literature are based on the Hveem and Marshall methods of mix design.
- Adequate mixing and compaction alike are important for cold mixes.
- Curing is required for cold mixes for maturation and development of stiffness. The use of OPC in such mixes aids evaporation of moisture and achieves quicker stiffness and strength development.
- Stiffness, fatigue response, deformation properties, resilient modulus, and water susceptibility are good means for assessing the performance of cold mixes.
- Knowledge about the interaction between virgin binders and aged residual binders in cold recycled mixes is still ambiguous and needs further exploration. RAPs are still regarded as black rocks when they are used in cold recycling.
- The effect of preparation temperature (mixing and compacting) of cold mixes has not been fully accounted for in literature.
- Structural design of pavements involving cold mixtures should consider the non linear behaviour of such mixtures for good results.

4 Laboratory Ageing Protocol for Producing Severely Aged RAPs

4.1 Overview

This chapter describes the procedure followed in producing the reclaimed asphalt pavements (RAPs) that were used in this research work. This was embarked upon with the aim of producing RAP materials in the laboratory with controlled properties, typical of those found in hot tropical belts (with residual binders of very low penetration). Evidence from the literature as reported in Chapter 3 of this thesis indicates that existing standard protocols for ageing bitumen and bituminous mixtures essentially mimic ageing in road asphaltic pavements in developed countries where their climates are mainly temperate. Asphaltic pavement conditions in such countries are rarely left to deteriorate to the level that was investigated in this work and thus makes the existing laboratory ageing standard unsuitable for this exercise. The produced RAPs were subsequently used for laboratory studies on cold recycling described in subsequent chapters of this thesis.

4.2 Why Laboratory Aged RAPs and not RAPs from the Industry?

In developing guidelines for the use of cold recycled asphalt mixes in hot tropical countries (with Nigeria being the case study) the first challenge is sourcing the recyclate. For this investigation, being conducted in the UK, three options were available, i.e. (1) Sourcing RAP from local construction industry, (2) Placing an order for RAP materials from Nigeria and (3) Making artificial RAP in the laboratory. The first option comes with some variables which could give misleading results in this research work. In the UK for example, road developers favour the use of limestone as aggregates for bituminous binder course mixtures due to the fact that the material is abundantly available and with the advantage of being good at resisting stripping when used in bituminous mixtures. On the contrary, in Nigeria, which is the case study in this work, granite is normally used for the production of bituminous mixtures since they are abundantly available.

Another fundamental issue concerns the severity of ageing of the RAP. It is not common in the UK to allow pavements to deteriorate to levels such that penetration values of recovered binders from RAPs would be as low as 5dmm. In fact, RAPs with mean penetration values less than 15dmm are not deemed suitable for use in recycling (BS EN 13108-8:2005) or at best classified and treated as black rock. Meanwhile, Nnanna (2003) observed that some roads constructed about 32 years ago in Nigeria have not yet been rehabilitated. Such long neglect must be partly responsible for the findings of Smith and Edward (2001) in which they reported penetration values as low as 6dmm for wearing course and 5dmm for binder course in some asphalt pavements in East Africa. Thus the suitable materials for this research work might not be sufficiently available in the UK. Similarly, the use of modified bituminous binders in the UK is a major factor that could introduce significant variations to the findings of this work if used. It might be a difficult task differentiating RAPs with modified binders from those that are not if they are to be used for this research work. Straight run bitumen is most frequently used as the binding medium for bituminous mixtures in Nigeria. Concerning the second option identified for sourcing RAP materials, the huge costs and delays that may be involved in the logistics of getting such materials to the UK both make it unappealing and thus it was abandoned.

The third option which is, producing the RAPs in the laboratory through ageing of virgin bituminous mixtures seems to be the most practical. Although laborious, this will obviously allow for an exercise of control over the quality and condition of the RAPs produced. As earlier observed in Chapter 3, standard laboratory protocols for long term ageing of asphalt mixtures have been developed during the Link BITUTEST and SHRP programmes in the UK and USA respectively (Bell, 1989; and Scholz, 1995). Long term laboratory ageing under these protocols is conducted at 85ºC over 5days, and the assumption is that this is representative of 15 years or more of ageing of pavements in service in temperate regions of the world (Bell 1989; and Scholz, 1995).

However, these standard laboratory ageing protocols use compacted specimens (Airey, 2003) and would require long laboratory ageing times in order to produce the desired properties. This is understandable as such protocols were not designed for the harsh conditions considered in this work. Ageing loose samples at higher temperatures (105ºC and 125ºC) than standard will generate more homogeneous (de la Roche et al, 2009) severely aged RAP in a practical time but risks unduly degrading the binder in the mix. Researchers have suggested that this degradation would normally affect the physico-chemical and rheological properties of the binder in the mix (Airey, 2003 and Wu et al, 2007).

It was considered important to conduct this investigation and come up with an ageing protocol that is as quick as possible without compromising the integrity of the residual binder or introducing any significant changes to its properties when compared with a standard ageing protocol.

4.3 Experimental Programme for Producing and Investigating Severely Aged RAPs

The investigation required verifying and comparing the chemical composition (asphaltene content in line with BS 2000: PART 143:2004), viscoelastic response (rheology in line with BS EN14770:2000) and temperature susceptibility (penetration and softening point in line with BS EN1426:2000 and BS EN1426:2000 respectively) of recovered binders (in line with BS EN12697-4:200) from mixtures immediately after; (1) mixing and compaction (representing short-term ageing), and (2) subsequent ageing at 85ºC, 105ºC and 125ºC (representing long-term ageing) over 24, 48, 72, 96, 120, 240, 480 and 840hrs in the forced draft oven. The flow chart in Figure 4.1 was followed in carrying out the exercise and Figure 4.2 (Start –A/End -B) illustrates the exercise in pictures as it was progressively carried out in the laboratory during mass production of the RAPs that subsequently followed the investigation reported in this chapter.

Just before being subjected to the long term ageing, the compacted mixtures were loosened in order to ensure homogeneous ageing. It was easy to do this at the preliminary stage of investigation, as they were all produced using the gyratory compactor. It is worth noting that the roller compactor and Kango Hammer were subsequently used thereafter for compacting and breaking compacted slabs (slabs of 305mm x 305mm x 50mm) at the level of large scale production of RAPs that followed. While at the initial stage it was practical to manually loosen compacted asphalt specimens prior to ageing in the forced draft oven because few kilograms of the materials were involved, the need to crush specimens using the Jaw Crusher (at a predetermined crusher opening that suits target gradation) prior to ageing at the later stage was inevitable as hundreds of kilograms of materials were processed.

These materials were placed in oven trays such that the thickness of the loose materials in the tray was not more than 40mm. This allowed for homogeneous ageing. As ageing progressed in the forced draft oven irrespective of the ageing temperature used in this work, the bituminous binder melted and once again bound (although slightly) most of the aggregates together forming lumps. This was not a major issue for the binder recovery tests, but it became obvious at the

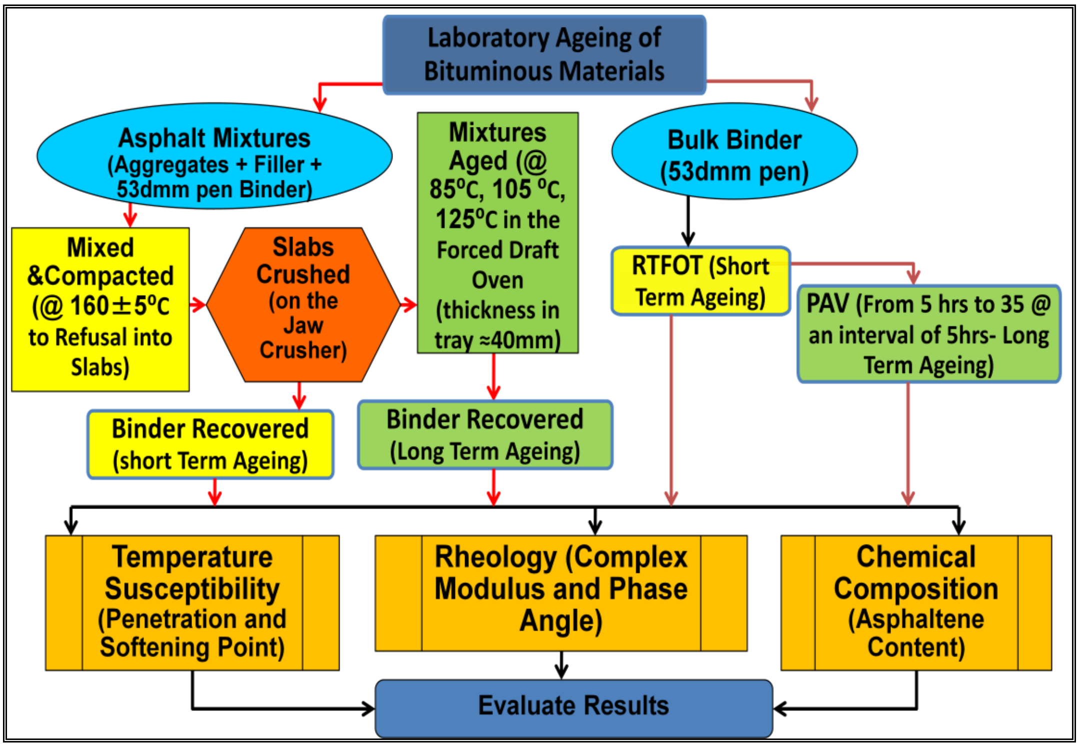


Figure 4.1: Flow Chart Showing Experimental Programme for Severely Aged RAP Production and Investigation

Figure 4.2: Laboratory Procedure for the Production of Severely Aged RAP

mass production level that, running them through the Jaw Crusher again was unavoidable. Apart from facilitating the production of a more homogeneous final product (Artificial RAP), using the Jaw Crusher grossly reduced handling time of the materials. Figure 4.3 shows the various stages involved and the condition of HMA in its progressive transformation to RAP. Plate A in the figure is the initial condition while Plate F is the final product – Artificial RAP. The yellow rings on the Plate E show the lumpy nature of the artificial RAP material immediately after ageing.

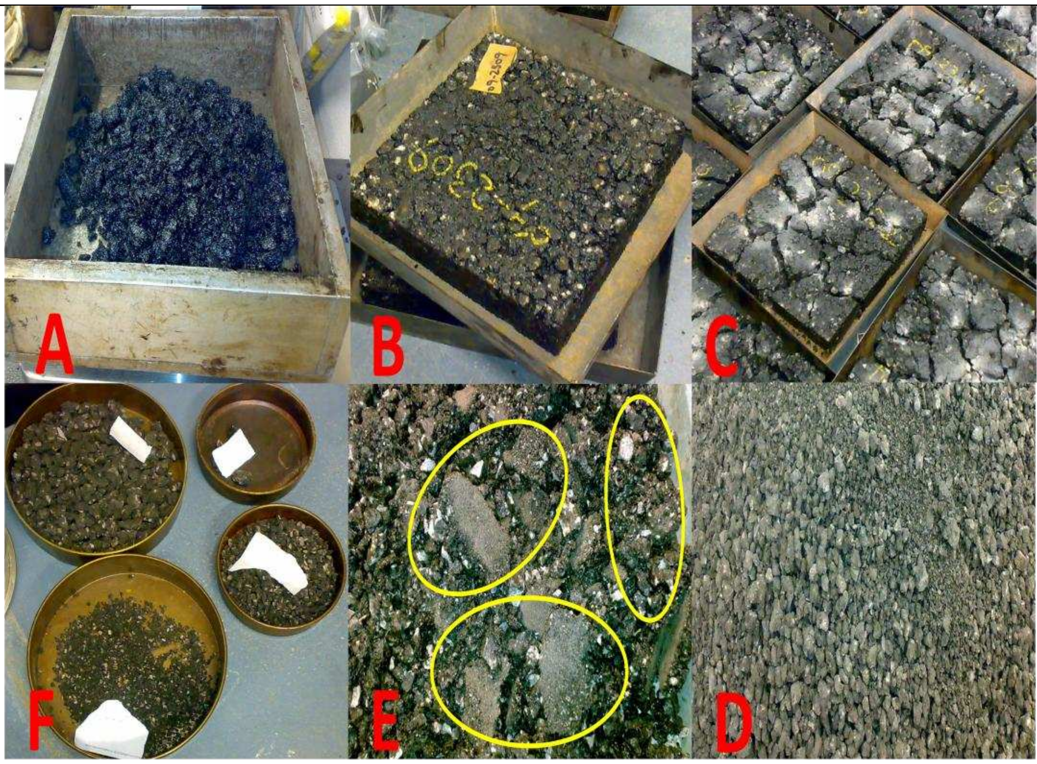
Meanwhile, to allow for direct comparison as ageing progresses, bulk binder was also aged in the Rolling Thin Film Oven (RTFO) and the Pressure Ageing Vessel (PAV) in line with BS EN12607-1:2007 and BS EN14769:2005 respectively. The RTFOT simulates short term ageing while the PAV test simulates long term ageing. Residues from the RTFOT test (conducted at 163ºC for 75minutes) were subsequently subjected to ageing over periods of 5, 10, 15, 20, 25, 30 and 35hrs in the PAV. The residues were aged in the PAV at 100ºC as this simulates what occurs in the tropics (Bahia and Anderson, 1995). The applied pressure used was 2.1MPa. For rheology, all the binders were tested within the linear viscoelastic range on the Bohlin Gemini Dynamic Shear Rheometer applying the following testing conditions:

- Mode of loading: Strain Controlled
- Temperatures: 10 to 80ºC
- Frequencies: 0.1 to 10Hz
- Plate geometries: 8mm diameter with a 2mm gap (10 to 45ºC) and 25 mm diameter with a 1mm gap (25 to 80ºC)

Figure 4.4 shows facilities for bulk binder ageing (i.e. the RTFO and the PAV), while Figure 4.5 details the binder recovery process, and physico-chemical/rheology testing facilities available in NTEC respectively. It is worth noting that Figures 4.2 and 4.3 in particular mimic all the processes involved in asphalt pavement construction, from mixing time to the point of reclamation for recycling.

4.4 Materials for the Investigation

Granite - This was sourced from Montsorrel Quarry, Loughborough, Leicestershire and chosen because granite is predominantly used for bituminous pavements in Nigeria. Gradation was in line with the



A – Coated Aggregates placed in mould and ready for compaction, B – Compacted HMA Slab (305 x 305 x 40)mm
C – Broken HMA Slab, D – Unaged Crushed Condition after 1st run on the Jaw, E – Aged Crushed Condition (Lumpy)
F – Artificial RAP in Final Condition after a 2nd run on the Jaw Crusher

Figure 4.3: Stages and Condition of HMA in its Progressive Transformation to RAP

Rolling Thin Film Oven (RTFO) Pressure Ageing vessel (PAV)

Figure 4.4: Facilities for Bulk Binder Ageing

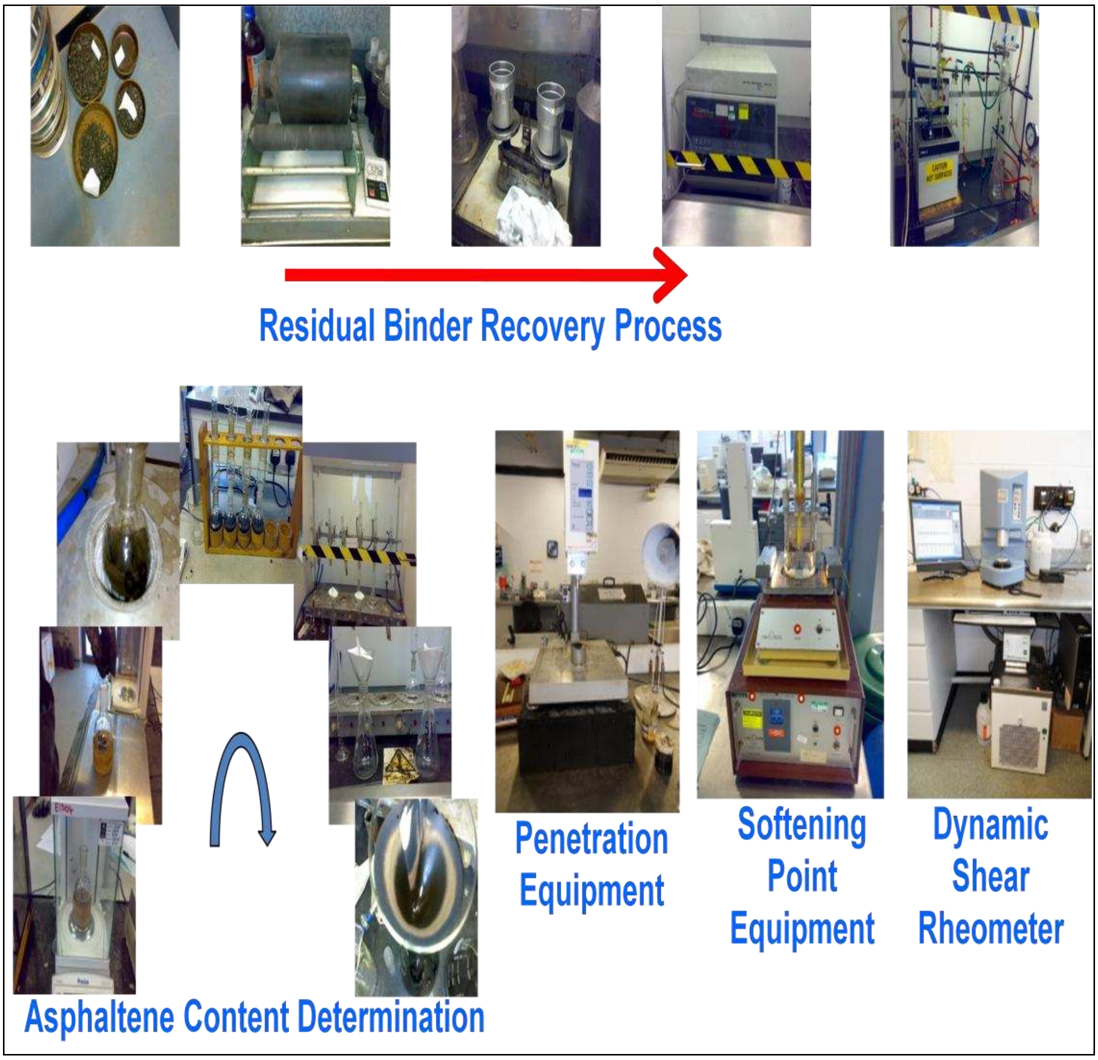


Figure 4.5: Binder Recovery and Physico-chemical/Rheology Testing Facilities

requirements of ORN 19 and 31 (TRL and DFID, 2002 and TRRL, 1993) for heavily trafficked roads in the tropics.

Bitumen - This was supplied by Nynas Bitumen and it was a straight run paving grade bituminous binder 40/60 (with measured 53dmm pen and 51ºC softening point) of Venezuelan origin.

HMA - Prepared in the laboratory (BS EN12697-35:2004). The mix was compacted to refusal in line with the requirements of ORN 19 and 31 for heavily trafficked roads in the tropics. Figure 4.6 details the gradation of the granite aggregate used for the production of the HMA. The gyratory compactor was used for compaction of the samples at this initial stage and the specification for production of samples that was used is:

- Number of Samples: 36
- Sample Size: 100mm diameter 63.5mm thickness
- Mix Type: DBM (Wearing Course)
- Binder Content: Will be a function of Voids in Mineral Aggregate (VMA) (4.6% but could be increased to around 5% to achieve a better coating)
- Aggregate Type: Granite
- Aggregate Gradation: In line with ORN 19 design for a 14mm close graded surface (see Figure 4.6)
- Compaction Method: Gyratory
- Angle of Gyration and Compaction Stress: 1deg and 600KPa
- Desired Density: Refusal at about 3-5% Voids in Mix (VIM)

4.5 Physico-Chemical and Rheological Properties of Recovered Binders from Aged Mixtures and Bulk Binders

The tests carried out on the studied bituminous binders (virgin and recovered alike) focused on the determination of their physical properties which include penetration and softening point. Also tests were conducted in the Dynamic Shear Rheometer (DSR) in the oscillation mode of testing, essentially to determine the viscoelastic response of the binders. The asphaltene contents of all the binders were also determined in order to ascertain their chemical compositions.

4.5.1 Determination of penetration

This test basically measures the consistency of bitumen (Read and Whiteoak, 2003). The test is such that a needle of specified dimensions is allowed to penetrate a sample of bitumen under a known load which normally is 100g, at a fixed temperature of 25ºC, for a known time of 5s (see Figure 4.7). Penetration is

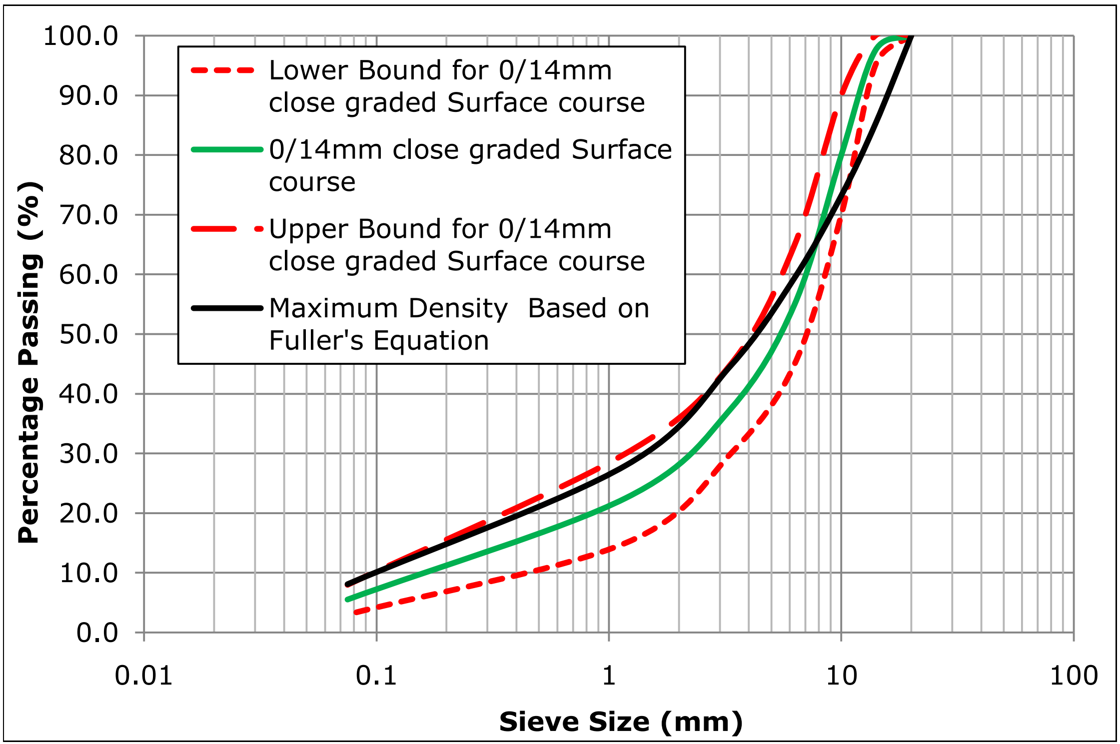


Figure 4.6: Gradation of the Granite used for the Production of HMA

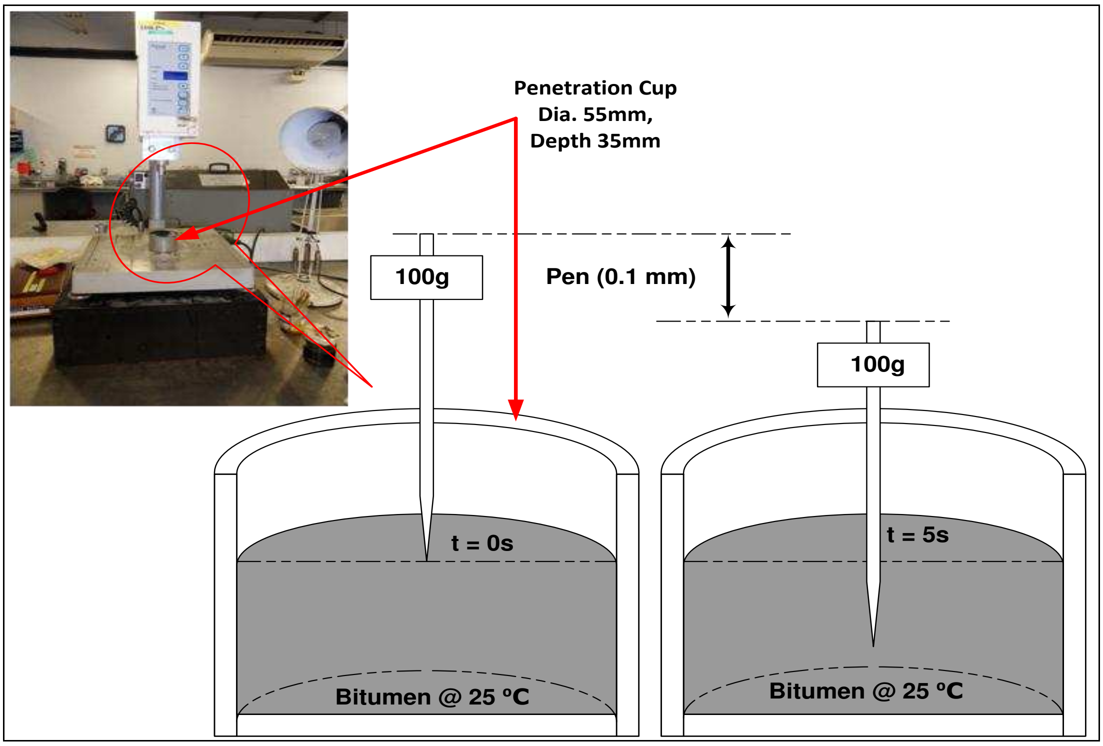


Figure 4.7: The Penetration Test

defined as the distance travelled by the needle into the bitumen, and it is measured in tenths of a millimetre, i.e. decimillimetre- dmm (Read and Whiteoak, 2003). A low value indicates that the bitumen is hard, thus the lower the value, the harder the bitumen and vice versa. The standard that was used as the guide for the conduct of the tests is BS EN1426:2000. Twenty six bituminous binders were tested, one of which was conducted on the virgin binder and the remaining twenty five on recovered binders. The tests were conducted three times and the values obtained were averaged. These averaged values were taken as the penetration of each binder and the values ranged from 6 to 53dmm. The details of the results obtained for penetration of the recovered bituminous binders from mixtures and aged bulk bituminous binder are detailed in Tables 4.1 and 4.2 respectively. As envisaged, and in line with the findings of previous researchers, the highest value obtained was for the virgin binder.

4.5.2 Determination of softening point (SP)

This is another empirical test normally conducted to measure the consistency of bitumens. In this test, a steel ball of 3.5g in weight is placed on a sample of bitumen contained in a brass ring that is then suspended in a water or glycerine bath (Read and Whiteoak, 2003). Figure 4.5 shows the softening point testing apparatus that was used in this work. Read and Whiteoak (2003) stated that water is used for bitumen with a softening point of 80ºC or below and glycerine is used for softening points greater than 80ºC. The bath temperature is raised at 5ºC per minute, the bitumen softens and eventually deforms slowly with the ball through the ring. At the moment the bitumen and steel ball touch the base plate 25mm below the ring, the temperature of the water (glycerine) is recorded as the softening point of the binder. Although there are some other guides for conducting this test, the standard that was used in this work was BS EN1427:2000. In all, twenty six bituminous binders were tested, one of which was done on a virgin binder and the remaining twenty five on recovered binders.

The test was performed twice and then the values obtained were averaged and recorded as the softening point. BS EN1427:2000 requires that if the difference between the two results obtained exceeds 1.0ºC, the test is repeated. The averaged values obtained as softening points (SP) during the exercise ranged from 51.0 to 123.0ºC. The lowest value obtained was for the virgin binder. A low value indicates a soft binder while high values suggest it as hard. Read and Whiteoak (2003) further stated that values obtained for softening points are also regarded as equi-viscous temperatures. Values for the softening point of the binders tested during this exercise are detailed in Tables 4.1 and 4.2.

Table 4.1: Physical Properties, Chemical Composition and Mechanical Properties of Virgin and Recovered Binders at Different Ageing Temperatures and Time

Unaged and After Mixing							
Time of Ageing (hrs)	**Physical Properties**			**Chemical Composition**		**Rheology (@0.4Hz &25ºC)**	
	Pen (dmm)	SP (ºC)	PI	Asphaltene Content (%)	Maltene Content (%)	G* (MPa)	δ (º)
Unaged	53	51	-0.81	12.5	87.5	0.39	69.9
After Mixing	40	59	0.29	15.6	84.4	0.95	61.7
Ageing at 85ºC							
Time of Ageing (hrs)	**Physical Properties**			**Chemical Composition**		**Rheology (@0.4Hz &25ºC)**	
	Pen (dmm)	SP (ºC)	PI	Asphaltene Content (%)	Maltene Content (%)	G* (MPa)	δ (º)
24	31	60.0	-0.06	14.9	85.1	0.96	60.5
48	27	64.0	0.39	16.3	83.7	1.59	56.8
120	23	68.0	0.74	19.0	81.0	2.33	52.7
240	20	69.6	0.73	19.7	80.3	3.61	48.7
480	15	76.4	1.19	22.1	77.9	5.33	42.6
840	11	85.5	1.82	24.4	75.6	9.05	35.3
Ageing at 105ºC							
Time of Ageing (hrs)	**Physical Properties**			**Chemical Composition**		**Rheology (@0.4Hz &25ºC)**	
	Pen (dmm)	SP (ºC)	PI	Asphaltene Content (%)	Maltene Content (%)	G* (MPa)	δ (º)
24	23	67.5	0.66	17.3	82.7	2.24	52.6
48	18	71.8	0.87	20.0	80.0	2.96	50.1
120	14	78.5	1.35	22.3	77.7	5.57	42.7
240	9	91.0	2.11	29.9	70.1	12.40	32.6
480	6	123.0	4.40	36.2	63.8	19.70	24.6
Ageing at 125ºC							
Time of Ageing (hrs)	**Physical Properties**			**Chemical Composition**		**Rheology (@0.4Hz &25ºC)**	
	Pen (dmm)	SP (ºC)	PI	Asphaltene Content (%)	Maltene Content (%)	G* (MPa)	δ (º)
24	20	73.8	1.30	21.6	78.4	3.49	46.6
48	11	88.0	2.12	26.3	73.7	7.82	37.7
72	7	102.0	2.84	31.8	68.2	11.0	30.5
96	6	117.5	3.96	33.5	66.5	16.60	23.3

Note: Pen = Penetration, SP = Softening Point, PI = Penetration Index, G= Complex Modulus, and δ = Phase Angle*

Table 4.2: Physico-Chemical and Rheological Properties of Virgin, RTFO and PAV Aged Binders

Condition of Binder	Physical Properties		Chemical Composition		Rheology (@0.4Hz &25°C)	
	Pen (dmm)	SP (°C)	Asphaltene Content (%)	Maltene Content (%)	G* (MPa)	δ (°)
Original	53	51.0	12.5	87.5	0.39	69.9
RTFOT	34	57.0	15.1	84.9	0.99	62.8
PAV 5Hrs	25	61.0	17.1	82.9	1.58	58.2
PAV10Hrs	24	62.4	17.6	82.4	1.93	56.6
PAV 15Hrs	20	64.6	18.7	81.3	2.27	53.9
PAV 20Hrs	21	65.2	17.5	82.5	2.30	53.7
PAV 25Hrs	23	68.0	19.0	81.0	2.95	50.8
PAV 30Hrs	18	67.8	19.9	80.1	3.25	49.6
PAV 35Hrs	17	69.2	23.5	76.5	3.70	48.7

Note: Pen = Penetration, SP = Softening Point, G= Complex Modulus, and δ = Phase Angle*

The values obtained for the penetration and softening point of the bituminous binders were used in determining the temperature susceptibility i.e. penetration index (PI), of the tested binders. Saleh (2006), defined temperature susceptibility as a measure of the way the binder's consistency changes with a change in temperature. Values below -2 indicate high temperature susceptibility. The values obtained for the PI are also in Tables 4.1 and 4.2. The values were calculated after Read and Whiteoak (2003) from:

$$\mathbf{PI} = \frac{\mathbf{1952 - 500logPen - 20SP}}{\mathbf{50logPen - SP - 120}} \qquad 4.1$$

(Where Pen and SP are the penetration and the corresponding softening point values of the binder).

4.5.3 Chemical composition of recovered binders

The test was carried out with the aim of determining the chemical composition of the binders and was conducted in line with BS 2000 Part 143:2004, in which the asphaltene contents were precipitated using heptanes (C_7H_{16}). This was done by first dissolving each bitumen sample in hot toluene (methylbenzene- C_7H_8 ($C_6H_5CH_3$)) in order to get rid of the inorganic and the waxy components of the binder.

Pure bitumen dissolves completely in toluene while the inorganic and waxy components come out as precipitates. The precipitates were filtered off using a filter paper. Thereafter, the bitumen was recovered in the fractionating column from the toluene solution. The recovered pure bitumen was subsequently dissolved in hot heptane. Maltenes dissolve completely in heptane, while asphaltenes are precipitated in the process. The solution was filtered and the precipitates - asphaltenes were subsequently recovered. The asphaltenes are normally expressed in percentage i.e. asphaltene contents percentage (%) of the bitumen sample. Two tests were conducted on each bitumen sample and the results were averaged and subsequently recorded as the asphaltene content. Tables 4.1 and 4.2 detail the chemical composition of the binders i.e. asphaltene and maltene contents while Figure 4.4 captures some of the processes involved in the determination thereof. The results obtained are consistent with those of other researchers, as the asphaltene contents increased, the degree of hardness of the recovered binders increased. The asphaltene contents of the virgin binder and the recovered binder after mixing are 12.45% and 15.55% respectively. These results and others are discussed later in this chapter.

4.5.4 Tests on the Dynamic Shear Rheometer (DSR)

The Dynamic Shear Rheometer (DSR) is used for measuring the dynamic viscoelastic properties of bituminous binders among other things (Papagiannakis and Masad, 2008). Rahman (2004) stated that the DSR is capable of describing the linear viscoelastic responses of binders (see Figure 4.8) over a range of temperatures and frequencies. The DSR used in this work is shown in both Figures 4.5 and 4.9, and the standard used for the conduct of the tests was BS EN 14770:2005**.** The DSR applies sinusoidal shear strain to a sample of bitumen sandwiched between two parallel plates as shown in Figure 4.9. The plates used were 8mm (for temperatures between 10ºC and 45ºC and binder thickness of 2mm) and 25mm (for temperatures between 25ºC and 80ºC with a binder thickness of 1mm) in diameter. Rahman (2004) stated that the amplitude of the stress is measured by determining the torque transmitted through the sample in response to the applied strain. The DSR only records two measurements i.e. torque (T) and angular rotation (θ), while the remaining mechanical properties are calculated using these two measurements. The relevant equations and the testing geometry are as shown also in Figure 4.9. Meanwhile 'r' in the equations stands for the radius of the parallel plates used. The mechanical properties of interest in this work are the complex shear modulus (G*) and the phase angle (δ) of the binders. While G* is determined as indicated in Figure 4.9, δ is measured automatically by the instrument by accurately determining the sinusoidal waveforms of the strain and torque as indicated in Figure 4.9.

Before conducting any test on the bitumen, the linear viscoelastic range is first determined by conducting an amplitude sweep. This linear range is defined from the point G* (maximum point) to 0.95G* as indicated in Figure 4.8. The subsequent frequency sweep and single frequency tests are conducted using the target strain obtained from the plots of the linear viscoelastic range. The essence of this is to ensure that all tests were conducted in the linear viscoelastic range of each binder. Obtaining the values of the complex shear modulus (G*) and the phase angle (δ) of the binders tested are now easy as they are automatically recorded through the software specifically written for processing the raw data obtained during the test in the DSR. Three tests; amplitude sweep (at 1Hz and temperatures 20ºC and 70ºC for the 8mm and 25mm plates respectively), single frequency tests (at 0.4Hz and temperature 25ºC for both the 8mm and 25mm plates) and frequency sweeps (at a frequency range of 0.1Hz to 10Hz, temperature ranges of 10ºC to 45ºC and 25ºC to 80ºC for both the 8mm and 25mm plates respectively) were conducted. In all one hundred and fifty six (156) tests were conducted on the 26 binders investigated in this exercise. The results

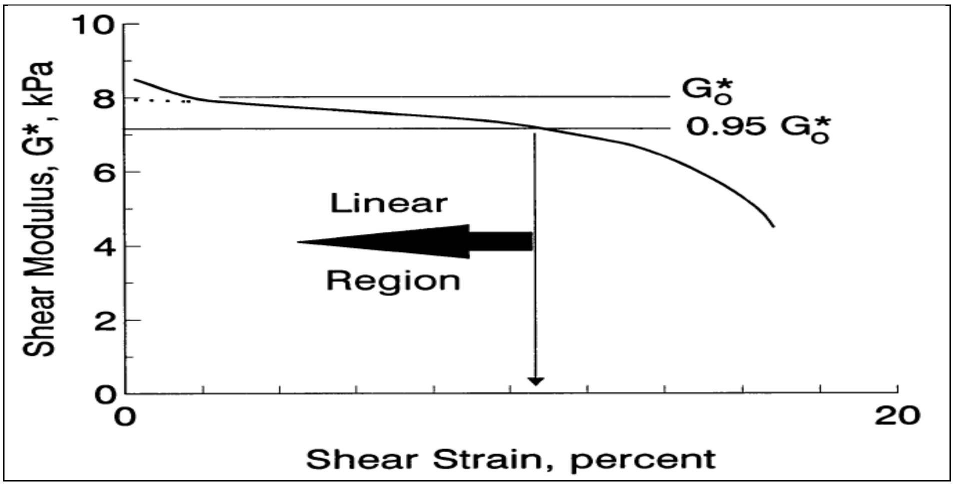
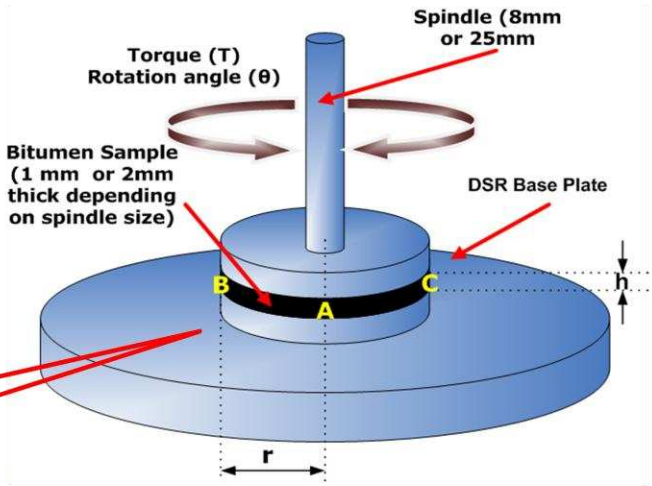


Figure 4.8: Determining the Linear Viscoelastic Range of Binders (Petersen et al, 1994)



$$\tau = \frac{2T}{\pi r^3}, \gamma = \frac{\theta r}{h}, G^* = \frac{\tau_{max} - \tau_{min}}{\gamma_{max} - \gamma_{min}}$$

Where
γ = shear strain (%),
δ = phase angle (°)
θ = deflection angle,
τ = shear stress (Pa)
ω = angular frequency,
G* =complex modulus
T = torque,
r = radius of plate,
h = thickness of bitumen specimen

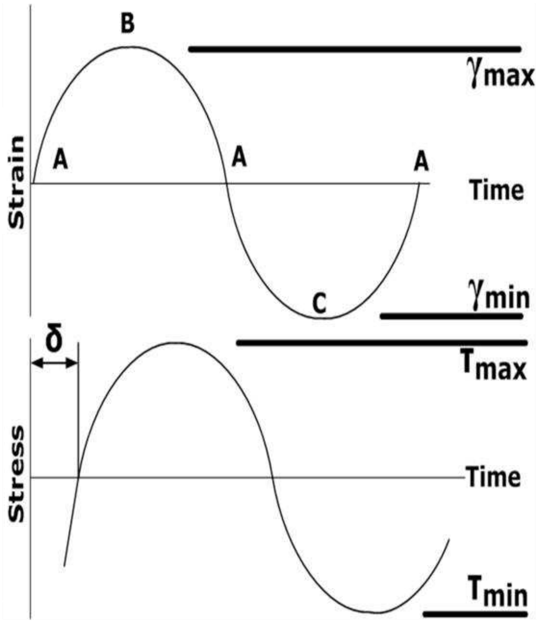


Figure 4.9: The Dynamic Shear Rheometer and its Geometry

obtained (based on the single frequency tests with the earlier mentioned testing configurations) for the complex shear modulus (G*) and the phase angle (δ) for each of the binders are as stated in Tables 4.1 and 4.2 shown earlier.

4.6 Discussion and Analysis of Results

As mentioned in section 4.5 of this chapter, the integrity of the recovered binders was examined through the determination of their softening points (BS EN 1427:2000), penetration (BS EN 1426:2000) (data obtained from both the softening point and penetration tests were used in studying the temperature susceptibility of the recovered binders), asphaltene contents (BS 2000:Part 143:2004) determination (for chemical composition of the binders) and the determination of the viscoelastic response (BS EN 14770:2005) using results of the measured complex modulus and phase angle obtained in the DSR. Tables 4.1 and 4.2 shown earlier detail the values obtained for the physical properties (penetration, softening point and penetration index), the chemical composition (asphaltene and maltene contents) and the mechanical properties (complex shear modulus and phase angle) of the recovered binders from aged mixtures and as well as aged bulk binders. All these were done in order to determine an effective and efficient protocol for ageing bituminous mixtures in the laboratory without significantly compromising the integrity of the binder in the bituminous mixture.

Meanwhile, the penetration of the binders has been used as the basis for analysing the results, since some of the recovered binders under the three ageing protocols i.e. 85ºC, 105ºC and 125ºC gave similar penetration values. Ageing of bituminous mixtures at 85ºC is used as the reference (control) being the standard practice. In all, four sets of similar penetration values occurred i.e. differently aged binders with the same penetration values. Based on this, some relationships were generated using the physical, chemical and mechanical properties of such recovered binders to ascertain whether the integrity of the binders had been compromised due to different ageing temperatures. The analysis was done carefully taking into consideration the time required for the aged bituminous mixtures to reach the desired severely aged level.

4.6.1 Penetration vs. ageing time

Figure 4.10 shows a plot of penetration against the time taken for ageing the bituminous mixtures to the desired levels. After several trials a model that fits the data was selected, and the model is indicated on the chart. It can be seen that ageing at 85ºC is only likely to produce sufficiently aged binder (initial target

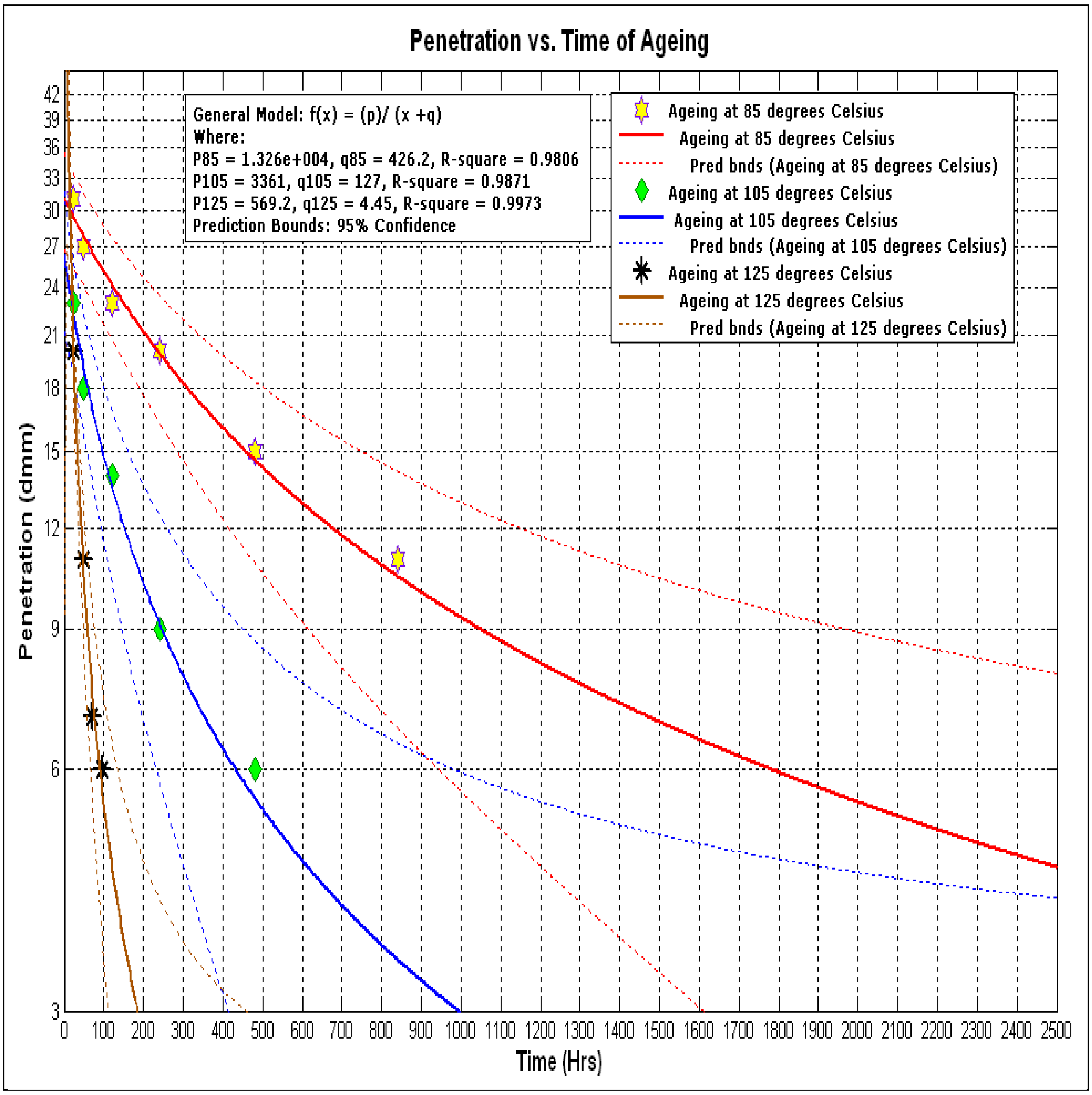


Figure 4.10: Penetration of Recovered Aged Binders vs. Time of Ageing

penetration of 3dmm was chosen and was later changed to 5dmm which was much more practical in the laboratory) after very long ageing times, if at all and is not a suitable ageing protocol for producing the desired RAPs. At 105ºC the fitted curve indicates that such severe ageing could probably be attained after long ageing times (about 1000hours) but the upper 95% confidence interval indicates possibly very much longer ageing times would be required. The results for ageing mixtures at 125ºC indicate a relatively short ageing time, by comparison (about 190 hours). Both the upper and lower 95% confidence intervals indicate reasonable laboratory times for the target ageing.

4.6.2 Temperature susceptibility of the recovered aged bituminous binders

Only the recovered binders from mixtures are considered here. Figures 4.11 and 4.12 show the temperature susceptibility of the binders recovered from the mixtures. The curves generated in these charts were also fitted. Figure 4.11 reveals that the binders aged at 85ºC and 105ºC have similar temperature susceptibilities throughout the range. The binder aged at 125ºC showed a temperature susceptibility that diverged slightly from the other two ageing protocols at higher penetration values i.e. before much ageing. As the binder hardens (into the range of interest for this study – see red ellipse in Figure 4.11), the temperature susceptibility is similar to that for the other two protocols. In Figure 4.12, the binders from the three ageing protocols displayed similar temperature susceptibilities throughout the range. Generally, the charts revealed that the binders, virgin inclusive, have low temperature susceptibilities as the PI values range from -0.81 to 4.40.

4.6.3 Chemical composition of the recovered aged bituminous binders

Results in Tables 4.1 and 4.2 generally confirm other researchers' findings that as bitumen ages, the asphaltene content increases at the expense of the maltenes, i.e. maltenes reduce in content (Read and Whiteoak, 2003). In Figure 4.13, there were slight variations in the chemical compositions of the binders above a penetration value of 15dmm in which the aged binder at 125ºC displayed a noticeable difference of more than 10% in its value when compared to the binder aged at 85ºC (BS 2000: Part 143:2004 recommends not greater than 10% variation in value). The aged binder at 105ºC showed this behaviour above a penetration value of 22.5dmm. The chemical compositions below these points were similar to that of the binder aged at 85ºC.

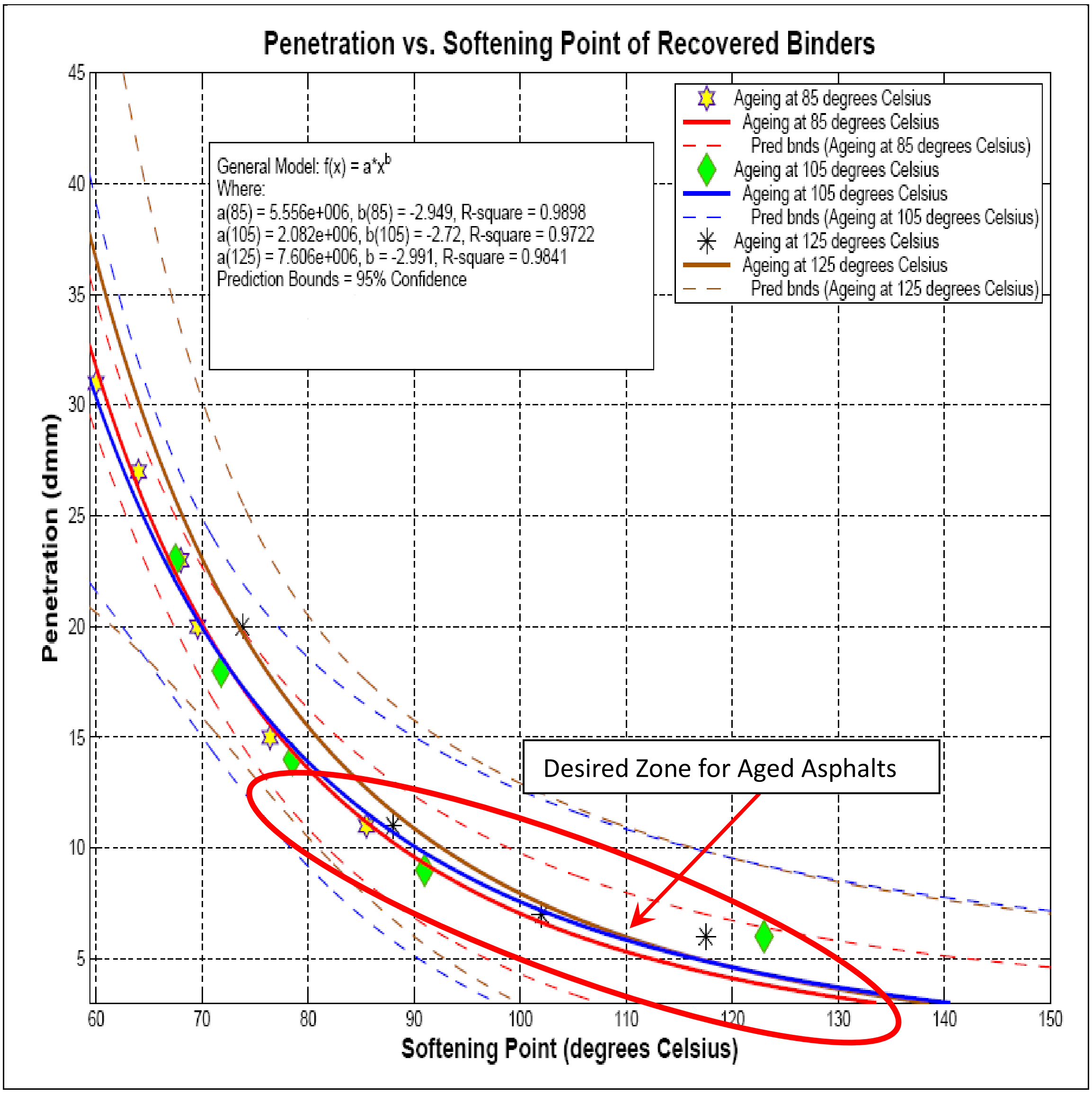


Figure 4.11: Temperature Susceptibility of Recovered Binders (Penetration vs. Softening Point)

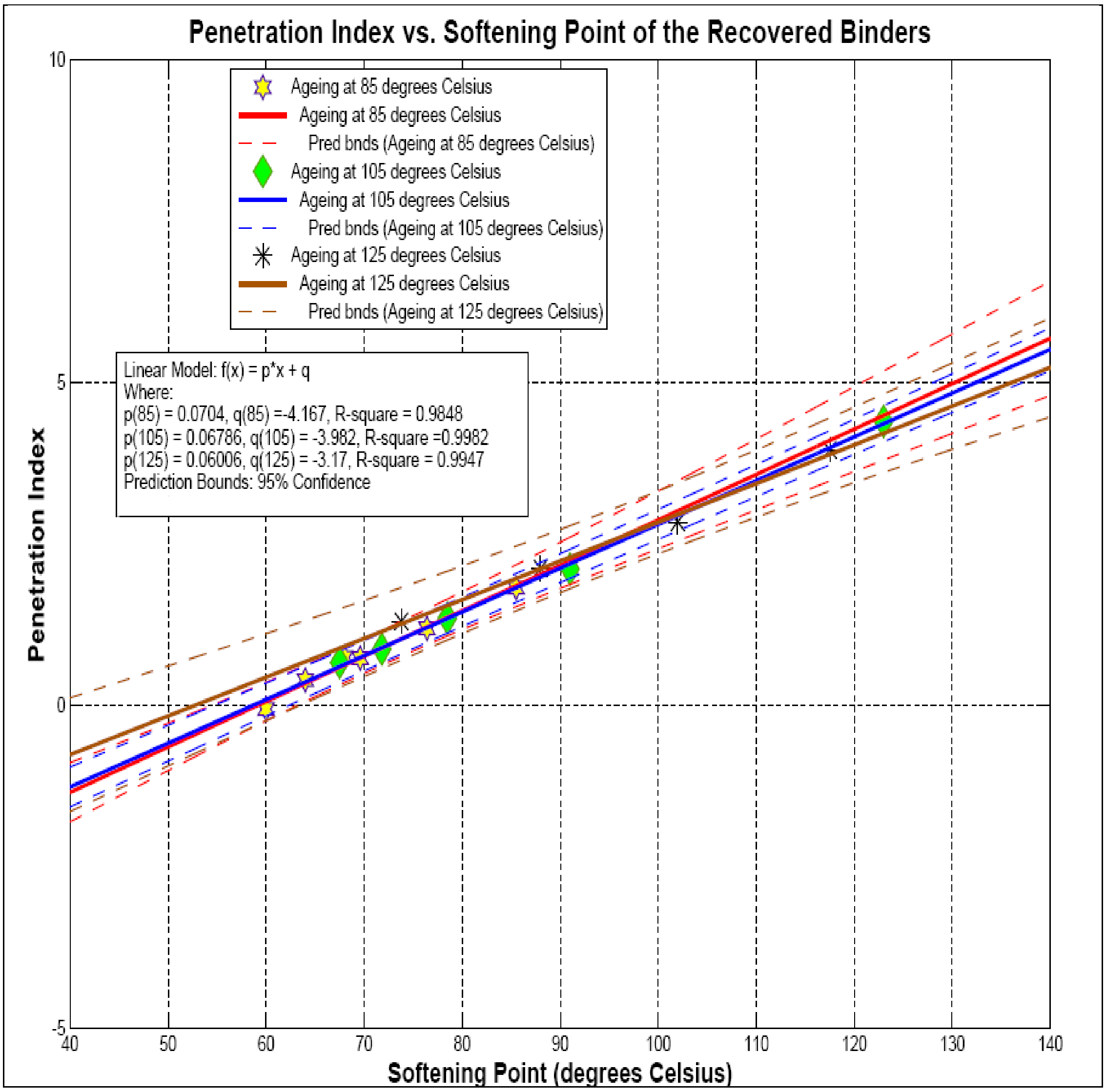


Figure 4.12: Temperature Susceptibility of Recovered Binders (Softening vs. Penetration Index)

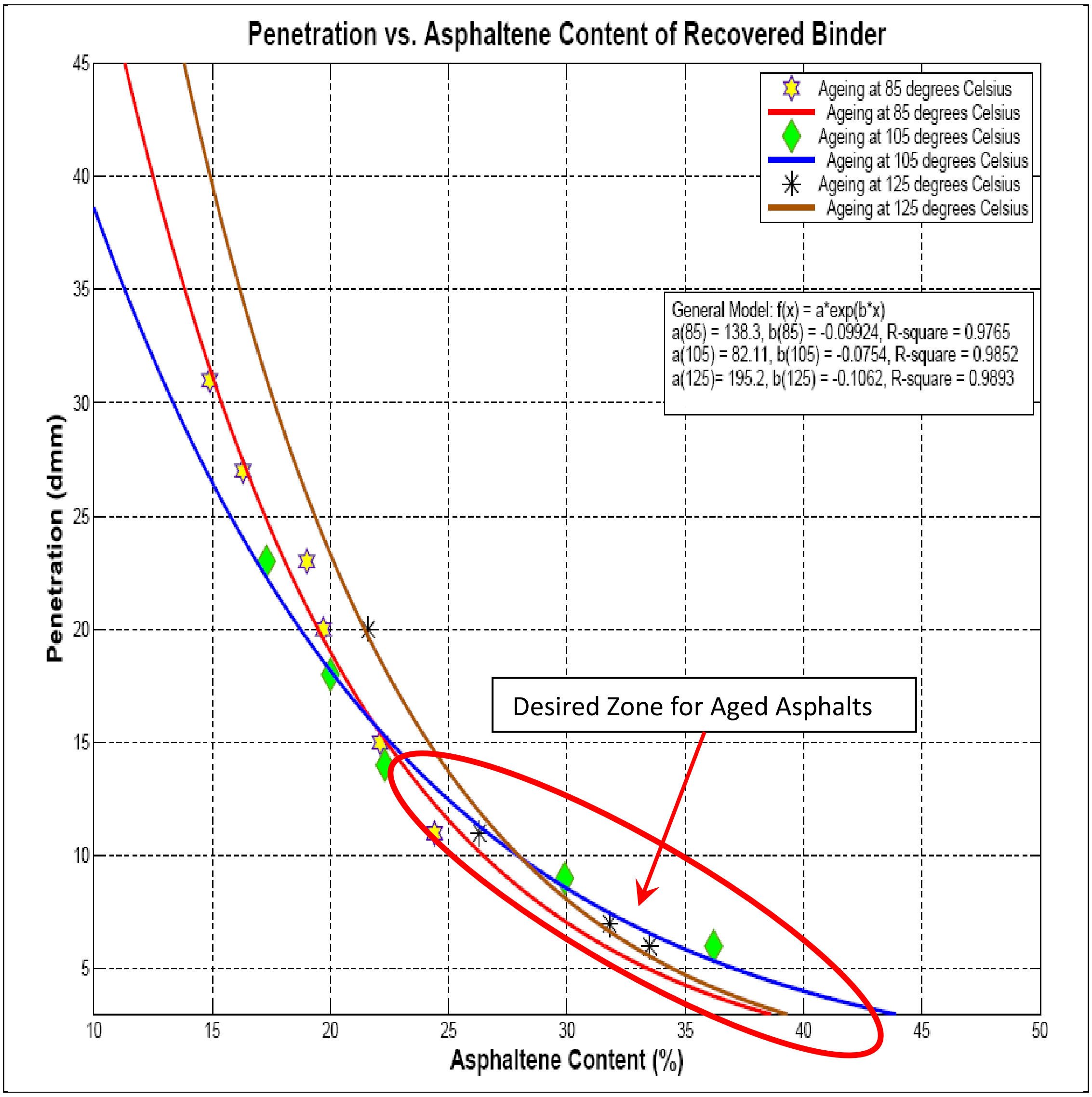


Figure 4.13: Penetration of Recovered Binders vs. Asphaltene Content

Figure 4.14 shows a slightly different behaviour for the binders compared to Figure 4.13. At a softening point of about 100ºC and above (at which penetration is about 7.5dmm and lower), the binders displayed varying chemical compositions. Considered alongside Figure 4.11, the implication is that, when aged binders from the three different ageing protocols were much softer, they have different chemical compositions, but as they harden, the chemical compositions assume the same state.

4.6.4 Rheological properties of the recovered aged bituminous binders

The rheological properties of the RTFO and PAV aged bulk binders (over the durations indicated in Table 4.2) and that of the recovered binder from a 20mm DBM mixture (using granite aggregate and 40/60 paving grade Venezuelan binder obtained from a local quarry) have been considered along with the recovered binders from the laboratory mixtures aged at 85ºC, 105ºC and 125ºC (over the hours stated in Table 4.1). In order to allow for direct comparison, binders with similar penetration values though with different ageing protocols were examined. For example, Figure 4.15 shows the viscoelastic responses of recovered aged binders with penetration values of 23dmm, but from different ageing protocols of 85ºC (mixtures ageing over 120hrs) and 105ºC (mixtures ageing over 24hrs) and the PAV at 100ºC (bulk binder ageing over 25hrs). It is obvious from the figure that they all have similar viscoelastic responses, more importantly, they all remained thermorheologically simple just as the parent 53dmm penetration original binder.

Similarly, Figure 4.16 shows the viscoelastic responses of recovered aged binders with penetration values of 20dmm, but from different ageing protocols of 85ºC (mixtures ageing over 240hrs) and 125ºC (mixtures ageing over 24hrs), PAV at 100ºC (bulk binder ageing over 15hrs) and recovered binder from 20mm DBM. The chart here reveals that there is a slight difference in the viscoelastic responses (slight shift in the trend of the curves) of the binders at this level, most especially between the PAV aged binder and the recovered binder from mixtures aged at 125ºC, at the lower frequencies and complex shear modulus. All the plots still indicate that the binders remained thermorheologically simple without any significant difference in the trend of their curves.

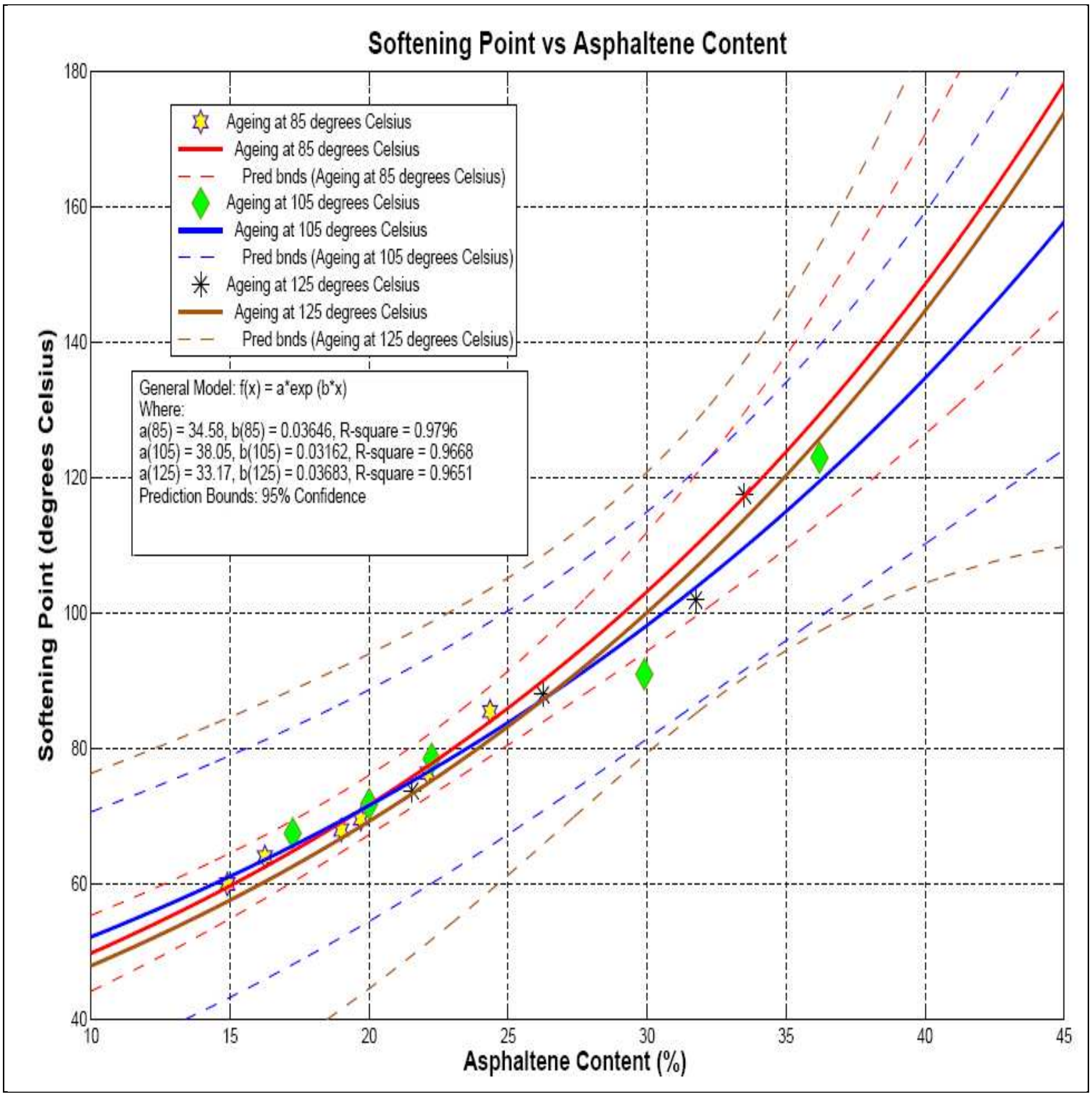


Figure 4.14: Softening Point of Recovered Binders vs. Asphaltene Content

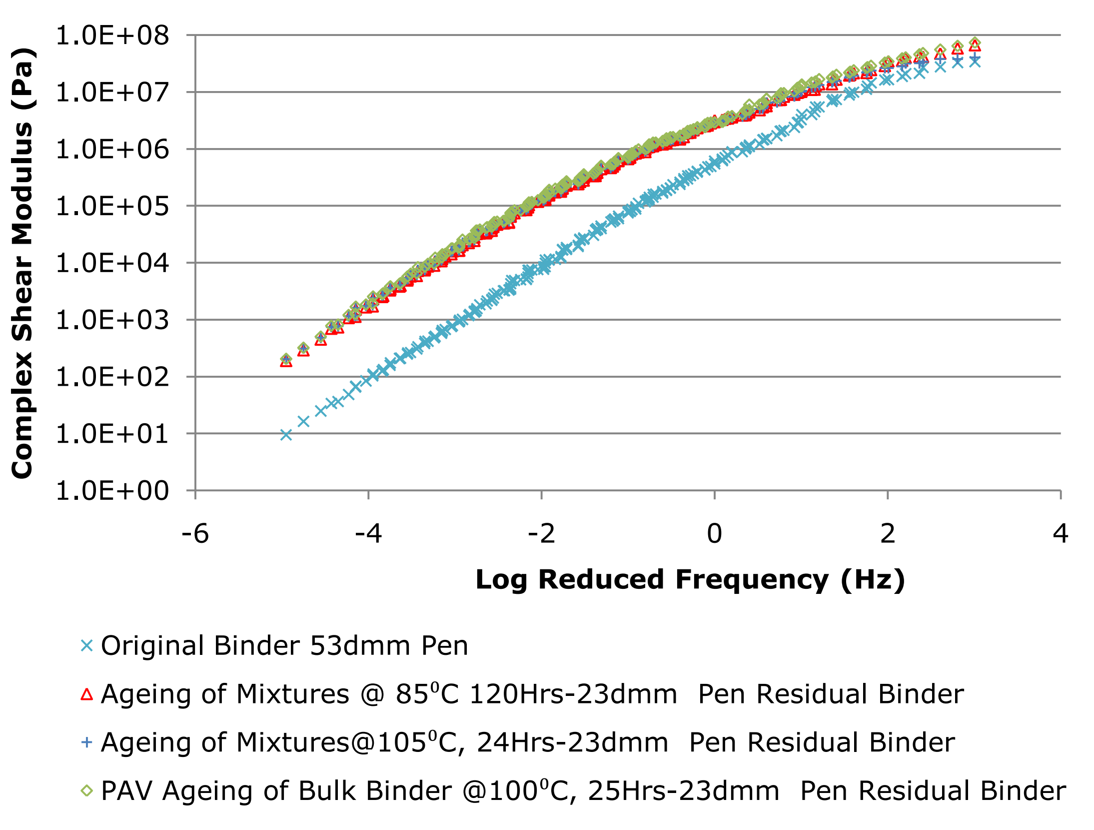


Figure 4.15: Complex Modulus Master Curves of 23dmm Pen Binders Aged Differently Compared to the Parent 53dmm Pen Binder

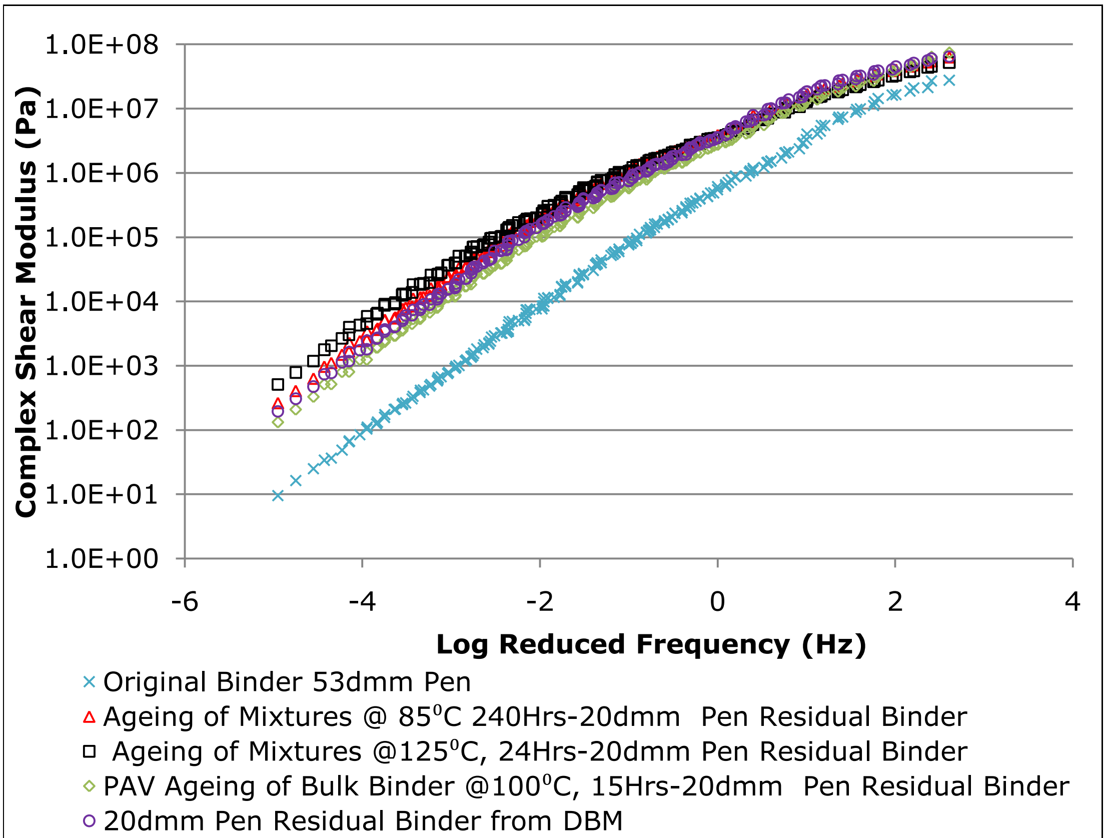


Figure 4.16: Complex Modulus Master Curves of 20dmm Pen Binders Aged Differently Compared to the Parent 53dmm Pen Binder

Figure 4.17 shows that bituminous mixtures aged at 85ºC and 125ºC have recovered binders of similar viscoelastic responses and they both remained thermorheologically simple. The penetration at this level is 11dmm. This observation is important to the theme of this investigation as it shows that there is little or no difference in the dynamics of ageing (or properties of the severely aged binder) in bituminous mixtures aged to a low penetration level, irrespective of whether it was at a temperature of 85ºC or 125ºC. Significantly, the findings also show that there is no departure from the thermorheological simplicity.

4.6.5 Comparison of rheological and physicochemical properties of binders

Figures 4.18 to 4.23 compare rheological properties (complex modulus and phase angle at 0.4Hz and 25ºC) with physicochemical properties of the binders recovered from aged mixtures and the bulk binders aged under the RTFO and PAV protocols. It can be seen from these figures that the binder, over the tested ranges and irrespective of the protocol by which it is aged, shows similar trends whether in mixture or as bulk binder.

4.7 Decision for Ageing Protocol and Mass Production of RAPs

Overall, from Figures 4.10 through to 4.23, it is obvious that, recovered binders from mixtures aged at 105ºC possess similar behaviours both in temperature susceptibility and viscoelastic response when compared to those aged at 85ºC. The chemical compositions of the binders also follow a similar trend. Recovered binders from bituminous mixtures aged at 125ºC displayed slightly different behaviour when soft (see Figure 4.13) but later exhibited similar behaviour to that of 85ºC and 105ºC when hardened. However, the differences displayed should not introduce significant errors if used for ageing mixtures under the proposed scenario being investigated, since there is not much difference exhibited between the binders in physical, chemical and mechanical properties most especially at the desired severely aged levels. This present work will make use of RAPs with recovered binders of penetration that is as low as 5dmm. More importantly, the beneficial role that ageing bituminous mixtures at 125ºC will facilitate getting down to the desired penetrations in a much shorter and predictable manner (see Figure 4.10) compared to those of 85ºC and 105ºC makes it attractive. From the said Figure 4.10, it has been projected that such desired levels could be attained at the latest in about 460Hrs (19.17days).

Similarly, comparing the results in this work with those carried out using

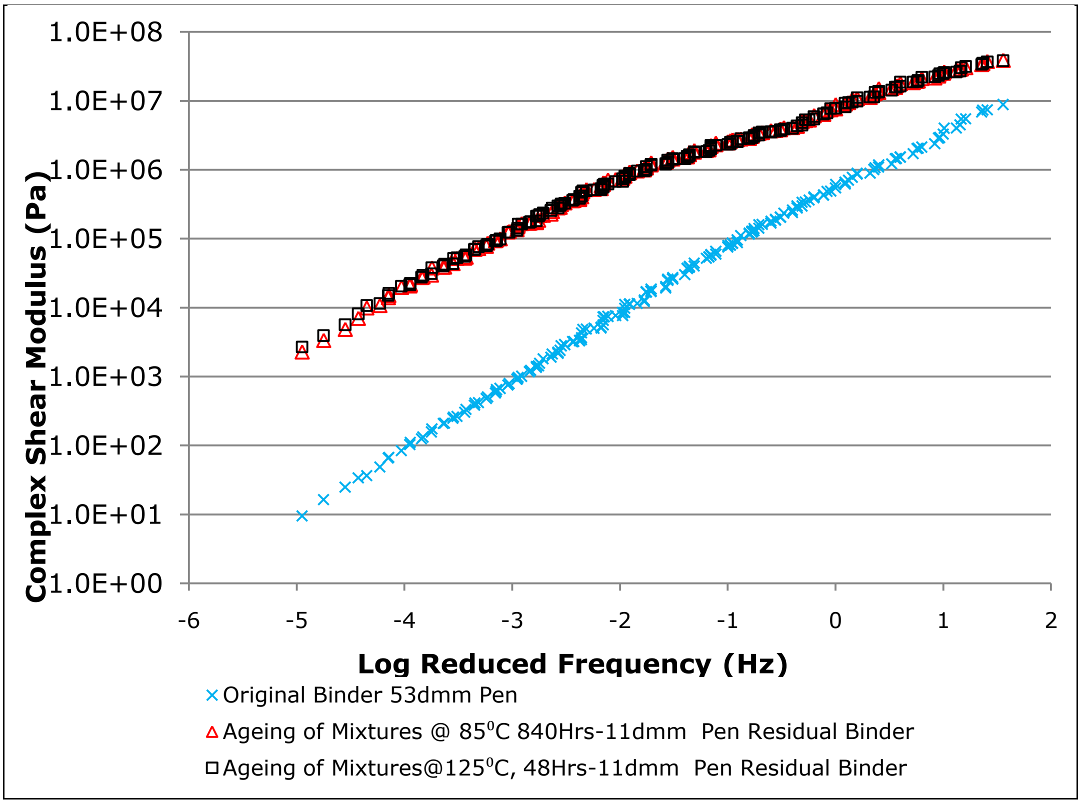


Figure 4.17: Complex Modulus Master Curves of 11dmm Pen Binders Aged at 85°C and 125°C Differently Compared to the Parent 53dmm Pen Binder

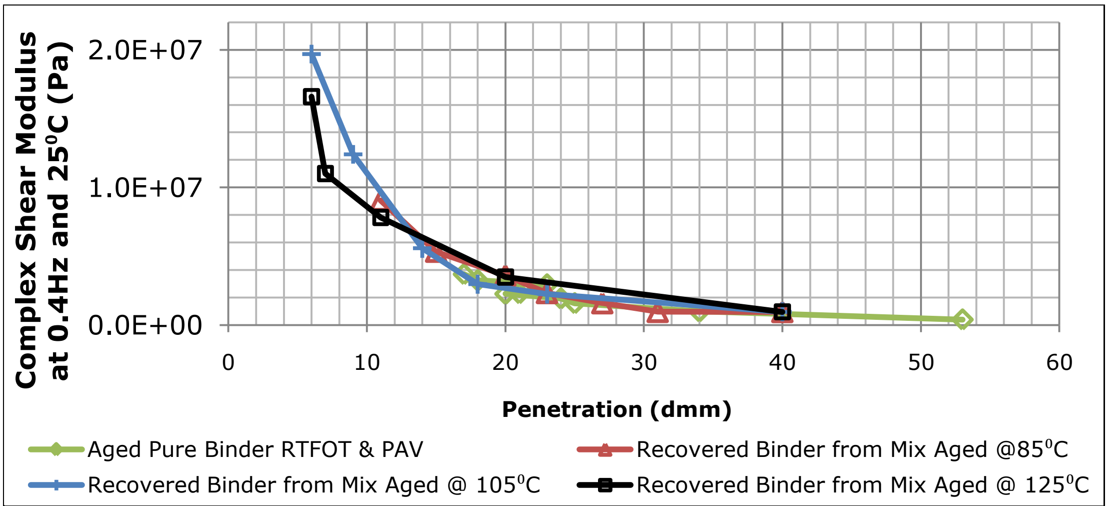


Figure 4.18: A Comparison of the Complex Modulus and Penetration of PAV and RTFO Aged 53dmm Pen Binder with that of Recovered Binder

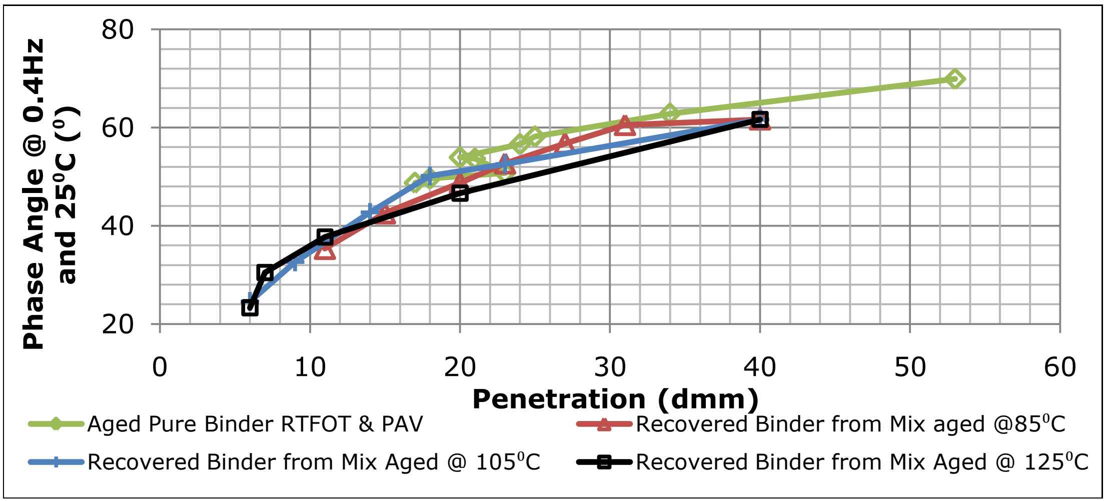


Figure 4.19: A Comparison of the Phase Angle and Penetration of PAV and RTFOT Aged 53dmm Pen Binder with that of Recovered Binder

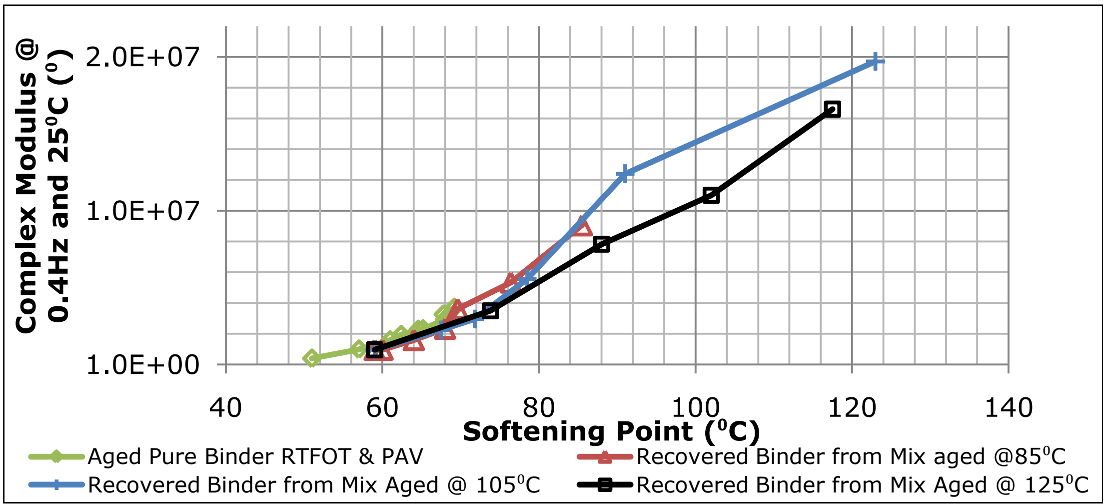


Figure 4.20: A Comparison of the Complex Modulus and Softening Point of PAV and RTFOT Aged 53dmm Pen Binder with that of Recovered Binder

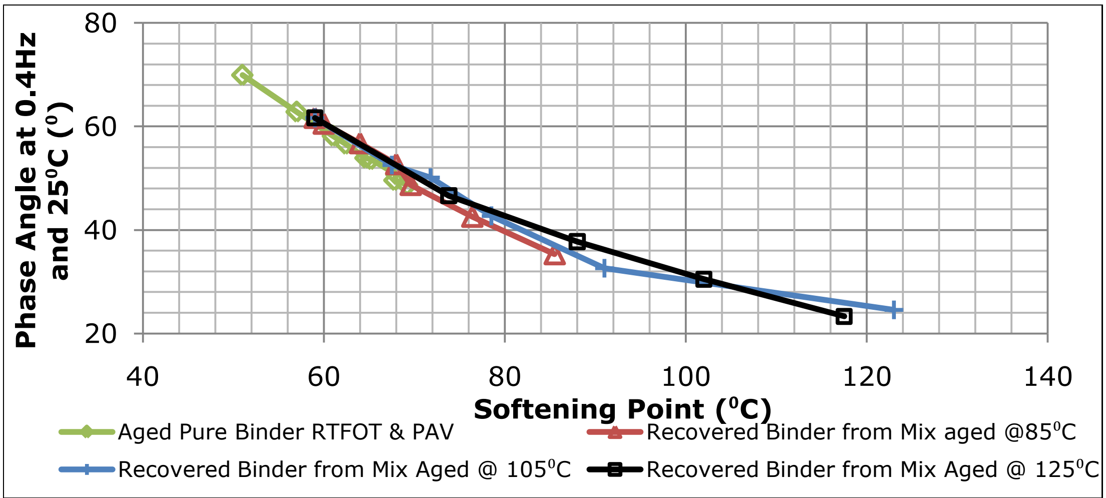


Figure 4.21: A Comparison of the Phase Angle and Softening Point of PAV and RTFOT Aged 53dmm Pen Binder with that of Recovered Binder

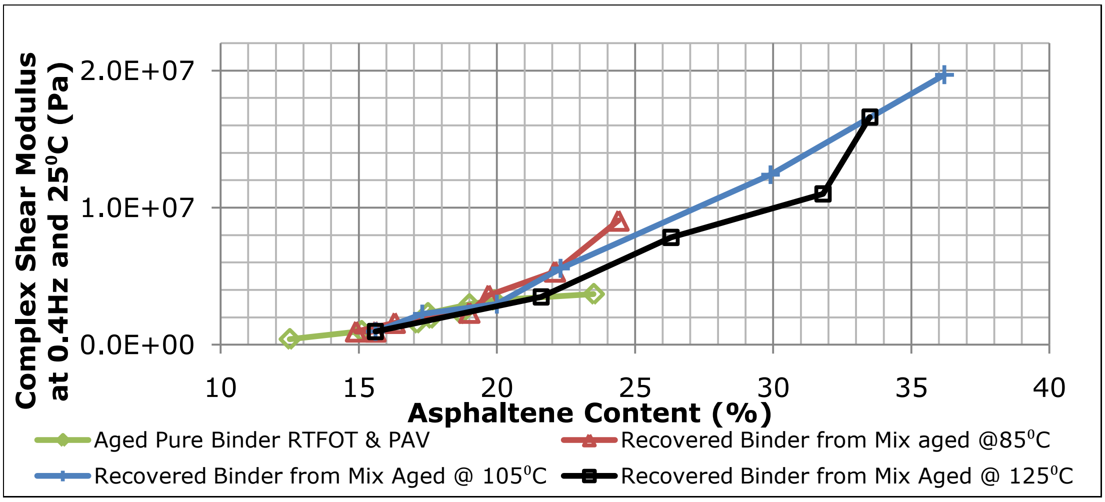


Figure 4.22: A Comparison of the Complex Modulus and Asphaltene Content of PAV and RTFO Aged 53dmm Pen Binder with that of Recovered Binder

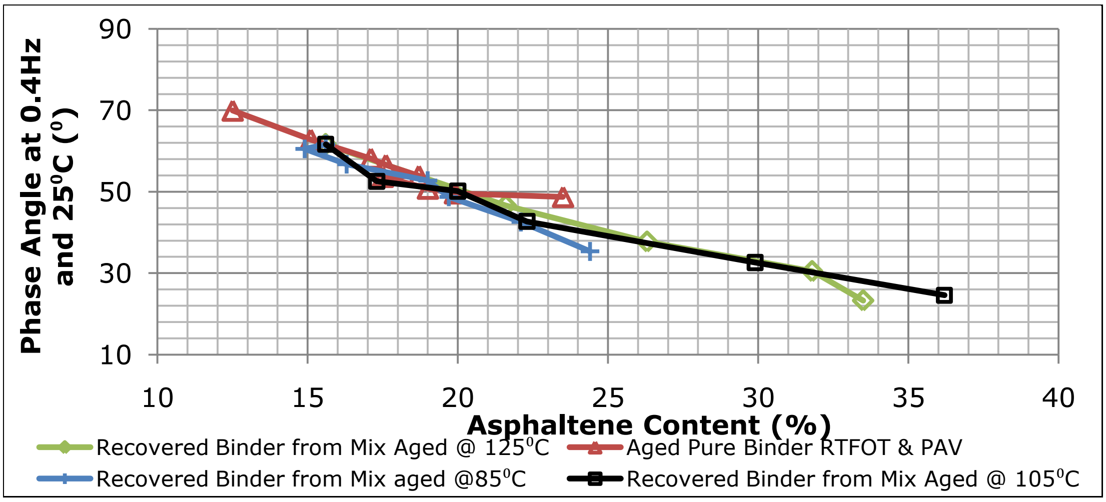


Figure 4.23: A Comparison of the Phase Angle and Asphaltene Content of PAV and RTFOT Aged 53dmm Pen Binder with that of Recovered Binder

compacted specimens shows that ageing loose asphalt material accelerates ageing. For example, ageing such loose mixtures at the standard temperature of 85ºC over 24hrs yielded a pen of 31dmm in this work. Nguyen (2009) in his work in which he used compacted specimens with similar binder reported a similar penetration level of 31dmm after 120hrs of ageing. The larger direct contact area in the loose mixtures that facilitates quicker oxidation (oxidation is the major factor that accelerates ageing in the presence of heat) must have been responsible for this significant difference in the rate of ageing between loose and compacted specimens.

Consequently, the ageing protocol that will be used for ageing samples in this research work, in order to mimic what obtains in asphalt pavements typical of hot tropical belts is 125ºC. This overall should assist in saving much laboratory time and at the same time obtaining sound and realistic results comparable to ageing bituminous mixtures either at 85ºC or 105ºC.

4.8 Summary

Based on the laboratory study presented in this chapter, the following conclusions can be drawn:

- Generally, as the binder used in this study (straight run paving grade binder 40/60 of Venezuelan origin with a penetration of 53dmm) ages, complex modulus, asphaltene content and softening point values increase while penetration values reduce irrespective of the ageing protocol i.e. either as bulk binder ageing or mixture ageing, in line with other researchers' findings.
- Ageing bituminous materials at 85ºC over 120hrs (5days) in the forced draft oven did not yield the severity of ageing (below 6dmm) required to study heavily aged pavements typical of the tropics. This standard protocol yielded a residual binder of 23dmm penetration for the original 53dmm penetration binder used. However, it was able to age to 11dmm penetration after 840hrs (35days).
- Ageing bituminous mixture at a higher temperature of 125ºC in the forced draft oven accelerates ageing, such that it only took 48hrs to reach a penetration of 11dmm compared to 840hrs (35days) for the standard protocol, thus saving laboratory time.
- Ageing loose bituminous mixtures in the forced draft oven accelerates ageing. For example, ageing such mixtures at the standard temperature of

85ºC over 24hrs yielded a pen of 31dmm in this work. Nguyen (2009) in his work in which he used compacted specimens with similar binder reported a similar penetration level of 31dmm after 120hrs of ageing.

- The rheological properties and temperature susceptibilities of less severely aged binders (20dmm penetration and above in this study), seem to differ slightly. This was most obvious between the PAV aged binders and the recovered binders of mixtures aged at 125ºC.
- The properties of more severely aged binders are independent of the ageing protocol.

5 Cold Bitumen Emulsion Mix Design and Initial Investigation on its Mechanical Performance

5.1 Overview

For most civil engineering structures, the need for adequate design that ensures efficient and effective performance in service cannot be overemphasized. It is common knowledge that in executing such designs, safety, aesthetics, function and economics are almost always given priority. Undoubtedly, flexible road pavement structure, which is the focus of the work reported in this thesis, is no exception. A flexible road pavement structure must principally be designed, such that it will facilitate smooth and safe ride (or passage of vehicular traffic both in size and weight) throughout its design life. These requirements by implication pose the two challenges of ensuring structural and functional viability of the road pavement throughout its design life. The design of course must be within the budget constraints. These are accomplished by proper mix and structural design of the pavement.

This chapter presents the process of cold bituminous emulsion mix design and results of the initial work done in assessing the mechanical performance of the mix. The cold mix design entailed optimising the volumetrics of the materials involved. The mechanical tests assessed the engineering properties of the designed mix bearing in mind durability and long term performance of the mix. Results obtained from such mechanical performance tests are used for the structural design aspect. Although the design principle adopted here is used for the well established hot mixtures, it should continue to be used for cold mixes until their nature is fully understood. More importantly, the trend in the industry is that such cold mixtures should meet same requirements set for hot mixtures.

5.2 Design Consideration for Cold Mixtures

As there is no general existing equipment made specifically for the design of cold mixtures, those for hot mixtures are most frequently employed though with slight modifications. These modifications are required because cold mixtures being intermediate materials behave as unbound granular materials in their early life stages (because of the presence of moisture) while they assume the state of bound materials just as hot mixtures after curing (due to the evaporation of a greater part of the moisture content in the mix). This behaviour is responsible for the various modifications that have been introduced to cold mix design over years. However, there is no rule of the thumb that can be followed.

5.3 Experimental Plan

The objective here is to come up with a cold mix design for mixes made from RAPs, some virgin aggregates and bitumen emulsion. Such will be suitable for executing a study on cold recycled mixtures for road pavements in developing countries with hot tropical climates. In this regard, harmonised recommendations from the works of, Ibrahim (1998), Jenkins (2000), Thanaya (2007), and Sunarjono (2008) for cold mix design as discussed in Chapter 3 of this thesis and the guidelines given by ARRA (2001) will be used as guide. Similarly, the suggestions given in Overseas Road Notes (ORN) 19 and 31 for aggregate (RAP) gradation and binder selection for heavily trafficked roads in the tropics will be applied in this work. The mix design procedure followed generally in this work follows thus:

- Produce artificial RAP in the laboratory.
- Determine RAP gradation to suit the recommendations of ORN 19, 31 and other relevant TRL documents for heavily trafficked roads in the tropics. Determine RAP binder content and gradation of extracted aggregates.
- Determine gradation of virgin aggregates. Determine the amount of additional granular materials required.
- Determine the type and grade of bitumen emulsion required for compatibility with aggregates. Characterise the bitumen emulsion.
- Estimate the initial residual bitumen content and initial emulsion content using the empirical relationship given in MS-14 (Asphalt Institute, 1989)
- Select suitable mixer (Hobart/Sun and Planet Mixers) and mixing protocol based on the gradation of RAP-virgin aggregates.
- Check Compactability of aggregates using a suitable compactor for mix (Gyratory, vibratory, roller or Marshall). Determine Optimum Moisture Content (OMC) and Maximum Dry Density (MDD) by varying moisture content.
- Select suitable procedure for curing (should simulate what obtains in the road pavement).
- Determine pre-wetting water content along with Optimum Total Fluid content (OTFC). Use the estimated initial binder content along with varying moisture content as Total Fluid Content (TFC). Start with TFC = OMC. Test cured specimens (dry and wet) using indicative tests such as Indirect Tensile Stiffness Modulus (ITSM) and the Indirect Tensile Strength (ITS) tests.

- Determine Optimum Bitumen Emulsion Content (OBEC) in Optimum Total Fluid Content (OTFC) using indicative tests just as above.
- Determine particle density and water absorption properties of aggregates. Determine maximum density of the mix.
- Determine compaction level to meet porosity target (10% air voids content).
- Conduct more indicative tests on fully optimised mix (ITS, ITSM and water sensitivity tests).
- Check compliance with relevant standards for performance.
- Estimate Job Mix Formula – **End of Mix Design.**
- Investigate engineering properties fully - fatigue response, rutting potential, resilient modulus, water susceptibility, ageing and temperature susceptibility.
- Proceed to structural design.

Figure 5.1 is the flow chart describing this mix design process that was followed. Meanwhile effort was made to ensure that the temperature of curing adopted was very close to the maximum temperature that obtains in the pavement layer being designed here (road base) for hot tropical climates. This was achieved by using the pavement temperature prediction model suggested by Mallick and El-Korchi (2009). The model allows for computation of temperature at the surface and any other depth below the surface.

$$T_{surf} = T_{Air} - 0.00618Lat^2 + 0.2289\,Lat + 24.4 \qquad (5.1a)$$

Where, T_{surf} = Temperature at the surface (◦C)

T_{Air} = Ambient temperature (◦C)

Lat = Latitude of the region concerned (Degrees)

For temperatures at different depths, the relationship below is used:

$$T_d = T_{surf}\,(1 - 0.063d + 0.007d^2 - 0.0004d^3) \qquad (5.1b)$$

Where, T_d = Temperature at depth d (°F)

T_{surf} = Temperature at the surface (°F)

d = Depth from the surface (inches)

The use of this model is demonstrated in Chapter 6 of this thesis.

5.4 Philosophy behind Experimental Plan

As mentioned earlier, mix design involves determining the optimum proportions of the mixture constituents that will ensure a satisfactory performance. ARRA (2001) noted that three basic theories have been proposed for designing such mixtures when RAP is involved:

Figure 5.1: Flow Chart for Cold Bituminous Emulsion Mix Design

I. RAP acts as black aggregate and mix design only requires determining the recycling additive required to coat the aggregate.
II. Based on the evaluation of the physical and chemical characteristics of the recovered bitumen, design is done to restore the residual aged bitumen to its original consistency - complete restoration of binder to its initial soft condition occurs.
III. Combines Theories I and II in which the design is based on the fact that the degree of softening is related to the properties of the old bitumen in RAP, recycling additive, and environmental conditions. Recommends mechanical testing since the degree of softening of aged bitumen is difficult to quantify.

The philosophy adopted in this work has an inclination towards the first theory. This is because the RAP materials considered in this work have very low penetration levels (5, 10, 15 and 20dmm), and at such levels, RAPs are regarded as black rocks and implicitly, the residual bitumen in such RAPs is inactive. While this may be the best practice for a cold temperate climate, it might not be a good assumption for hot tropical climates. This is understandable since bitumen is temperature dependent in behaviour with little or no activity taking place in it at low temperatures (becomes brittle). Karlsson (2002) opined in his work that one of the factors that enhance rejuvenation of bituminous binders is temperature.

Part of the primary objective of this research work is to ascertain the level at which residual bitumen from such severely aged RAPs begins to enhance the engineering properties of cold recycled mixes under a hot tropical climate. A comparative study therefore must be conducted alongside with cold mix made from 100% virgin mixtures with the same bitumen emulsion content. Having been treated similarly, the engineering properties of the mixes i.e. recycled (with varying degrees of ageing severity) and 100% virgin aggregate should give an indication of the extent of contribution by residual bitumen from severely aged RAPs to the mix properties.

The mix design here considered relies on the information obtained for the strength and stiffness using the Indirect Tensile Strength (ITS) test and the Indirect Tensile Stiffness Modulus (ITSM) test respectively. The RAP with 20dmm penetration residual bitumen was used for the mix design process.

5.5 Materials and Their Characterisation

The materials used for this work were:

- Nymuls CP 50 (Bitumen Emulsion) supplied by Nynas Bitumen,
- 20mm DBM obtained from Cliffe Hill Quarry. Used for producing RAPs,
- Granite Aggregates (of sizes 20mm, 14mm, 10mm, 6.3mm, 5mm dust and filler) obtained from Cliffe Hill Quarry, Ellistown, Leicestershire,
- Filler material (Limestone) supplied by Longcliffe Quarries Limited, Matlock, Derbyshire,
- Demineralised water (obtained from the lab).

The bitumen emulsion and 20mm DBM were both characterised in line with the respective BSEN standards.

5.5.1 Bitumen emulsion

The bitumen emulsion used was the cationic type though a proprietary one – Nymuls CP 50 from Nynas bitumen. Evidence from literature as reviewed in Chapter 2 is in favour of using such emulsion with granite for compatibility reasons. An emulsion adheres best to aggregates when they have opposite charges. The implication is that anionic emulsions should be used with positively charged aggregates such as limestone, while cationic emulsions are deem suitable for negatively charged aggregates, the group to which both granite and quartzite belong (Thanaya, 2003). The bitumen emulsion was characterized before it was used. Residues recovered (BSEN13074: 2002) by evaporation (curing in the Oven at 30 ± 2^0 over 72hrs to simulate a typical ambient tropical condition) from Nymuls CP 50 were subjected to penetration and softening point tests in line with BS EN1426:2007 and BS EN1427:2007 respectively. The rheological properties (BS EN1477:2005) of the residues were also determined (using the Bholin Gemini DSR).

Figure 5.2 shows moisture loss and time relationship for the bitumen emulsion while the values in Table 5.1 were used for plotting the relationships. The result showed among other things that the bitumen-water ratio is approximately 60:40. The Penetration of the residue (binder) was determined as 48dmm and Softening Point 51.4^0C. As expected, it was observed that these values are close to 53dmm and 51^0C which were the penetration and softening point values respectively for the original binder used for the production of the bitumen emulsion (40/60 paving grade Venezuelan bitumen). The slight differences could have been as a result of the residue recovery procedure/emulsion additives. Also, the results showed that

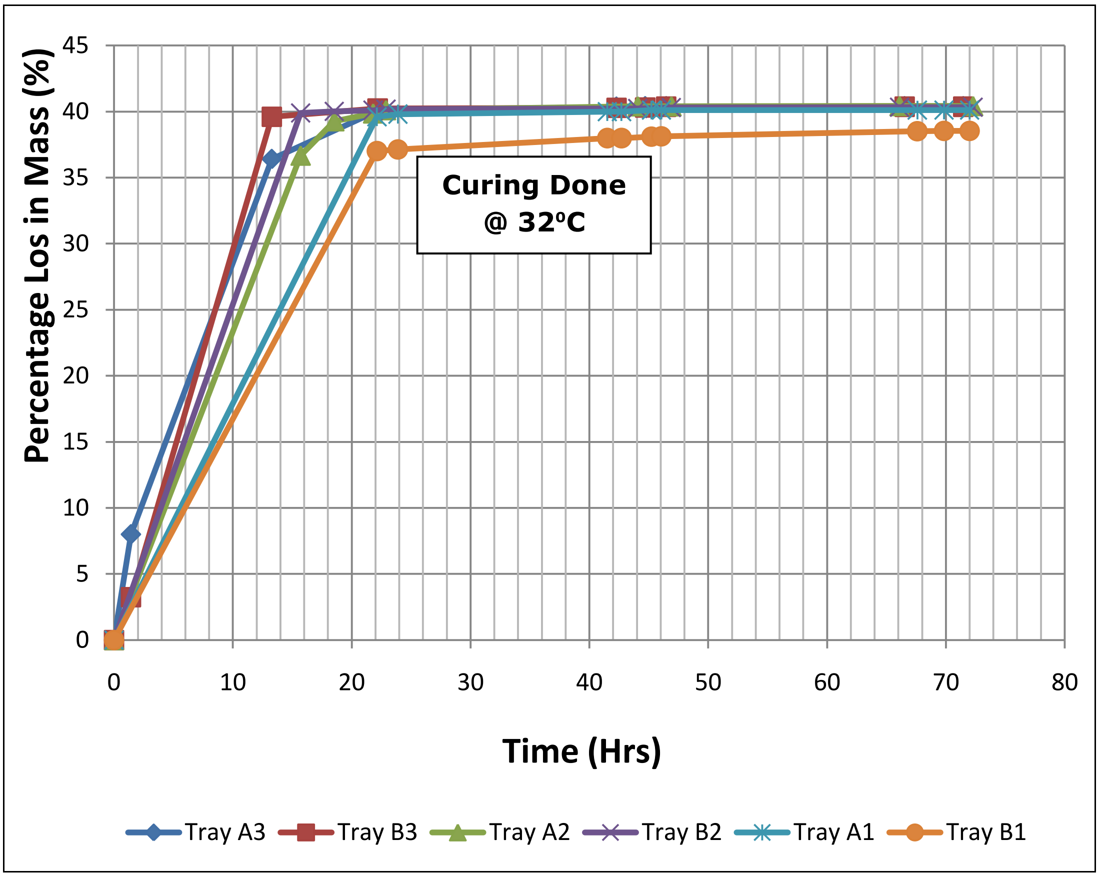


Figure 5.2: Percentage Loss in Mass of Cured Bitumen Emulsion at 30±2°C over 72Hrs

Table 5.1: Percentage Loss in Mass of Cured Bitumen Emulsion at 30±2°C over 72Hrs

Time (Hrs)	Percentage Loss in Weight (%)		Time (Hrs)	Percentage Loss in Mass (%)		Time (Hrs)	Percentage Loss in Mass (%)	
	Tray A1	Tray B1		Tray A2	Tray B2		Tray A3	Tray B3
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
22.12	39.61	37.02	15.70	36.67	39.90	1.42	8.00	3.25
23.88	39.81	37.13	18.53	39.27	40.01	13.27	36.40	39.61
41.50	40.00	37.99	21.80	39.86	40.10	22.18	40.06	40.23
42.70	40.00	37.99	22.88	40.11	40.19	42.28	40.36	40.28
45.22	40.11	38.11	44.07	40.40	40.24	44.70	40.39	40.31
46.05	40.11	38.13	46.90	40.40	40.30	46.47	40.39	40.37
67.60	40.11	38.51	66.10	40.42	40.33	66.53	40.39	40.37
69.85	40.11	38.54	72.28	40.42	40.33	71.47	40.39	40.37
72.00	40.11	38.54						

more than 90% of the moisture in the emulsions had evaporated in the first 13hrs of curing. Figures 5.3a and 5.3b show the master curve relationships for residues as obtained from four (4) trays before they were all combined. The rheology of all the residues compared well. The slight shift in the curve of the original binder is understandable as it had a penetration (at 25ºC) of 53dmm compared to 48dmm of the residues. The Redwood II Viscosity of the emulsion was determined in line with BS434: PART 1: 1984, though at 30^{0}C as 5.14s and at 20^{0}C as 15s. This gave an idea of the viscosity of the emulsion under ambient conditions.

5.5.2 20mm DBM

It was thought that obtaining this material right from the quarry in bulk rather than producing it for subsequent laboratory ageing as determined in Chapter 4 would go a long way in saving laboratory time on the research work. Therefore, about 1.5 Tonnes of 20mm DBM materials which were produced using paving grade 40/60 Venezuelan bitumen (supplied by Nynas) and granite aggregates, were collected from Cliffe Hill Quarry. These were bagged in paper bags (about 70bags of 25kg) while still hot and subsequently transported to the laboratory (see background picture in Figure 5.4) for storage.

The composition and condition of this material (i.e. 20mm DBM), was ascertained by conducting a composition analysis in line with BS598-102:2003, BSEN 933-1 and BS598-101:2004. The result indicated that the residual binder content was 4.25% while the penetration and softening point values were determined as 21dmm and 66^{0}C respectively. The gradation of the reclaimed aggregate compared to the standard 20mm DBM curve is shown in Figure 5.5. Gradation (particle size distribution) of aggregate sample entails separating individual compositional elements of such loose aggregate to group of sizes starting from the maximum size down to the minimum. This is normally achieved by conducting sieve analysis on the mechanical sieve shaker using a stack of sieves, which are arranged in descending order of sizes (maximum size is on top and receives the sample first). Figure 5.4 shows some inset pictures at the bottom of the figure in which the appearance of the loose form of the 20mm DBM has been compared with that of its reclaimed aggregates. The gradation indicates that the reclaimed aggregate curve is within the limits of both the upper and lower bounds for 20mm DBM gradation. Figure 4.15 in Chapter 4 shows the master curves of the recovered binder (recovery by distillation following BSEN12697-4:2005) from the 20mm DBM. As discussed earlier, the rheology compares well with those aged in the laboratory (with similar penetration value).

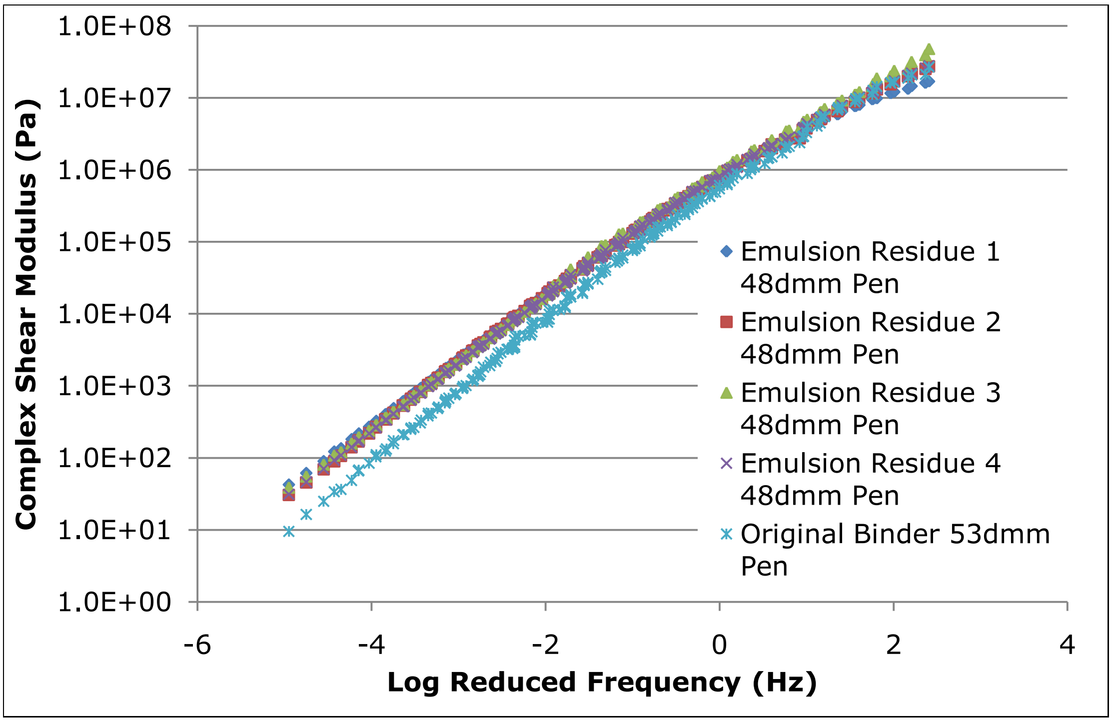


Figure 5.3a: Complex Modulus Master Curve Relationship for residues as obtained from 4 trays before they were all combined

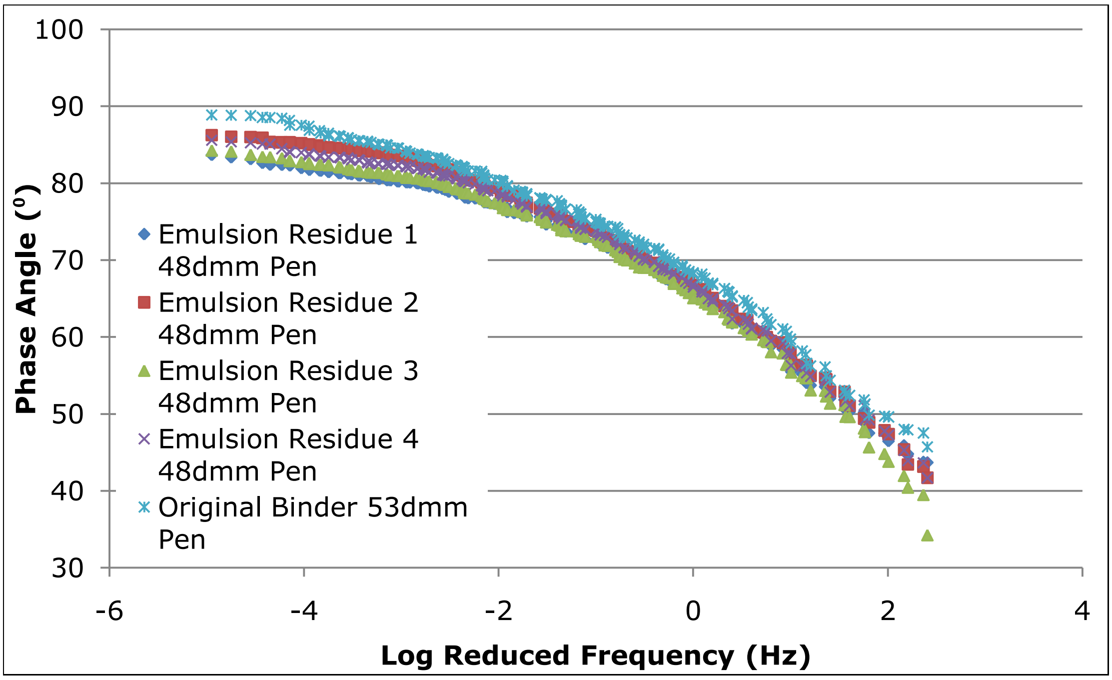


Figure 5.3b: Phase Angle Master Curve Relationship for residues as obtained from 4 trays before they were all combined

Figure 5.4: 20mm DBM (inset at the bottom right hand corner shows the appearance of reclaimed aggregates from the 20mm DBM)

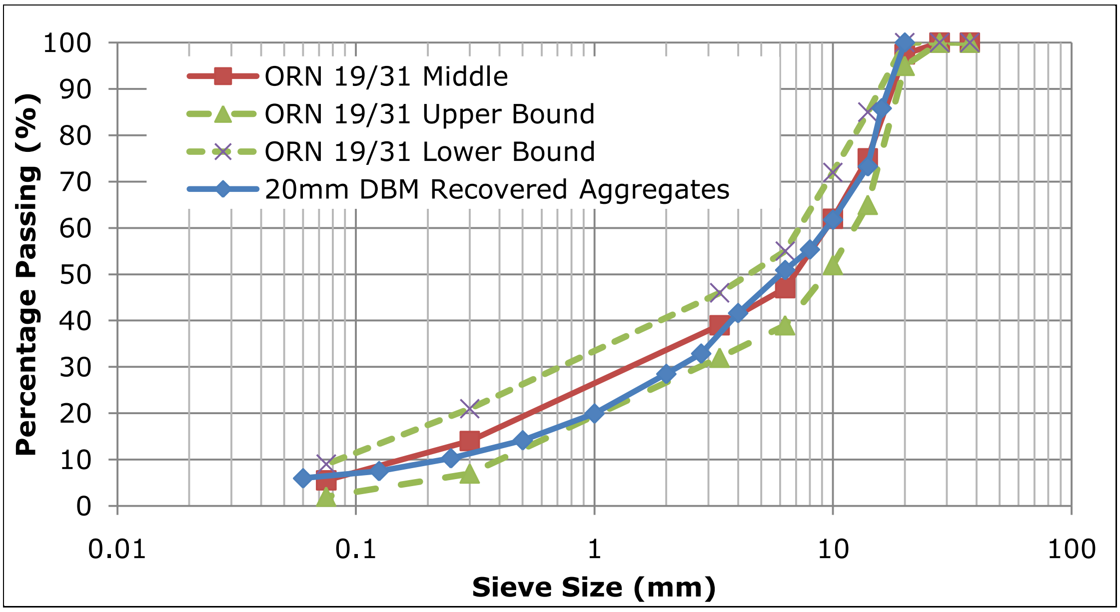


Figure 5.5: Gradation Curve for Reclaimed Aggregates from 20mm DBM

5.5.3 Virgin aggregates

As earlier stated in Chapter 4, granite was used in this work for the purpose of simulating road pavement construction practices in Nigeria. For the obvious reason that granite is abundantly available in Nigeria, they are commonly used for road pavement construction. In this work, seven granite aggregate sizes including filler were collected from Cliffe Hill Quarry. Some limestone fillers were also obtained from the Longcliffe Quarry. These were immediately dried in the forced draft oven, bagged (using polythene bags) and tinned for storage.

The 20mm DBM gradation curve as detailed in the ORN 19 & 31 was adopted for this work mainly because it is suitable for the road base of heavily trafficked roads in the tropics. The upper and lower bounds and the maximum density curves are showed in Figure 5.5, while Table 5.4 showed the relevant percentages passing through. Except for sizes 5mm dust and filler which were properly characterised for gradation, all the other sizes were screened using the relevant sieves on the mechanical sieve shaker. The virgin aggregates generally had Elongation Index of 11.0 and Flakiness Index of 3.16 (all within acceptable standard limits).

5.5.4 Laboratory aged RAPs

Artificial RAP materials were used in this investigation. The 20mm DBM earlier described in Section 5.5.2 was used in producing the RAP materials. Figure 4.10 in Chapter 4 proved very useful in this exercise as it assisted in the production of RAP materials with different degrees of ageing as required for this laboratory exercise. RAPs were categorised as those with residual bitumen of 5dmm, 10dmm, 15dmm and 20dmm penetration values and produced accordingly using the timings specified on the said figure. The other aspects considered are further discussed in the succeeding section. As earlier mentioned, the RAP with the residual binder of 20dmm penetration was used for the mix design.

5.6 Determination of Aggregate Gradation for Proposed Mix

RAP materials were produced from the 20mm DBM by first slightly heating and compacting them into slabs (305mm x 305mm x 50mm) at a temperature of about 150^{o}C using the roller compactor. Exposure to heat was carefully controlled (took average of 21minutes in the microwave oven) in order not to introduce any significant ageing to the material. This precaution proved useful as penetration and softening point values were determined thereafter and found to be 20dmm and 68.8^{0}C respectively (pre-compaction condition was penetration = 21dmm and softening point = 66^{0}C). Meanwhile the compaction was generally done to refusal (i.e. void content of about 3-5%). This compaction was done in order to simulate the normal condition of a typical road pavement.

After compaction, the materials were allowed to cool down and then broken down into sizeable units (70mm x 70mm x 50mm) using the Kango Hammer. The materials were subsequently crushed using the Jaw Crusher. Crushing was done to facilitate effective and uniform ageing within a relatively short time. Openings of 8mm, 14mm, 18mm, 22.5mm, 24mm, 28mm and also 28mm to 14mm, 28mm to 8mm and a combination of 18mm and 8mm were tried in a bid to determine a suitable opening that would give a gradation which requires minimal virgin material for the proposed design gradation i.e. 20mm DBM gradation. At the end of the exercise, the 18mm opening was found suitable. Table 5.2 shows the gradation properties of some of the openings used on the Jaw Crusher. Table 5.3 details the gradation properties of materials produced using the 18mm opening on the Jaw Crusher. The materials produced in the process generally had an average Elongation Index of 17.02 and an average Flakiness Index of 6.38 (all within acceptable standard limits).

While the materials, just as produced directly from the crusher were taken as the least aged set of RAPs (residual binder being 20dmm), some other sets of the same material were further aged in the forced draft oven using the appropriate timings in Figure 4.10 of Chapter 4. The target penetration values were 5dmm, 10dmm and 15dmm for studying the effect of varying degrees of RAP ageing on the engineering properties of cold bituminous emulsion mixtures (CBEMs). The gradation of the RAP material as indicated in Figure 5.6 shows that it is coarse and thus would require some dust and filler portions in order to fall within the ideal grading envelope for 20mm DBM. It is worth mentioning here that the materials following this ageing became lumpy. The procedure that was followed in rectifying this problem is fully discussed in Chapter 4 of this thesis and thus needs no revisiting here.

After some rounds of iterations using the gradation properties of RAP, 5mm dust and filler, with the maximum density curve properties set as target, a ratio of 65:30:5 for combining RAP, 5mm dust and mineral filler material respectively was found suitable and subsequently adopted. Table 5.4 details the gradation properties of the RAP, 5mm Dust, Mineral Filler, Maximum Density and ORN 19&31 for 20mm DBM, while Table 5.5 shows how the materials were combined to produce the design gradation curve. Figure 5.6 shows graphically the gradation of the materials while Figure 5.7 reflects the composition and proportion of the design mix.

Table 5.2: Gradation Properties of Some of the Openings used Compared to that of the Maximum Density

Size (mm)	Percent Passing (%)								
	28mm to 8mm JCO	14mm JCO	18mm + 8mm JCO	28mm to 14mm JCO	18mm JCO	22.5mm JCO	24mm JCO	28mm JCO	Max Density Curve (Target)
37.5	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
28	100.0	100.0	100.0	100.0	100.0	98.7	91.7	86.7	100.0
20	100.0	100.0	94.3	99.8	94.3	84.2	71.1	59.7	86.0
14	100.0	89.3	65.4	91.1	65.4	62.9	46.4	40.8	73.2
10	97.0	54.8	43.6	60.7	43.6	43.0	29.5	26.6	62.9
6.3	60.0	30.0	39.7	34.7	25.2	26.0	16.8	15.4	51.1
5	51.6	24.7	34.7	29.8	21.7	22.6	14.2	12.8	46.1
3.35	31.5	14.3	20.8	17.8	12.4	13.6	8.4	7.1	38.5
2.36	26.6	12.1	18.3	15.0	10.6	11.3	7.0	6.1	32.9
1.18	13.4	6.5	9.9	6.7	5.4	5.4	3.9	3.1	24.1
0.6	6.7	3.7	5.6	3.1	3.0	2.5	2.2	1.7	17.7
0.3	3.4	2.1	3.2	1.5	1.8	1.1	1.2	1.0	13.0
0.212	2.3	1.5	2.3	1.0	1.3	0.7	0.9	0.7	11.1
0.15	1.5	1.1	1.7	0.6	1.0	0.5	0.6	0.5	9.5
0.075	0.5	0.5	0.8	0.2	0.5	0.1	0.3	0.2	7.0

JCO implies, Jaw Crusher Opening

Table 5.3: Gradation Properties of Materials Produced using 18mm Opening on the Jaw Crusher

Size (mm)	18mm (% Retained)					Average
	Test 1	Test 2	Test 3	Test 4	Test 5	
37.5	0.0	0.0	0.0	0.0	0.0	0.0
28	5.2	3.6	5.4	5.7	7.7	5.5
20	27.5	29.7	38.0	44.4	37.2	35.4
14	19.5	16.7	17.6	17.7	19.9	18.3
10	14.1	16.1	13.9	15.1	16.8	15.2
6.3	7.4	7.2	4.3	4.2	4.5	5.5
5	13.7	15.0	9.6	8.5	8.2	11.0
3.35	3.0	3.0	1.9	1.4	1.5	2.2
2.36	6.4	6.0	5.2	2.2	2.8	4.5
1.18	2.2	1.8	2.1	0.5	0.8	1.4
0.6	0.8	0.6	1.0	0.2	0.4	0.5
0.3	0.2	0.1	0.3	0.1	0.1	0.2
0.212	0.1	0.1	0.2	0.1	0.1	0.1
0.15	0.1	0.1	0.3	0.1	0.1	0.1
0.075	0.0	0.0	0.1	0.0	0.1	0.1
Total	100.0	100.0	100.0	100.0	100.0	100

Table 5.4: Gradation Properties of the Designed Mix, RAP, 5mm Dust, Mineral Filler, Maximum Density, and ORN 19&31 for 20mm DBM

Size (mm)	RAP (% Passing)	Dust (% Passing)	Filler (% Passing)	Maximum Density (% Passing)	ORN Lower Bound	ORN Upper Bound
28	100.00	100.00	100.00	100.00	100.00	100.00
20	94.50	100.00	100.00	85.95	95.00	100.00
14	59.10	100.00	100.00	73.20	65.00	85.00
10	40.80	100.00	100.00	62.92	52.00	72.00
6.3	25.60	99.82	100.00	51.11	39.00	55.00
3.35	9.10	83.26	100.00	38.46	32.00	46.00
0.3	0.50	23.79	100.00	12.99	7.00	21.00
0.075	0.10	4.95	97.45	6.96	2.00	9.00

Table 5.5: Combination of RAP, 5m Dust and Mineral Filler to Produce the Design Gradation

Size (mm)	RAP (0.65) Design (% Retained)	Dust (0.3) Design (% Retained)	Filler (0.05) Design (% Retained)	Combination Design (% Retained)	New Mix (Percent Passing)
28	0.00	0.00	0.00	0.00	100.00
20	3.58	0.00	0.00	3.60	96.40
14	23.01	0.00	0.00	23.00	73.40
10	11.90	0.00	0.00	11.90	61.50
6.3	9.88	0.05	0.00	9.90	51.60
3.35	10.73	4.97	0.00	15.70	35.90
0.3	5.59	17.84	0.00	23.40	12.50
0.075	0.26	5.65	0.13	6.00	6.40
Pan	0.07	1.49	4.87	6.40	

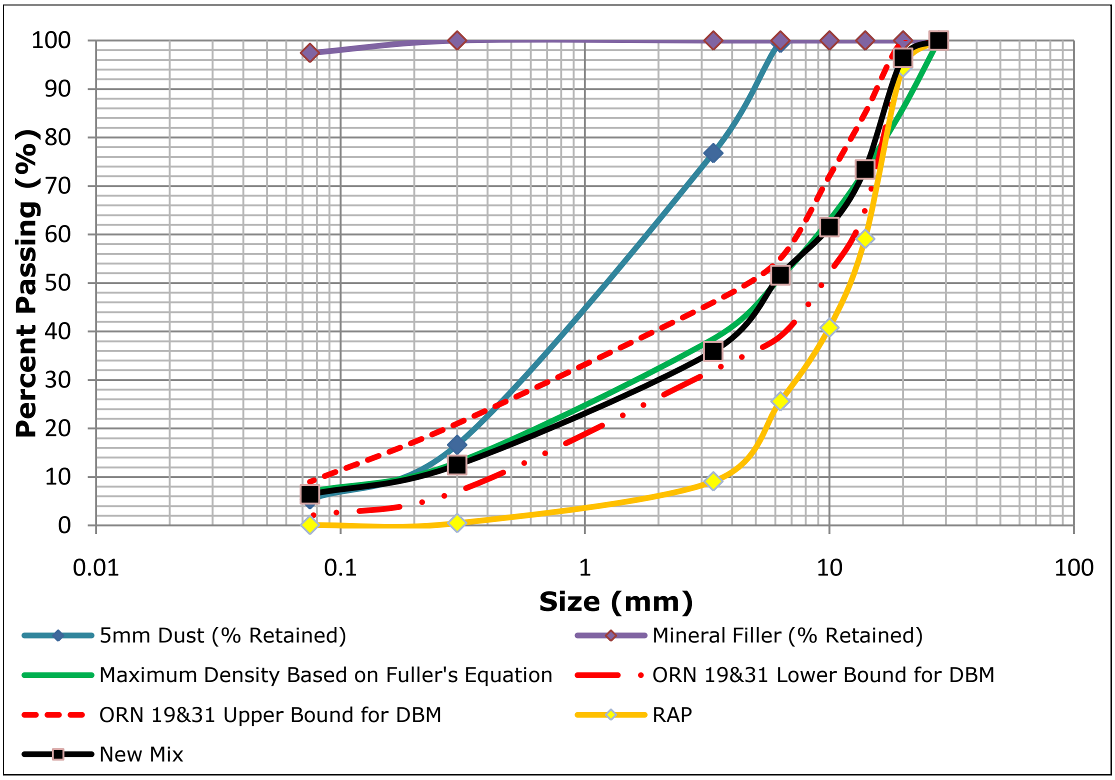


Figure 5.6: Gradation Curves of the Designed Mix, RAP, 5mm Dust, Mineral Filler, Maximum Density and ORN 19&31 for 20mm DBM

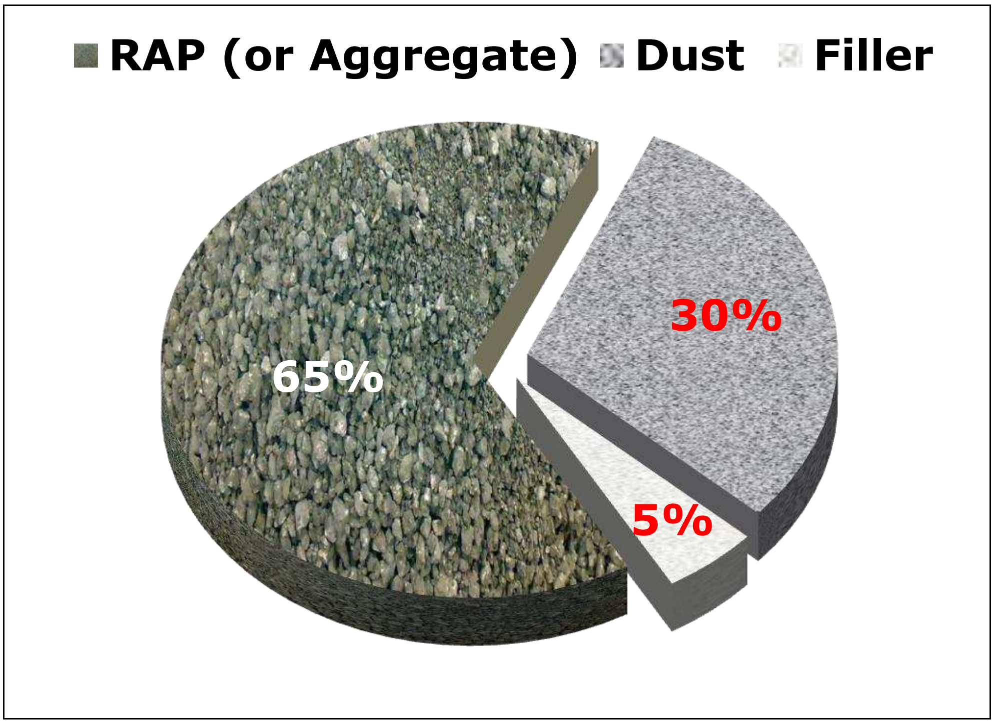


Figure 5.7: Composition and Proportion of the Designed Mix

5.7 Particle Density and Water Absorption of RAP and Virgin Aggregate

These were determined for both the RAP-virgin aggregate and virgin aggregate loose dry mixtures. The mixtures were in the proportions determined in the preceding section. Table 5.6 details some of the properties for the RAP-virgin aggregate while Table 5.7 details such for virgin aggregate mixture. The properties determined were particle density, saturated surface dry density, apparent particle density and water absorption all after BSEN1097-6:2000. Figure 5.8 shows the various stages and the condition of aggregates during the test.

As expected, the virgin aggregates mixture was generally denser than the RAP mixture. However, the virgin aggregates mixture had lower water absorption value compared to the RAP mixture. This observation was also made by Locander, (2009), Sunarjono (2008) and Jitareekul (2009) in their works. This was against logic since RAP aggregates were presumed to have been coated by the aged residual bitumen in them and in essence sealed a substantial part of the pores (voids in mineral aggregate) in the aggregate which ordinarily would have held or absorbed the moisture.

Although the earlier mentioned researchers did not give explanation for this, the fact that RAP aggregates are more or less conglomerates of coarse and fine grains gives an insight into it. The assumption is that despite most of the pores on the RAP already being sealed, there is a tendency for moisture to get locked up in spaces between fine RAP particles that are glued to the coarse RAP. Such spaces are difficult to reach while drying the surface (using dry cloths) of the RAP (RAP conglomerate) at the saturated surface dried (SSD) level as amplified in Figure 5.9. These are likely to have held more moisture compared to those held by the pores in the virgin aggregates. The observed values for water absorption of the coarse portions for the two cases were less than 2% i.e. the maximum set standard for aggregates.

5.8 Estimation of Initial Binder Demand

The Asphalt Institute method (MS-14, 1989) was used for the calculation of binder demand for the proposed mix. Based on the gradation curve in Figure 5.4 (i.e. the black curve in the Figure), the initial binder demand was calculated from an Asphalt Institute empirical formula as follows:

$$P = (0.05A + 0.1B + 0.5C) \times (0.7) \qquad 5.2$$

Where,

Table 5.6: Particle Density and Water Absorption for a Mixture of 20dmm Pen Residual Binder RAP (65%), 5mm Dust (30%), and Mineral Filler (5%)

Description	4mm - 31.5mm	0.063mm – 4mm
Particle Density	2.62(Mg/m^3)	2.37 (Mg/m^3)
Particle Density on Saturated and Surface Dried Basis	2.65(Mg/m^3)	2.46(Mg/m^3)
Apparent Particle Density	2.69(Mg/m^3)	2.61(Mg/m^3)
Water Absorption	1.11 (%)	3.99 (%)

Table 5.7: Particle Density and Water Absorption for a mixture of Virgin Aggregates (95%) and Mineral Filler (5%)

Description	4mm - 31.5mm	0.063mm – 4mm
Particle Density	2.77(Mg/m^3)	2.57 (Mg/m^3)
Particle Density on Saturated and Surface Dried Basis	2.79(Mg/m^3)	2.61(Mg/m^3)
Apparent Particle Density	2.81(Mg/m^3)	2.74(Mg/m^3)
Water Absorption	0.52 (%)	2.28 (%)

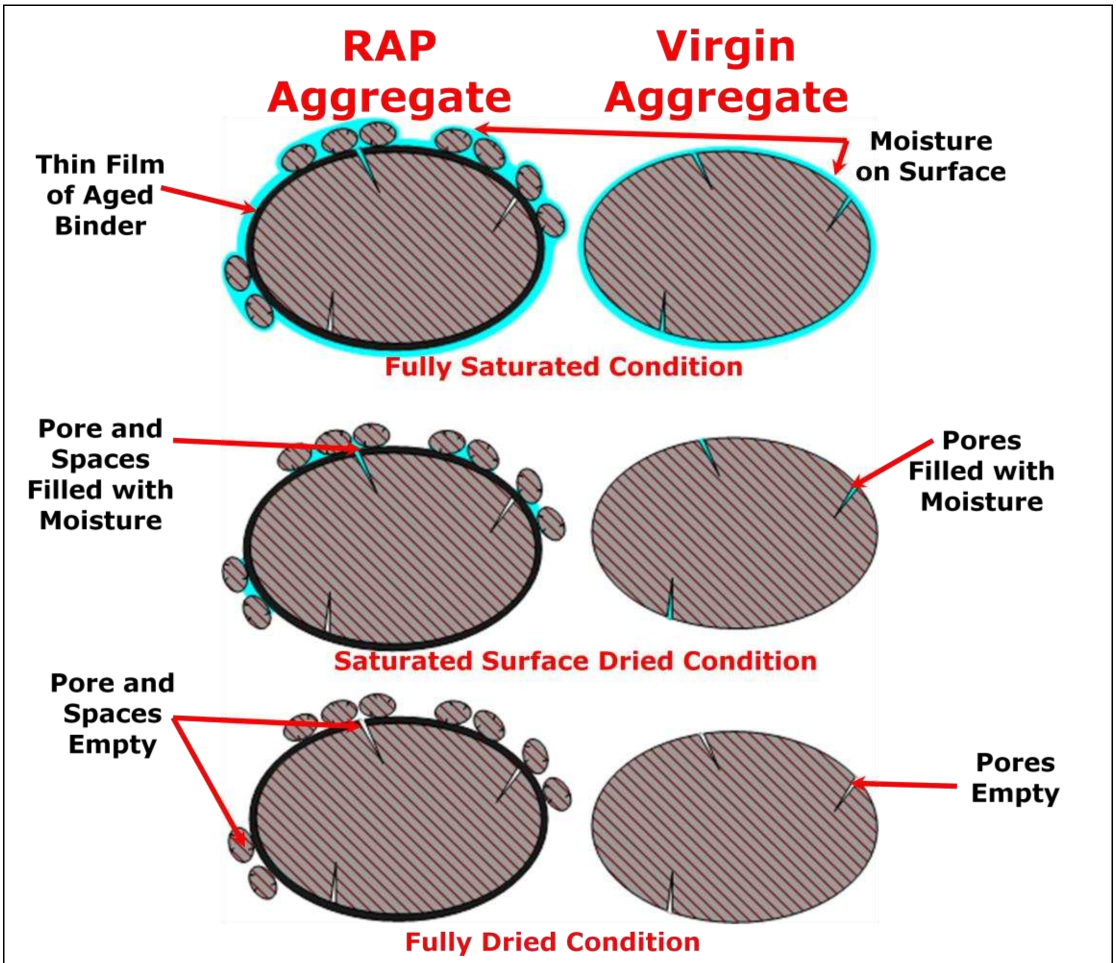


Figure 5.8: Conditions of Substrates (Aggregates & RAP) During Particle Density and Water Absorption Determination

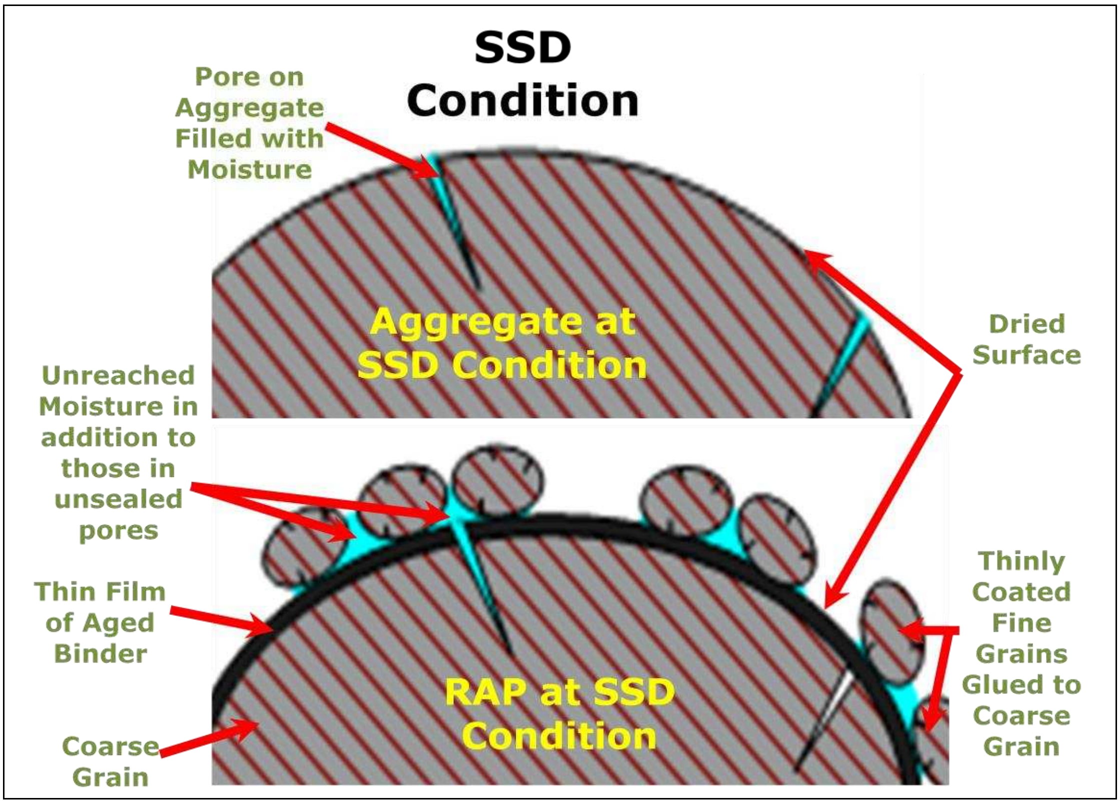


Figure 5.9: RAP Conglomerates Holding more Moisture than Virgin Aggregate at the Saturated Surface Dried (SSD) Condition

P= % by weight of binder demand by mass of total mixture

A= % of mineral aggregate >2.36mm = **100 – 31.8 =68.2**

B=% of mineral aggregate <2.36 and > 0.075mm= **31.8 - 6.4 = 25.4**

C=% of mineral aggregate < 0.075mm= **6.4**

Equation (5.2) was thus evaluated as 6.4%. Meanwhile, since the available emulsion has a bitumen content of 60% , it implies that, taking into consideration the residual binder in RAP (which constitutes 65% of the entire mix), the total initial binder and bitumen emulsion demands are 3.6% and 6.1% (approximated to 6% for convenience) respectively. This value eventually proved useful in the determination of Optimum Total Fluid Content (OTFC) for Cold Bitumen Emulsion Mixtures (CBEMs) as will be found later in this chapter.

5.9 Methods for Specimen Manufacturing

Ibrahim (1998) advised in his work that mixture composition and preparation of test specimens are significant factors affecting the behaviour of cold bituminous emulsion mixtures. So a number of measures were taken to ensure consistency in this laboratory work. For example, there are two approaches for preparing and using cold bituminous emulsion mixtures in the construction industry. These could be mixed and stored until they are needed or mixed and immediately laid. The approach used here was the latter as both time and space for storage were major constraints in this work.

For the laboratory compaction of such mixtures, there were two alternatives available as of the time that this research work commenced - the roller compactor, and the gyratory compactor. The gyratory compactor was opted for since it was capable of directly manufacturing the required cylindrical specimens without introducing any significant modifications to specimens.

The roller compactor produces compacted slabs from which the required cylindrical specimens would have to be cored out. Water jets are normally used as lubricant for the coring and there is the tendency for specimens to be significantly deformed during this process as a result of washing off of fine aggregates or flipping off of coarse aggregates. Cold bituminous emulsion mixtures at this desired stage are very fragile and so this method was not used at the mix design level.

Several methods have been advanced for curing as discussed in Chapter 3 of this thesis. In this exercise, no cyclic curing was done also due to the shortness of time and considering the number of material types (5) that was investigated. The curing regimes did not involve the alteration of temperature nor relative humidity in the course of curing. The specimens were also only sealed while in the mould and unsealed when in the oven. The other factors considered during the laboratory manufacture of specimens are detailed in the succeeding sections.

5.9.1 Mixing

Two types of mixers - Hobart and Sun & Planet were experimented with in this laboratory work. The Hobart mixer was not used beyond the mix design stage because it frequently broke down. This was principally due to the fact that the available 20 Quarts Hobart Mixer was not designed for mixing the gradation (20mm DBM) adopted in this work. The coarse portion of the mix was frequently snagged between the agitator (flat type) and the mixing bowl thus preventing smooth movement of the agitator.

The Sun and Planet mixer effectively worked well during the mixing operation without breaking down. It also had the advantage that mixing temperature could be controlled in it. This eventually proved useful as it afforded the opportunity to investigate the effect of mixing and compacting CBEMs at two different temperatures on stiffness and other engineering properties of cured CBEM cores. The results are later reported in this chapter. Figures 5.10 and 5.11 show the Hobart, and the Sun and Planet Mixers that were used in this work respectively.

5.9.2 Compaction

The Cooper Research Gyratory Compactor available in the NTEC laboratory was used for the compaction of the CBEM materials. This facility served a dual purpose in this work. These were in determining the compactability (compaction characteristics) of mixtures, and manufacture of cores for subsequent tests. In whatever capacity, degree of compaction is required to be close to what obtains in the pavement. Meanwhile, this compactor was found suitable as it was easy to operate and more importantly, density and shear stress values could be read off immediately as the machine compacted. The 100mm steel mould was used for the exercise in order to conserve materials. Although some other parameters were experimented with, as presented later in this chapter, a pressure of 600kPa, 50 gyrations, 30 rev/min and an angle of gyration of 1.25^{o} which have been widely used by researchers (Jenkins, 2000; and Sunarjono, 2008 among others) were employed on the gyratory compactor at the mix design stage. Figure 5.12 shows the gyratory compactor. Since temperature could not be controlled in the compactor, each specimen (already in mould) was first placed in the conditioning

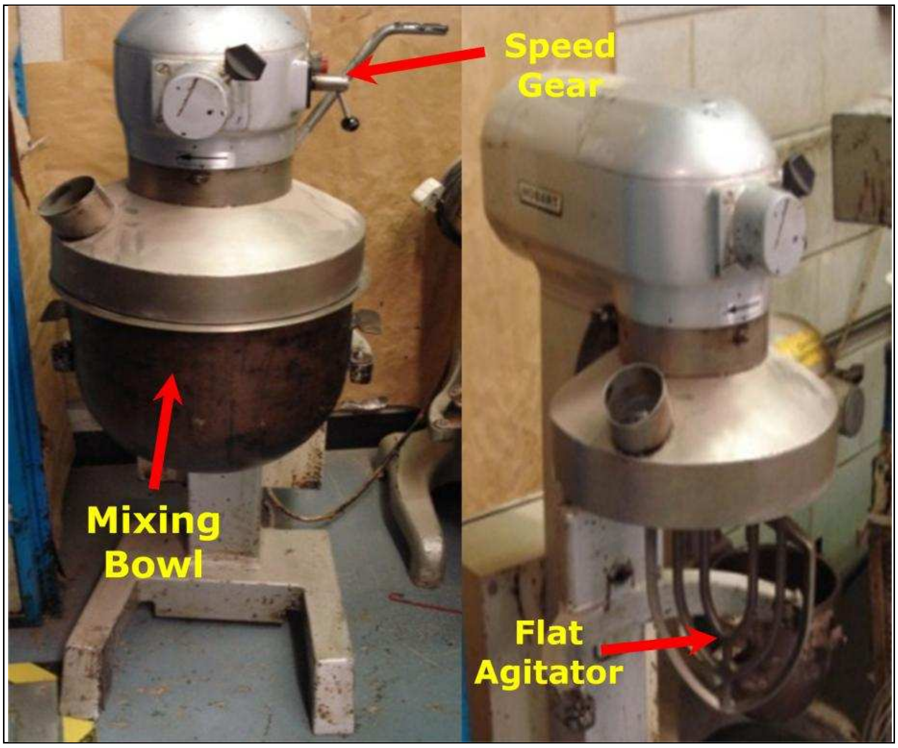


Figure 5.10: 20 Quarts Hobart Mixer

Figure 5.11: The Sun and Planet Mixer

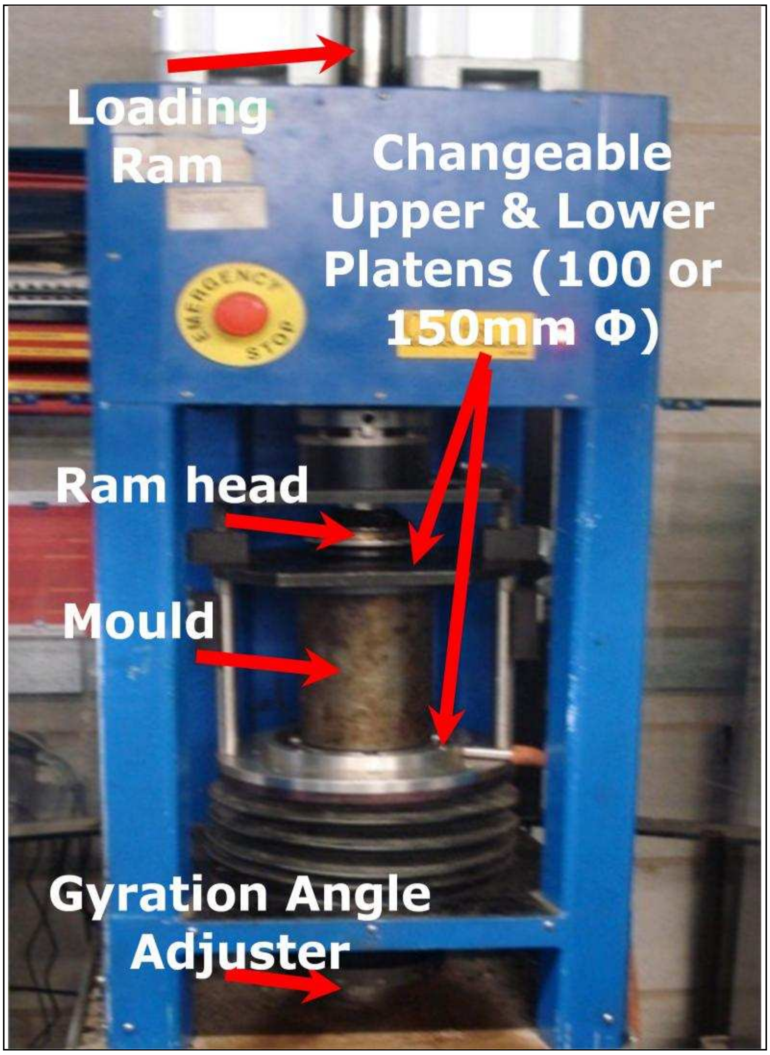


Figure 5.12: Cooper Research Gyratory Compactor

chamber at the desired temperature for about 30mins before compaction.

5.9.3 Curing Method

As discussed in Chapter 3 of this thesis, curing is mandatory for studies on cold mixtures in order to obtain any useful information regarding their mechanical performance. Three curing regimes were eventually used in this work. For this design stage, an intermediate curing condition of 40ºC over 72hrs was used. The curing was done using a forced draft oven. Early life and fully cured conditions of 40ºC over 12hrs and 60ºC over 96hrs were also later applied on specimens at the full investigation stage. The relative humidity was approximately 40% throughout. The protocol followed was such that the specimens were left in the mould (in a sealed condition) immediately after compaction for up to 24hrs after which they were extruded. Specimens were subsequently placed in the forced draft oven at the required temperature for the specified time. Just before and immediately after curing, the specimens' masses were measured. The volumetric properties were measured after curing and then they were placed in the cold store at 5ºC until they were ready for testing.

5.10 Compaction Characteristics and the Determination of OMC and MDD

For cold bituminous emulsion mixtures (CBEMs), evaluating the compaction characteristics of the mix entails determining both the Optimum Moisture Content (OMC) and Maximum Dry Density (MDD) of such. These give an idea of desirable total fluid content in the cold bituminous mixture. The Hobart Mixer (speed II) was used for mixing the materials, while the Cooper Research Gyratory Compactor was used for the compaction of the materials. Having added the specified water content, the mix was mixed for 60s. This compactor automatically displays progressively the height, density and shear stress values of specimens as the machine compacts. The 100mm diameter steel mould was used for the exercise in order to conserve materials. The parameters - loading pressure of 600kPa, 50 gyrations, rate of gyration of 30rev/min and a gyration angle of 1.25º were employed in the gyratory compactor during the exercise.

The moisture contents of the materials were increased at intervals of 2% starting from 1% and ending at 9%, making a total of five levels of observation. The materials were subsequently compacted at room temperature (20ºC). The details in Tables 5.8 (a and b) were used for plotting the accompanying moisture – density graph in Figure 5.13. The dry density of the mixtures as detailed in Table 5.8b was determined using equation 5.3:

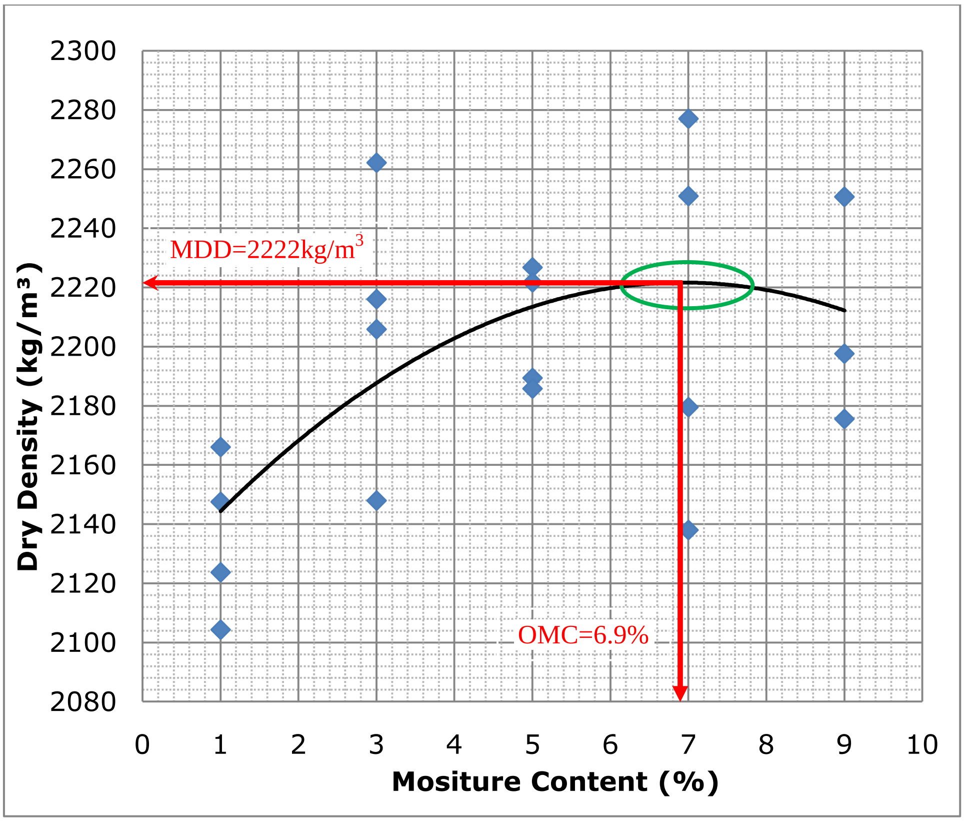


Figure 5.13: Compaction Characteristics of Cold Bitumen Emulsion Mixtures

Table 5.8a: Bulk Density of Compacted Materials as Read off the Gyratory Compactor

Moisture Content (%)	Bulk Density (kg/m^3)			
	Specimen 1	Specimen 2	Specimen 3	Specimen 4
1	2125.40	2187.75	2169.04	2144.98
3	2272.07	2212.43	2282.52	2330.03
5	2338.12	2298.89	2295.12	2332.82
7	2436.43	2287.70	2408.70	2332.12
9	2453.20	2395.43	2371.33	2453.20

Table 5.8b: Dry Density of Compacted Materials as Read off the Gyratory Compactor

Moisture Content (%)	Dry Density (kg/m^3)			
	Specimen 1	Specimen 2	Specimen 3	Specimen 4
1	2104.36	2166.09	2147.56	2123.74
3	2205.89	2147.99	2216.04	2262.17
5	2226.78	2189.42	2185.83	2221.73
7	2277.04	2138.04	2250.89	2179.55
9	2250.64	2197.64	2175.57	2250.64

$$\gamma_d = \frac{100\gamma}{(100+w)} \qquad 5.3$$

Where,

γ = Bulk density (Kg/m^3)

γ_d = Dry density(Kg/m^3)

w = Moisture content (%)

It is obvious from the graph that the OMC is somewhere within the range 6.5 to 7.4% (see the green ellipse in Figure 5.13) while the MDD is 2222kg/m^3. The intermediate value of 6.9% (≈7%) was adopted since one of the philosophies of cold mix design is to minimise the moisture input as much as possible. This should assist in reducing the void content of the mix which is most desirable for durability in bituminous mixtures.

5.11 Determination of the Optimum Total Fluid Content (OTLC) for CBEM

Moisture and binder both have important roles to play in cold bituminous mixtures. Apart from serving as a lubricant which facilitates a dense mixture, moisture assists in activating the needed charges on the surface of the siliceous aggregates (granite in this case) in the mixture which enhance attraction to cationic emulsions (Thanaya, 2003). As the mixture cures, moisture evaporates and the residual bitumen from emulsion starts acting its dual role of binding the medium together and also enhancing a denser medium. Thus, OMC value alone would not suffice for the total fluid content since the mechanism of compaction is slightly different when both moisture and binder are introduced to the mixture (Ibrahim, 1998; and Thanaya 2003) although it indicates the region in which total fluid content lies. These attributes necessitated the determination of the pre-wetting water content required for the proposed mixture before emulsion is added to the mix. Optimum values for both the pre-wetting water and bitumen emulsion contents give an idea of the optimum total fluid content (OTFC).

While keeping the emulsion content constant at 6% by mass of the aggregate (as empirically determined earlier in section 5.8), the pre-wetting water content was increased at intervals of 1.0% by mass of the aggregate starting from total fluid content of 7.0% and ending at 10%. The procedure followed for preparing the specimens was such that the pre-wetting water was first added to the dry batched mixture and mixed using speed II on the Hobart Mixer for 60s. The required emulsion was subsequently added and the mixture mixed for another 60s. These timings were found suitable for such mixes by Sunarjono (2008). Each batch of mixing had sufficient materials for four specimens. The required materials were then placed in 100mm diameter split moulds and subsequently compacted using the Cooper Gyratory Compactor at an ambient temperature of 20ºC, using a pressure of 600kPa, 50 gyrations, rate of gyration of 30 rev/min and a gyration angle of 1.25º. They were left in the moulds for 24hrs (in a sealed condition) at the same ambient temperature before they were carefully extruded. Curing followed immediately at 40ºC over 72hrs in the forced draft oven in an unsealed condition.

Eight (8) specimens were produced for each of the four (4) total fluid contents of 7, 8, 9, and 10%, and the densities of the manufactured specimens are as stated in Table 5.9. Four (4) specimens were tested for indirect tensile stiffness modulus and strength (ITSM @20ºC and ITS @25ºC) in a dry condition according to DD213-1993 ITSM (with target horizontal strain of 3microns for the obvious reason that materials are not fully cured and thus fragile) and BSEN12697-23-2003 respectively, using the Nottingham Asphalt Tester (NAT) and the MAND Uniaxial Compression Testing Machine respectively. The tests were done one after the other starting with ITSM and then followed by the ITS. The remaining four (4) specimens from each level were further subjected to a water sensitivity test at 40ºC over 72hrs in line with BSEN12697-12:2008. The wet specimens were also tested for ITSM and ITS just as was done with the dry specimens.

The obtained results were analysed for indirect tensile stiffness modulus and strength ratios (wet/dry). Figures 5.14 and 5.15 show the relationship between ITSM/ITS and Total Fluid Content, and Dry/Wet Stiffness/Strength Ratios and Total Fluid Content of the tested cold bituminous emulsion mixtures. Tables 5.10 and 5.11 were used respectively for plotting the relationships in these figures. The two figures indicate an Optimum Total Fluid Content (OTFC) of 8% (see red ellipses in the referred figures) and this value was adopted for the next level of the mix design.

5.12 Determination of Optimum Bitumen Emulsion Content (OBEC) in the Optimum Total Fluid Content (OTFC)

Now that the Optimum Total Fluid Content (OTFC) has been established, there is the need to optimise the binder content in the Optimum Total Fluid Content.

Table 5.9: Bulk Densities of Specimens with Varying TFC after Curing at 40°C over 72Hrs

Total Fluid Content (%)	Density (kg/m³)								
	S 1	S 2	S 3	S 4	S 5	S 6	S 7	S 8	**Ave.**
7	2147	2147	2108	2145	2197	2197	2191	2186	**2165**
8	2095	2164	2166	2089	2172	2169	2134	2131	**2140**
9	2188	2183	2120	2155	2199	2158	2144	2144	**2162**
10	2050	2136	2078	2130	2125	2128	2179	2130	**2119**

Note, S= Specimen

Table 5.10: Stiffness and Strength Values of Cold Bituminous Emulsion Mixtures with Varying TFC

Total Fluid Content (%)	**ITSM (MPa)**		**ITS (kPa)**		**ITSMR (%)**	**ITSR (%)**
	Dry	**Wet**	**Dry**	**Wet**		
7	2097	492	434.85	192.26	23.5	44.2
7	1857	592	514.51	212.57	31.9	41.3
7	2215	481	522.92	187.37	21.7	35.8
7	1993	563	565.81	229.64	28.2	40.6
8	1783	569	503.56	243.94	31.9	48.4
8	2273	636	565.81	233.79	28.0	41.3
8	1967	555	514.62	160.42	28.2	31.2
8	2677	847	517.21	265.38	31.6	51.3
9	1674	534	505.85	214.95	31.9	42.5
9	1869	532	454.93	214.72	28.5	47.2
9	2508	495	547.83	221.34	19.7	40.4
9	1744	508	506.32	199.21	29.1	39.3
10	1802	388	365.16	145.83	21.5	39.9
10	1966	412	463.44	145.97	21.0	31.5
10	1846	578	440.17	189.53	31.3	43.1
10	1928	586	433.00	206.13	30.4	47.6

Table 5.11: Dry and Wet Strength/Stiffness Ratio Values for CBEMs with Varying TFC

Total Fluid Content (%)	ITSR (%)	ITSMR (%)
7	40.3	26.1
8	43.0	30.0
9	42.2	26.5
10	40.4	26.0

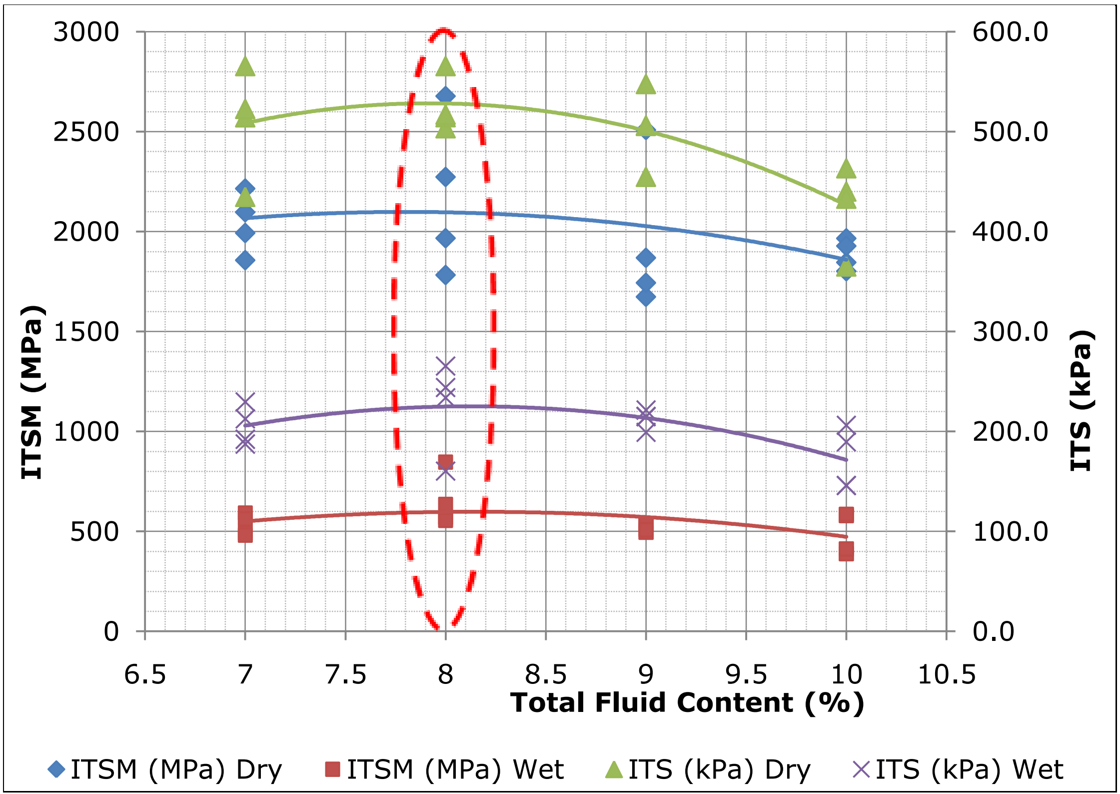


Figure 5.14: Strength/ Stiffness Characteristics of CBEMs with Varying Total Fluid Content

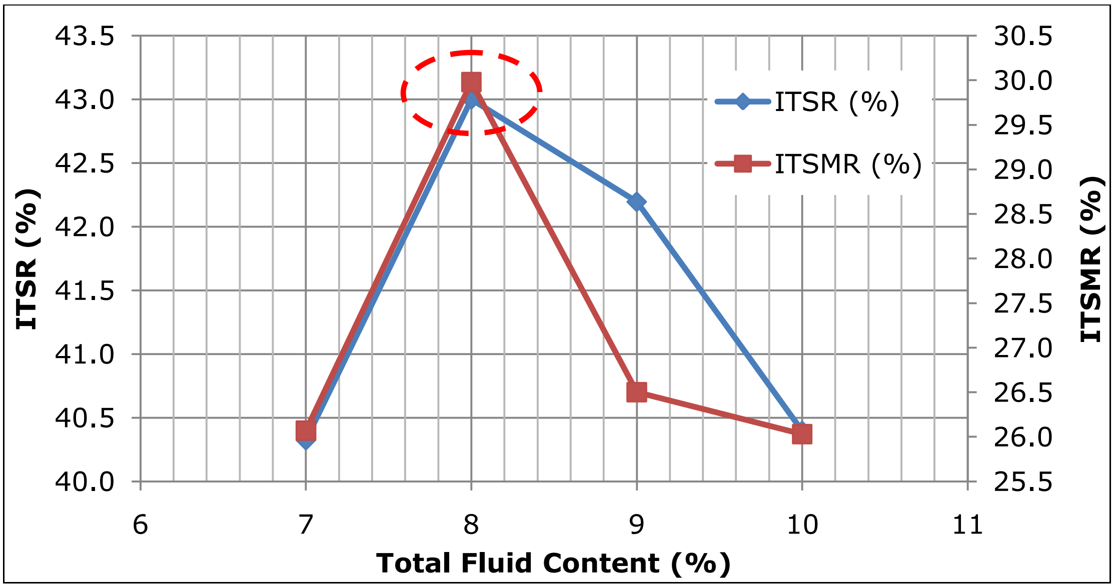


Figure 5.15: Dry and Wet Strength/Stiffness Ratios for CBEMs with Varying Total Fluid Content

Ideally, the best way of establishing this is by varying the bitumen/water ratio in the emulsion. However, the emulsion used is a proprietary emulsion, and it was difficult to go along this line. The alternative which was adopted involved keeping the TFC constant while varying the emulsion/ pre-wetting water content ratio. The procedure in section 5.10 was followed for preparing and testing all the thirty two (32) specimens except that the bitumen emulsion contents were varied at an interval of 1% starting from 4.5% and ending at 7.5% making a total of four (4) levels of observation.

While the bulk densities of the specimens are as shown in Table 5.12, Figures 5.16 and 5.17 show the relationships between ITSM/ITS and TFC, and Dry/Wet Stiffness/Strength Ratios and TFC of the tested CBEMs. The values in Tables 5.13 and 5.14 were respectively used for plotting the relationships in the said figures. Whereas, in the preceding section there was an agreement between the ITS, ITSM, ITSR and ITSMR values in determining the OTFC, in this case it was less clear.

From Figure 5.16, the $ITSM_{dry}$ curve indicated optimum emulsion content at 5.5% while the ITS_{dry} showed an optimum at 6.5%. However, ITSMR, the $ITSM_{wet}$ and the ITS_{wet} followed similar trends of increasing with an increase in emulsion content which by implication meant increase in binder content. This is expected as the more binder that is introduced to the mix, the less susceptible the mixture is to moisture. The ITSR curve was a bit different as it initially showed an optimum at 5.7% emulsion content, but immediately after emulsion content of 6.5%, it started following the same trend as those of ITSMR the $ITSM_{wet}$ and the ITS_{wet} respectively. The decision for the optimum value was subsequently based on ITS_{dry}. This appears logical as the $ITSM_{dry}$ at this level is greater than 2000MPa (Merril et al (2004) suggested a minimum of 1900MPa as long term design stiffness for bitumen bound recycled material for well compacted class B1 cold mix materials) compared to the $ITSM_{dry}$ optimum at which level the ITS_{dry} was almost at its lowest. More importantly, a number of researchers have found the ITS test the most indicative for making such decisions (Jenkins, 2000; and Kim et al, 2007) since it is a highly sensitive test in which specimens are loaded to failure.

This decision implies that the bitumen emulsion and pre-wetting water content adopted and which were subsequently used in this work were 6.5% and 1.5% respectively. Also, the aggregates composition used henceforth in this work was of the ratio 65:30:5 for RAP, 5mm dust and mineral filler respectively unless otherwise stated. Figure 5.18 shows these proportions pictorially.

Table 5.12: Bulk Densities of CBEMs with varying Emulsion Content after Curing at 40°C over 72Hrs

Emulsion Content (%)	Density (kg/m³)								
	S 1	S 2	S 3	S 4	S 5	S 6	S 7	S 8	Ave
4.5	2150	2158	2107	2115	2107	2076	2101	2104	**2115**
5.5	2052	2164	2123	2109	2126	2131	2161	2115	**2123**
6.5	2134	2097	2180	2177	2183	2092	2137	2126	**2141**
7.5	2150	2197	2153	2191	2153	2108	2147	2142	**2155**

Note, S= Specimen

Table 5.13: Stiffness and Strength Values of CBEMs with Varying Emulsion Content

Bitumen Emulsion Content (%)	ITSM (MPa)		ITS (kPa)	
	Dry	Wet	Dry	Wet
4.5	2325	497	424.70	124.97
4.5	1432	466	310.06	116.89
4.5	2028	475	336.69	129.98
4.5	1795	559	316.94	108.32
5.5	1804	596	299.09	148.03
5.5	2466	535	336.31	145.32
5.5	2488	695	415.67	142.61
5.5	2473	681	351.12	177.08
6.5	1791	655	438.56	154.68
6.5	2675	762	454.21	198.28
6.5	2018	666	407.90	156.78
6.5	1879	663	399.87	187.75
7.5	1484	840	425.34	220.25
7.5	1405	751	466.66	251.78
7.5	1856	669	362.37	196.77

Table 5.14: Dry and Wet Strength/Stiffness Ratio Values for CBEMs with Varying Emulsion Content

Bitumen Emulsion Content (%)	ITSMR (%)	ITSR (%)
4.5	26.3	34.6
5.5	27.1	43.7
6	30.0	43.0
6.5	32.8	41.0
7.5	45.2	53.2

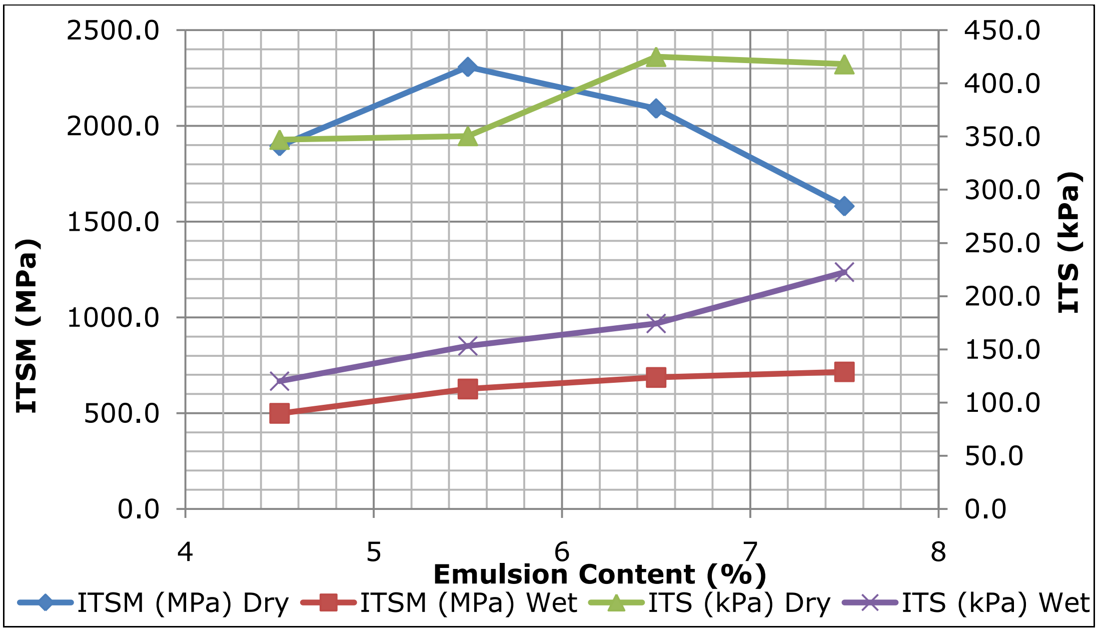


Figure 5.16: Strength/ Stiffness Characteristics of Cold Recycled Bitumen Emulsion Mixtures with Varying Bitumen Emulsion Content

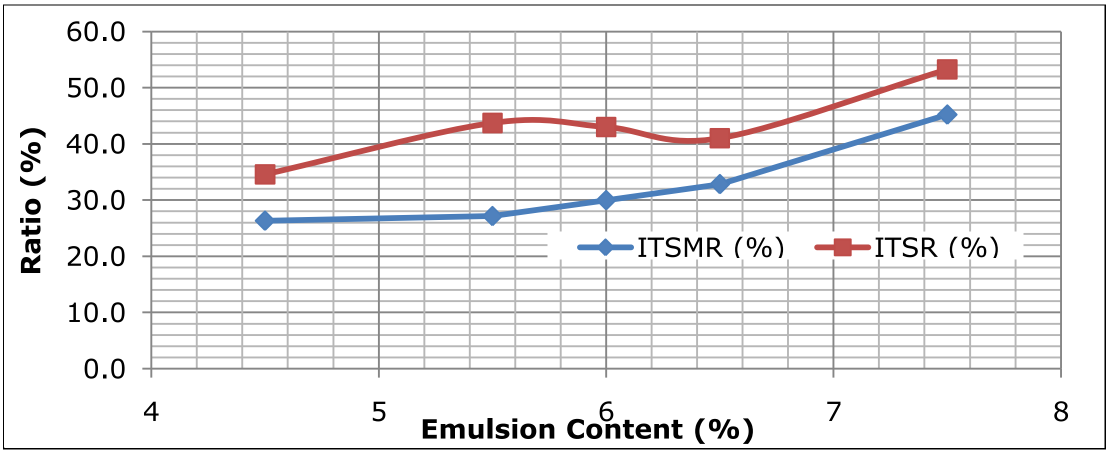


Figure 5.17: Dry and Wet Strength/Stiffness Ratios CBEMs with Varying Bitumen Emulsion Content

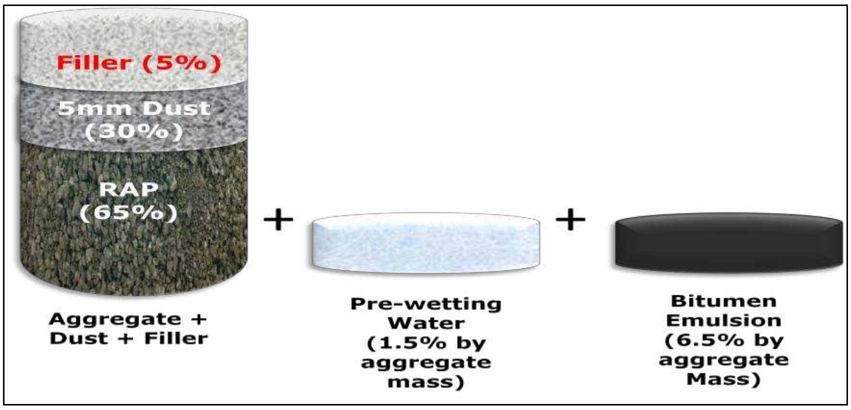


Figure 5.18: Mix Proportion for the Cold Bitumen Emulsion Mixture (CBEM)

5.13 Initial Exploration of Mechanical Properties of the Designed Cold Bituminous Emulsion Mixtures (CBEMs)

A number of investigations to evaluate the characteristics of the CBEMs due to the effect of certain factors which could adversely affect the performance of the produced mixtures were investigated. The following sections detail the findings of these investigations. The RAP with residual bitumen of 20dmm was used here, except where otherwise stated. While other parameters remained as for the mix design stage, it is worth noting that conditioning of the specimens for water susceptibility was done by first vacuum saturating the specimens at a pressure of about 140mbar for about 10minutes followed by soaking in water at a temperature 25ºC for 24hrs as suggested by Jenkins (2000) and Sunarjono (2008). This was considered helpful as most of the virgin aggregate CBEMs that were later discussed in Section 5.13.4 had become very weak with stiffness values in some cases just a little above 200MPa. All the mechanical tests were conducted at 20ºC on the NAT.

5.13.1 Determination of maximum density of CBEMs

Before any further investigations were made beyond this point, the maximum (Rice) densities of the mixtures (RAP and Virgin aggregate alike) were determined after BSEN12697-5:2002. This was needed in order to properly account for the void content of the produced mixtures henceforth. Tables 5.15 (a) and (b) detail the maximum density for the RAP and Virgin Aggregate CBEMs respectively.

5.13.2 The Effect of mixing time on the compactability and stiffness properties of CBEMs

As earlier mentioned in this chapter the need to change to the Sun and Planet Mixer was inevitable due to the inability of the Hobart Mixer to cope with the gradation for the design mix. The Sun and Planet Mixer was thus investigated for the appropriate mixing times for both the pre-wetting water and bitumen emulsion addition to the mix. Although temperature could be controlled on this mixer, it has only one speed. The experience of Ibrahim (1998) on this mixer with similar mixtures gave a head start for the investigation. He generally used a mixing time of 120s for pre-wetting water while he used times ranging from 90s to 120s for mixing bitumen emulsion (depending on the nature of the emulsion). Therefore, pre-wetting water mixing times paired with the following corresponding mixing times for bitumen emulsion were used respectively: 90s, 90s; 90s, 120s and; 120s, 120s.

The pre-wetting water was added to the mix from the onset of mixing. After

Table 5.15a: Maximum Density for the RAP CBEMs

Maximum Density kg/m³				
Test 1	Test 2	Test 3	Test 4	**Average**
2529.76	2509.28	2508.13	2515.23	**2516**

Table 5.15b: Maximum Density for the Virgin Aggregate CBEMs

Maximum Density kg/m³		
Test 1	Test 2	**Average**
2627.99	2601.25	**2615**

mixing halfway into each specified time (i.e. pre-wetting water and bitumen emulsion mixing times) the mixer was stopped to allow for manual stirring using spatulas before the mixing was completed. This further ensured uniform distribution of the fluids.

Except for mixing time, and the target number of gyrations of 200, the cylindrical cores were produced and cured similarly as in Section 5.11 of this chapter. Figure 5.19 and Table 5.16 both detail the results of the investigation. The results indicate that the mixing time pair of 120s, 120s for pre-wetting water and bitumen emulsion respectively gave the best stiffness response and the lowest air void content whether in dry (cured) condition or wet condition. The pair of 90s, 90s for mixing the pre-wetting water and bitumen emulsion with aggregates gave the lowest stiffness value and highest air voids content. However the moisture loss trend throughout the range upon curing was observed to be similar.

The difference in stiffness values for the 90s, 90s pair and the 90s, 120s pair was quite large i.e. 568MPa (which also represents a 36.1% improvement over the stiffness of the former) compared to that between 90s, 120s pair and the 120s, 120s pair, i.e. 142MPa (which similarly represents a 6.6% improvement over the stiffness of the former). While the results indicate that a paired mixing time of 90s, 90s for both pre-wetting water and bitumen emulsion respectively was inadequate, it was obvious that increasing the paired mixing time beyond 120s, 120s might not be beneficial as the difference between the stiffness values at this point and those of the 90s, 120s pair were not that significant. It is thus assumed that the 120s, 120s pair is the optimum (or very close to the optimum value). Going beyond this level could have induced emulsion breaking and subsequent inhibition of proper packing during compaction.

This investigation shows that CBEMs are very sensitive materials as a difference of 30s during mixing could make a marked difference to their stiffness and compactability properties especially in this case where the breaking of the bitumen emulsion is a physico-chemical process. Although this is fully investigated using statistical tools later in Chapter 6 of this thesis, there is an indication here that stiffness is influenced by air voids content. As air voids increase, stiffness modulus decreases and vice versa. The investigation summarily informs of the need to apply just enough mixing time for both the pre-wetting water and bitumen emulsion in order to enhance the performance of CBEMs. At such an optimum level, it is believed that the materials have been properly coated (optimum degree of aggregate wetness achieved) and also the emulsion in the mix has not started breaking which are desirable for performance.

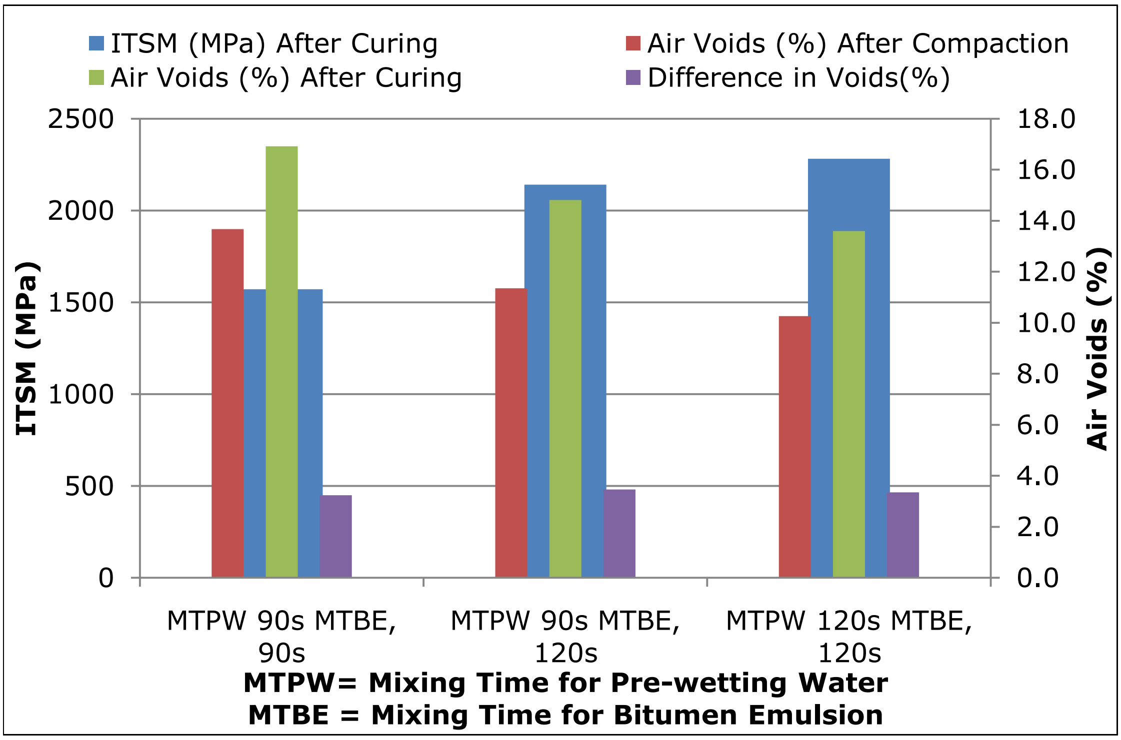


Figure 5.19: The Effect of Mixing Time on the Stiffness Modulus of CBEMs

Table 5.16: The Effect of Mixing Time on the Stiffness Modulus of CBEMs

MTPE + A (s)	MTBE + MTPW & A (s)	Wet Air Voids (%)	Dry Air Voids (%)	Ave Wet Air Voids (%)	Ave Dry Air Voids (%)	ITSM (MPa)	Average ITSM (MPa)
90	90	14.3	17.3	13.7	16.9	1638	1572
90	90	13.2	16.6			1718	
90	90	13.5	16.8			1359	
120	120	9.8	13.6	10.3	13.6	1748	2282
120	120	9.9	13.0			2323	
120	120	11.1	14.2			2775	
90	120	11.3	15.0	11.3	14.8	2143	2140
90	120	10.6	14.3			2152	
90	120	11.0	14.6			2088	
90	120	12.5	15.3			2179	

MTPW= Mixing Time for Pre-wetting Water A = Aggregates
MTBE = Mixing Time for Bitumen Emulsion

5.13.3 The effect of filler type, and mixing and compaction temperature on compactability and stiffness properties of CBEMs

Up till this point in this study, only limestone filler was used due to the fact that it was the only filler type that was available when the laboratory work started. Also, mixing and compaction activities have both been carried out at 20ºC (ambient temperature) since temperature could not be controlled on the Hobart Mixer which was used up till this point. However, replacement of the limestone filler with granite as soon as it was made available was considered necessary as granite fillers are always used for asphalt mixtures in Nigeria since they are abundantly available. Similarly, introducing a slightly elevated temperature of 32ºC for the mixing and compaction processes was thought necessary since the average maximum air temperature for Ibadan a typical tropical City in Nigeria has been selected for this research work, which is 32ºC (BBC Weather, 2008). As earlier noted, the Sun and Planet mixer had an advantage of temperature control while in operation. It was envisaged that the introduction of these new measures should simulate road pavement construction practice in Nigeria. Except for a fixed target number of gyrations of 200 applied and adoption of a mixing time of 120s for pre-wetting water and 120s for bitumen emulsion addition, all other parameters used in the previous section remained the same here.

The graph in Figure 5.20 and details in Table 5.17 clearly indicate that the CBEMs containing granite filler generally had higher dry stiffness properties than those with limestone filler although the latter performed much better both in wet stiffness and compactability properties. These observations were irrespective of the compaction and mixing temperatures used. The CBEMs with limestone filler demonstrated a good ITSMR with the observed values clearly above average compared to those containing granite filler which had values generally below average. This observation for ITSMR is in conformity with other researcher's findings on the good anti-stripping properties of limestone filler in hot asphalt mixtures (Airey et al, 2005). However, it is worth noting that irrespective of the filler type used and the temperature of mixing and compaction, moisture loss was similar just as observed in the preceding section. This implies that the degree of curing is not affected by filler type in the CBEMs studied.

Similarly, the effect of mixing and compaction temperature on the compactability and stiffness responses of CBEMs irrespective of the filler type used were studied just as indicated in Figure 5.20 and the accompanying Table 5.17. Mixing and compaction of CBEMs at 32ºC gave better responses in stiffness and compactability compared to 20ºC. For example, the granite filler CBEMs prepared

Table 5.17: Effect of Filler Type, Compaction and Mixing Temperature on Stiffness Properties of CBEMs

Filler Type	Mixing & Comp. Temp	(a) Wet Air Voids (%)	(b) Dry Air Voids (%)	(a) - (b) Ave (%)	ITSM Dry (MPa)	ITSM Dry Ave (MPa)	ITSM Wet (MPa)	ITSM Wet Ave (MPa)	ITSMR (%)
Limestone Filler	20°C	11.6	14.8	3.3	2424	2261	1427	1302	57.6
		12.5	15.7		2256		1165		
		11.4	15.1		2130		1217		
		11.0	14.3		2236		1401		
	32°C	6.7	10.0	3.4	2602	2613	1651	1816	69.5
		8.2	11.4		2660		1661		
		6.6	9.9		2586		1865		
		6.1	9.6		2606		2087		
Granite Filler	20°C	11.1	14.5	3.4	2621	2368	997	1023	43.2
		13.5	16.5		2291		986		
		11.6	15.0		2055		994		
		10.1	13.8		2505		1116		
	32°C	7.9	11.3	3.4	3272	2995	1354	1420	47.4
		9.0	12.3		3082		1491		
		7.3	10.8		2790		1327		
		6.2	9.5		2837		1506		

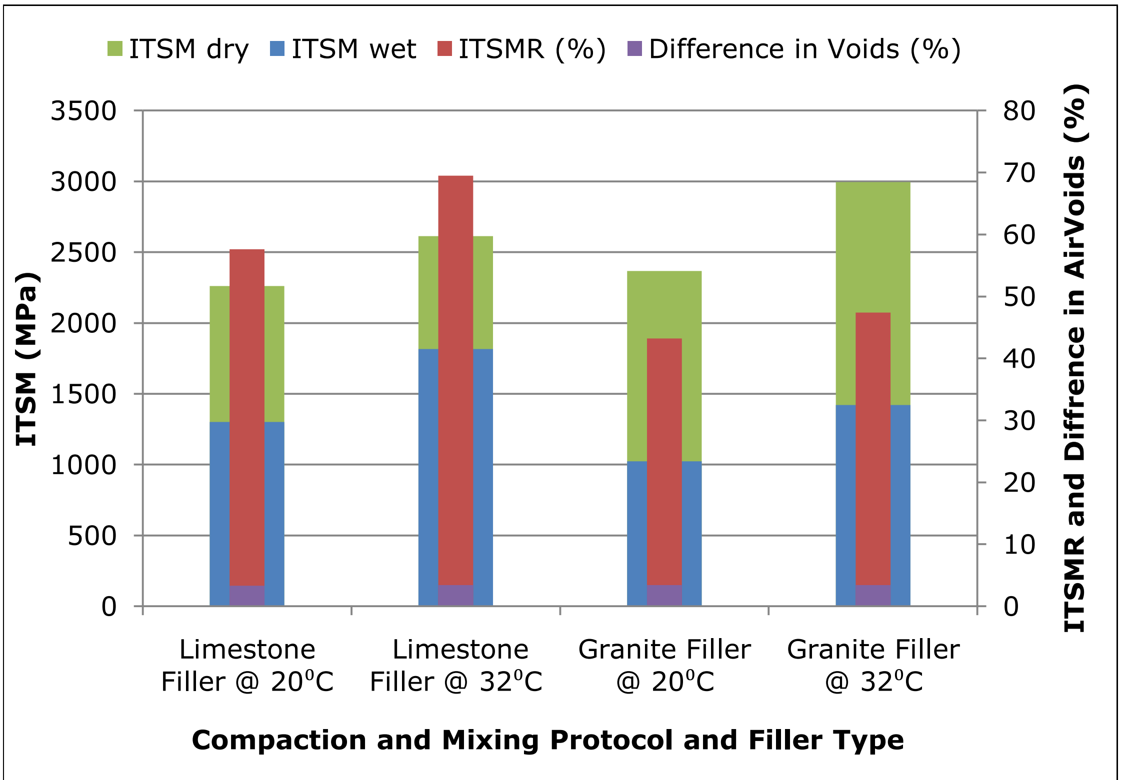


Figure 5.20: Effect of Compaction/Mixing Temperature and Filler Type (Note: same compactive effort of 200gyrations @ 600kpa pressure was applied)

at 32ºC gave improvements of 26.5%, 38.8% and 9.7% for dry stiffness modulus, wet stiffness modulus and ITSMR values respectively over those prepared at 20ºC. The observed values for limestone filler CBEMs were similar as improvements of 15.6%, 39.5% and 11.6% for dry stiffness modulus, wet stiffness modulus and ITSMR values respectively were noted for those prepared at 32ºC over those prepared at 20ºC. Irrespective of the temperature of mixing and compaction, moisture loss was similar just as observed in the preceding section. This also implies that the curing protocol used here was not affected by the temperature at which the CBEMs studied were prepared.

These performances were also observed for compactability of the CBEMs although a reduction here implies enhancement. For limestone filler CBEMs there were reductions of 40.9% and 31.6% reduction in wet and dry air voids contents respectively for those CBEMs prepared at 32ºC compared to those at 20ºC. In the case of granite filler CBEMs, the reductions observed were 38.8% and 31.0% reduction in wet and dry air voids contents respectively for those CBEMs prepared at 32ºC compared to those at 20ºC. These reductions could have been a result of the residual bitumen in the bitumen emulsion becoming less viscous at 32ºC compared to 20ºC as noted in section 5.5.1 of this chapter. Apart from enhancing the lubricating effect of the bitumen emulsion which subsequently facilitates good packing of the substrates (aggregates), it is also believed that at such temperature, the residual bitumen in the RAP is getting excited (active) and possibly rejuvenated by the fairly soft residual bitumen in the emulsion. Although the extent of this possible rejuvenation cannot be ascertained, Karlsson (2002) observed in his work that, diffusion which is enhanced by temperature is one of the key factors that facilitate the rejuvenation of aged bituminous binders. Possible changes in the volumetrics which subsequently enhanced compactability of the mixtures could also have been responsible for these observations.

Subsequent to these observations, the decision to use granite filler for producing CBEMs was adopted for the detailed investigation reported in Chapter 6 in order to properly represent what obtains in the road construction industry in Nigeria although it was obvious from this study that limestone fillers performed better. Also these investigations showed that slightly elevated temperature of 32ºC for mixing and the compaction of CBEMs resulted in better mixtures compared to those at 20ºC. This is expected to be a plus for cold recycling practice if adopted in a hot tropical country such as Nigeria. It similarly gives an indication that it is beneficial if such mixtures are prepared and laid at such temperatures in the temperate climates where cold recycling is already being practiced.

5.13.4 The effect of applied compactive effort on stiffness properties of CBEMs

CBEMs have been observed by researchers (Ebels, 2008; Ibrahim, 2008; Jenkins, 2000; Sunarjono, 2008; Thanaya, 2003; and Twagira, 2010) as materials that are difficult to compact compared to hot mixtures. These researchers are however of the opinion that when either the pressure or the gyration angle are increased individually or combined, the specimen reaches the desired density quicker. However, there is the possibility that some other important properties of the mixture could have been compromised in the process. An effort was thus made to study the effect of changing these two parameters on the stiffness (wet and dry) of the CBEMs. Along with CBEMs containing RAPs with residual binders of 20dmm penetration, the virgin aggregate mixture was studied here. While every other parameter holds for the virgin aggregate mixture, the aggregate proportion changed to 95% virgin aggregate and 5% filler.

It is worth restating here that the following parameters have been used on the Cooper Research Gyratory Compactor up to this point:

- Pressure – 600kPa
- Gyration angle – 1.25º
- Rate of Gyration - 30 rev/min
- Mould Size – 100mm diameter
- Target Number of Gyration – 50 (for design stage), 200

A pressure of 800kPa (maximum attainable) and gyration angles of 1.5º and 2.0º were thus explored along with those previously used. The effect of similar compactive effort on the stiffness and moisture susceptibility of CBEMs are shown in Figure 5.21 and Table 5.18. From Figure 5.21, it is obvious that the virgin aggregate CBEMs were only able to attain better $ITSM_{dry}$ values than the RAP CBEMs at a greater angle of gyration (1.5º) and later at a higher compactive pressure (800kPa). Irrespective of the property of the compactive effort (angle of gyration and pressure), the RAP CBEMs had better $ITSM_{wet}$ which is most desired for pavement materials being a major indicator of durability. This logically enhanced the ITSMR of the RAP CBEMs, almost doubling that of the virgin aggregate CBEMs at some points. Similarly, the result shows the superior positive effect of compaction at 600kPa and 1.25º on the ITSMR over those of higher compactive effort (higher angle of gyration and pressure). More importantly, having been treated similarly, the RAP CBEMs had lower void content than the virgin aggregate CBEMs. Low void content (porosity) has been widely regarded as

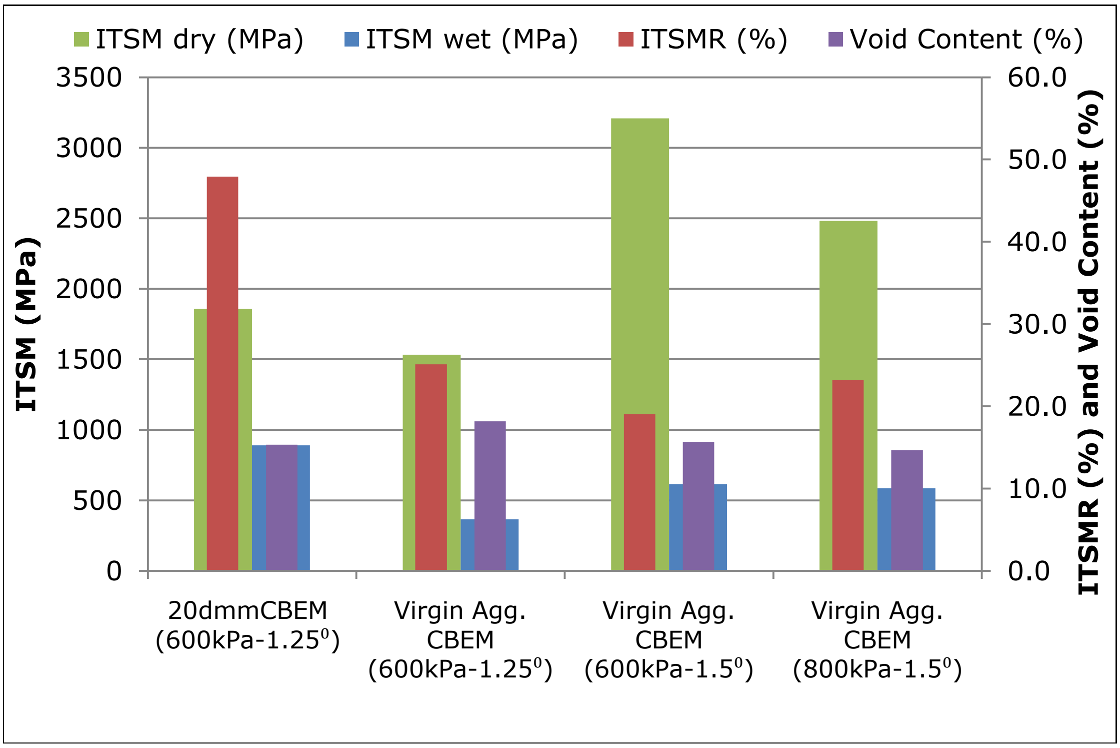


Figure 5.21: The Effect of Similar Compactive Effort on the Stiffness and Moisture Susceptibility of CBEMs

Table 5.18: Properties of CBEMs with Similar Compactive Effort

Material Description	ITSM dry (MPa)	ITSM wet (MPa)	ITSMR (%)	No. Gyros	Density (kg/m³)	Void Content (%)
20dmmCBEM (600kPa-1.25)	1720	756	44.0	200	2159	14.2
	2197	1064	48.4	200	2122	15.7
	1652	849	51.4	200	2110	16.1
Average	**1856**	**889**	**47.9**	**200**	**2130**	**15.3**
Virgin Aggregate CBEM (600kPa-1.5°)	3437	714	20.8	200	2237	14.4
	3001	433	14.4	200	2183	16.5
	3189	699	21.9	200	2192	16.1
	3209	**615**	**19.0**	**200**	**2204**	**15.7**
Virgin Aggregate CBEM (800kPa-1.5°)	2097	335	16.0	200	2237	14.4
	2867	687	24.0	200	2227	14.8
	2371	561	23.6	200	2227	14.8
	2591	756	29.2	200	2230	14.7
Average	**2482**	**585**	**23.2**	**200**	**2230**	**14.7**
Virgin Aggregate CBEM (600kPa-1.25°)	1154	253	21.9	200	2128	18.6
	1474	401	27.2	200	2134	18.4
	2301	405	17.6	200	2164	17.2
	1201	405	33.7	200	2131	18.5
Average	**1533**	**366**	**25.1**	**200**	**2139.3**	**18.2**

a critical factor that enhances the durability of such CBEMs.

Also, the effect of similar void contents on the stiffness and water susceptibility of CBEMs was briefly examined. Figure 5.22 and Table 5.19 refer. Overall, the RAP CBEMs performed better in terms of $ITSM_{dry}$, $ITSM_{wet}$ and ITSMR irrespective of the properties of the applied compactive effort (i.e. angle of gyration and pressure). In order to attain the same void content, the virgin aggregate CBEMs required more than double the compactive effort used for the RAP CBEMs even while both were being compacted at 800kPa and 1.5^o.

Figure 5.23 and Table 5.20 detail the effect of varying void contents on the stiffness and water susceptibility properties of the CBEMs (RAP and virgin aggregate alike). Overall, as void content decreases (see trend line in purple in Figure 5.23), $ITSM_{dry}$ increases (see trend line in blue Figure 5.23). This trend also consequentially holds for $ITSM_{wet}$. Although this trend holds for ITSMR to a point, when void content is excessively high the ITSMR increases significantly.

5.13.5 The effect of curing protocol on CBEMs

The effect of curing protocol on the stiffness property of the CBEM was studied here mainly to ascertain those that will be appropriate for further studies on the early life, intermediate and fully cured conditions. As previously discussed in Chapter 3, many protocols have been developed but there is no universally accepted one. For this investigation, protocols involving temperatures of 40^{o}C over various hours and 60^{o}C over 96hours have been investigated.

Figure 5.24 and Table 5.21 are both relevant here. It is obvious from Figure 5.24 that stiffness of the CBEMs improved with curing time for those cured at 40^{o}C. Although the plan was to keep the curing temperature for subsequent mixtures at 40^{o}C mainly because this is very close to road pavement temperatures for road bases in the tropics as earlier discussed in this chapter, but the stiffness values obtained even after long curing time were still below expectation. Curing at 60^{o}C over 96hrs however gave a better performance in stiffness (3000+MPa) compared to the CBEMs cured at 40^{o}C over 624hrs (2700+MPa).

The curing protocol at 60^{o}C looked appealing for the shortness of time involved, but there was also the concern that CBEMs cured at such high temperatures could have been aged. Thus a binder recovery was carried out on the CBEMs prior to curing. It was similarly done for those cured at 40^{o}C over 624Hrs (26days) and 60^{o}C over 96hrs. The physico-chemical properties of the recovered binders were

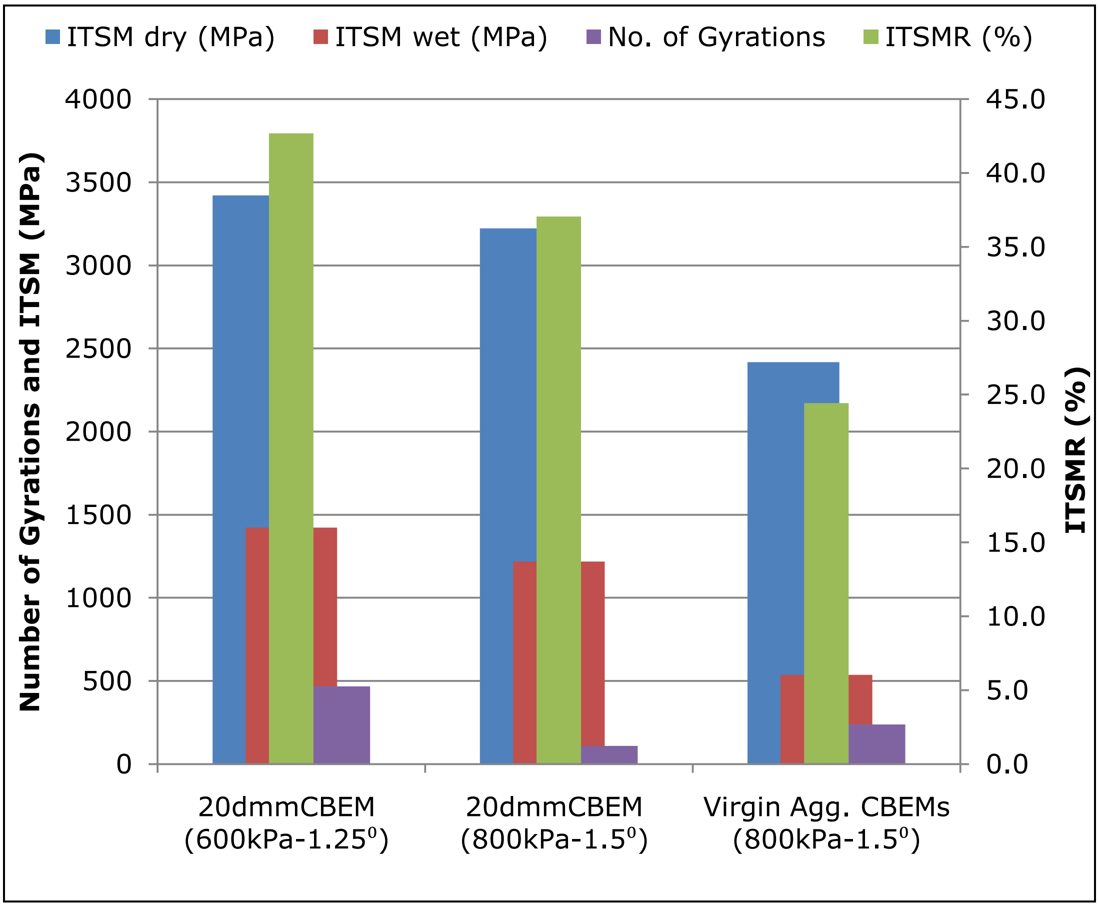


Figure 5.22: The Effect of Similar Void Content on the Stiffness and Water Susceptibility of CBEMs

Table 5.19: Properties of CBEMs with Similar Void Contents

Material Description	ITSM dry	ITSM wet	ITSMR (%)	No. Gyros	Density (kg/m^3	Void Content (%)
Virgin Aggregate CBEM (800kPa-1.5°)	1133	373	32.9	500	2306	11.8
	3121	586	18.8	154	2253	13.8
	2999	647	21.6	62	2248	14.0
Average	**2418**	**535**	**24.4**	**239**	**2269**	**13.2**
20dmmCBEM (600kPa-1.25	3436	1444	42.0	45	2282	9.3
	3378	1396	41.3	521	2189	13.0
	4002	1217	30.4	619	2220	11.8
	2867	1635	57.0	681	2189	13.0
Average	**3421**	**1423**	**42.7**	**467**	**2220**	**11.8**
20dmmCBEM (800kPa-1.25)	2903	1058	36.4	28	2189	13.0
	3835	1758	45.8	192	2288	9.1
	2868	733	25.5	55	2134	15.2
	3281	1325	40.4	164	2237	11.1
Average	**3222**	**1218**	**37.0**	**110**	**2212**	**12.1**

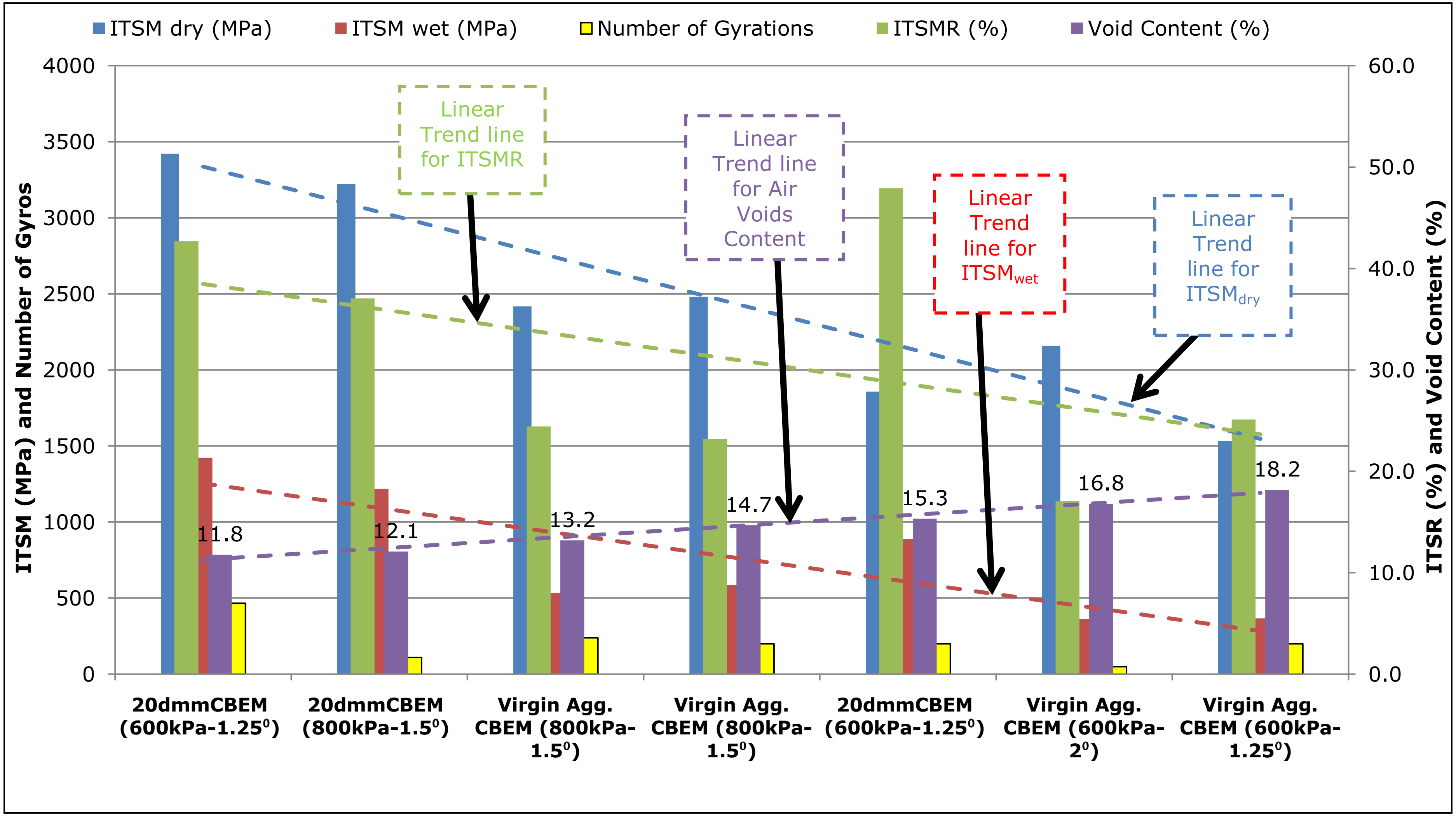


Figure 5.23: The Effect of Varying Void Content on the Stiffness and Water Susceptibility of CBEMs

Table 5.20: Properties of CBEMs with Varying Void Contents

Material Description	ITSM dry	ITSM wet	ITSMR (%)	No. Gyros	Density (kg/m^3	Void Content (%)
20dmmCBEM (600kPa-1.25)	3436	1444	42.0	45	2282	9.3
	3378	1396	41.3	521	2189	13.0
	4002	1217	30.4	619	2220	11.8
	2867	1635	57.0	681	2189	13.0
Average	**3421**	**1423**	**42.7**	**467**	**2220**	**11.8**
20dmmCBEM (800kPa-1.5)	2903	1058	36.4	28	2189	13.0
	3835	1758	45.8	192	2288	9.1
	2868	733	25.5	55	2134	15.2
	3281	1325	40.4	164	2237	11.1
Average	**3222**	**1218**	**37.0**	**110**	**2212**	**12.1**
20dmmCBEM (600kPa-1.25)	1720	756	44.0	200	2159	14.2
	2197	1064	48.4	200	2122	15.7
	1652	849	51.4	200	2110	16.1
Average	**1856**	**889**	**47.9**	**200**	**2130**	**15.3**
Virgin Aggregate CBEM (800kPa-1.5°)	1133	373	32.9	500	2306	11.8
	3121	586	18.8	154	2253	13.8
	2999	647	21.6	62	2248	14.0
Average	**2418**	**535**	**24.4**	**239**	**2269**	**13.2**
Virgin Aggregate CBEM (800kPa-1.5°)	2097	335	16.0	200	2237	14.4
	2867	687	24.0	200	2227	14.8
	2371	561	23.6	200	2227	14.8
	2591	756	29.2	200	2230	14.7
	2482	**585**	**23.2**	**200**	**2230**	**14.7**
Virgin Aggregate CBEM (600kPa-2°)	2352	335	14.2	60	2177	16.7
	1967	391	19.9	37	2172	16.9
Average	**2160**	**363**	**17.1**	**49**	**2175**	**16.8**
Virgin Aggregate CBEM (600kPa-1.25°)	1154	253	21.9	200	2128	18.6
	1474	401	27.2	200	2134	18.4
	2301	405	17.6	200	2164	17.2
	1201	405	33.7	200	2131	18.5
Average	**1533**	**366**	**25.1**	**200**	**2139.3**	**18.2**

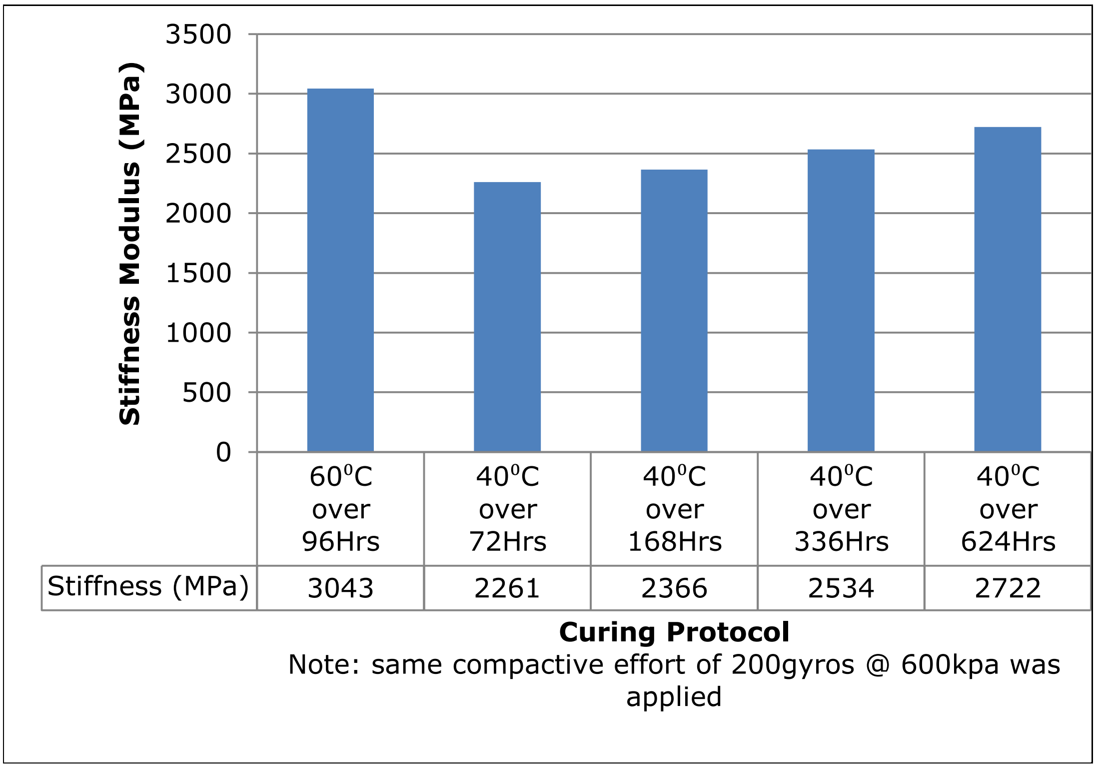


Figure 5.24: Effect of Curing Protocol on Stiffness Properties of Cold Recycled Mixtures

Table 5.21: Effect of Curing Protocol (Limestone Filler used)

Curing Protocol	Stiffness 1 (MPa)	Stiffness 2 (MPa)	Average (MPa)	Air Voids (%)	Air Voids (%) Average
60°C over 96Hrs	2816	2685	3043	15.6	15.1
	3272	3142		14.9	
	3210	3403		15.5	
	2918	2899		14.2	
40°C over 72Hrs	2441	2406	2261	14.8	15.0
	2283	2228		15.7	
	2224	2035		15.1	
	2458	2014		14.3	
40°C over 168Hrs	2262	2489	2366	14.4	15.6
	2543	2043		15.6	
	2777	2278		15.7	
	2277	2257		16.8	
40°C over 336Hrs	2490	2597	2534	14.4	15.6
	2636	2144		15.6	
	2808	2734		15.7	
	2375	2484		16.8	
40°C over 672Hrs	2917	2726	2722	14.4	15.6
	2480	2488		15.6	
	2949	3023		15.7	
	2565	2628		16.8	

examined subsequently.

The results of the penetration and softening point tests as detailed in Table 5.22, indicated that the physical properties of the recovered binders from those cured at 40ºC over 624Hrs (26days) and 60ºC over 96hrs were very close. On a comparative basis, the result also indicated that the two protocols have not significantly degraded the binders in the CBEMs as just 4 and 3 points were lost on penetration for curing at 60ºC and 40ºC respectively when compared to the CBEMs' condition prior to curing.

Consequently, since higher stiffness values were obtained for cores subjected to the 60ºC protocol, it was adopted for long term curing, while 40ºC over 3 days was adopted for short term (intermediate). Curing at 40ºC over 12hrs was deemed suitable for the early life of the cold mixture. Although these curing temperatures are commonly used by researchers (Lee and Kim, 2006; Jacobson, 2002; and Thanaya, 2007), it is worth noting that they are close to pavement temperatures in the tropics. Using the model (equations 5.1a and 5.1b) earlier presented in this chapter which is further discussed in Chapter 6, predicted surface temperature can be as high as 57.76ºC (very close to 60ºC). For a road base layer with 200m thickness and an overlay of 50mm, predicted pavement temperature can be as high as 41.76ºC (very close to 40ºC) at a depth of 150mm from the surface of the pavement.

5.14 Permanent Deformation Properties of CBEMs

The rutting potential of the CBEMs (RAP and virgin aggregate) were briefly studied. As earlier discussed, one of the major failure mechanisms in asphalt road pavements is rutting. Figures 5.25 and 5.26 and Table 5.23 detail the permanent deformation properties of the CBEMs studied using the NAT (DD226:1996 RLAT) and at a temperature of 20ºC. Overall, the virgin aggregate CBEMs had better permanent deformation properties than the RAP Cold BEMs. This must have been due to better aggregate interlocking system that the virgin aggregate CBEMs possess at that level. However, it must be noted that all the specimens tested were dry samples and judging by the stiffness characteristics and water susceptibility properties of the mixture examined earlier, it is believed that such mixture would have performed poorly when tested in a soaked condition compared to the RAP CBEMs. Some of these issues have been further investigated in Chapter 6 at higher temperatures.

Table 5.22: Properties of Recovered Binders under Different Curing Conditions

Type of Mix	Pen (dmm)	SP (°C)	Asphaltene Content (%)
Before Curing	26	61	14.8
60°C over 4days	22	64.0	18.2
40°C over 26days	23	66.8	22.4

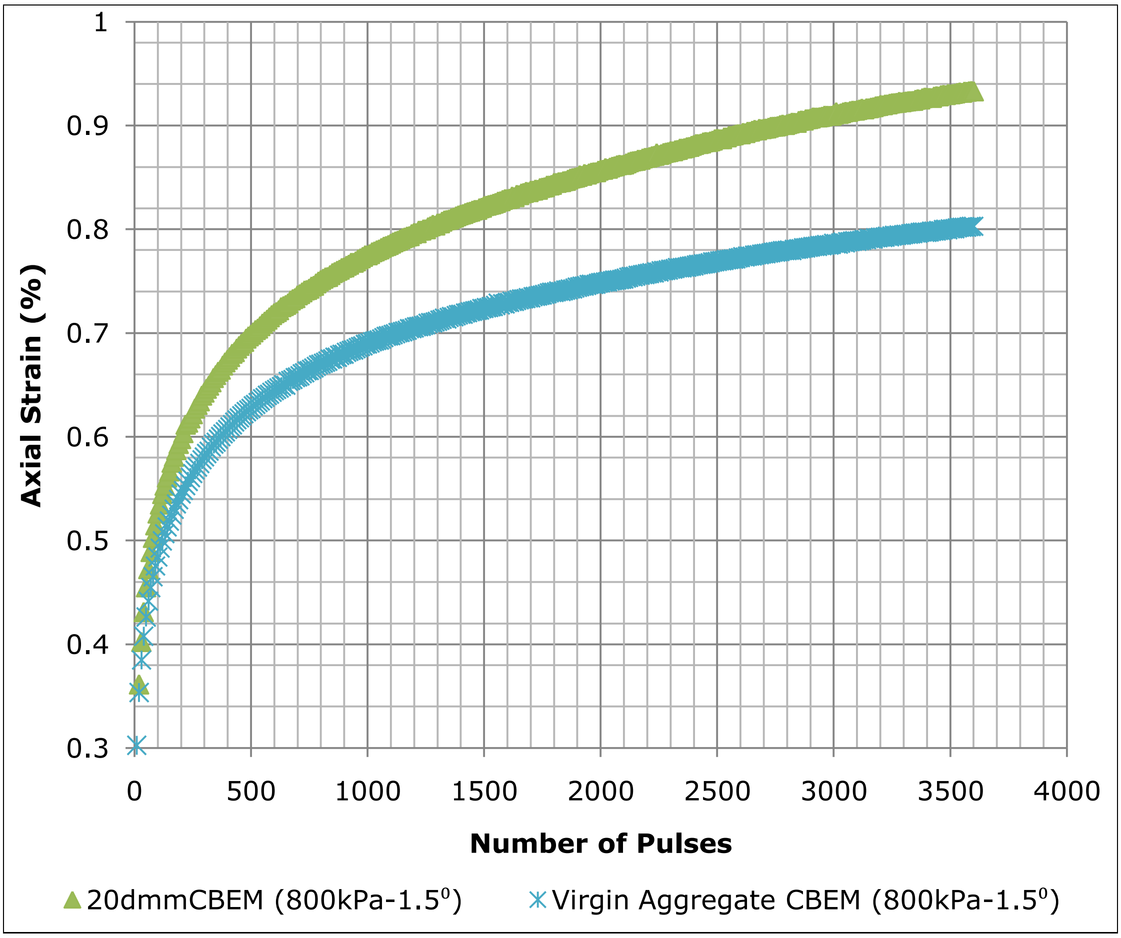


Figure 5.25: Permanent Deformation Properties of CBEMs

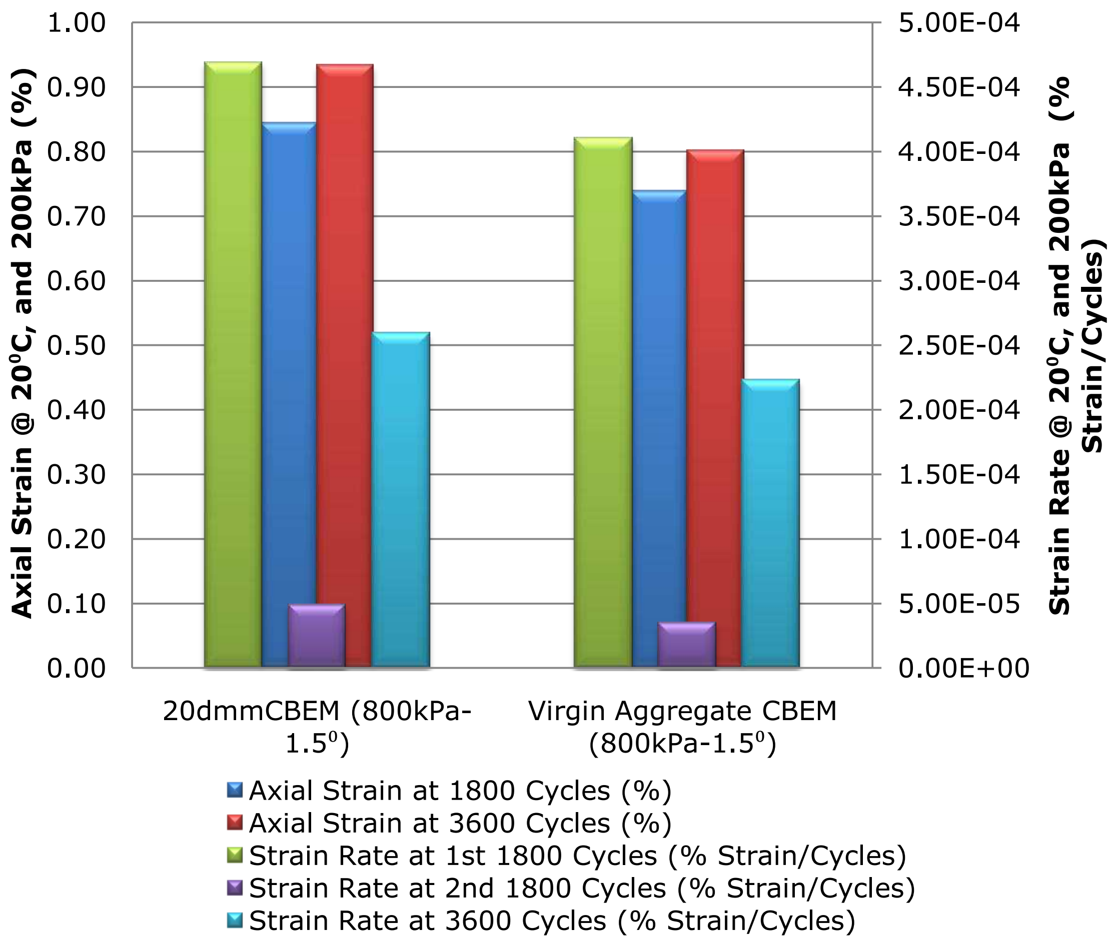


Figure 5.26: Permanent Deformation Characteristics of CBEMs

Table 5.23: Permanent Deformation Properties of CBEMs

Material Description	**Void Content (%)**	**Axial Strain @ 1800 Cycles (%)**	**Axial Strain @ 3600 Cycles (%)**	**Strain Rate @ 1st 1800 Cycles (% Strain/ Cycles)**	**Strain Rate @ 2nd 1800 Cycles (% Strain/ Cycles)**	**Strain Rate @ 3600 Cycles (% Strain/ Cycles)**
20dmmCBEM (800kPa-1.5°)	12.0	0.8563	0.9374	4.76-04	4.51E-05	2.66-04
	12.0	0.8334	0.9288	4.63-04	5.3-05	2.58-04
	12.0	**0.8449**	**0.9331**	**4.69**-04	**4.90E-05**	**2.59-04**
Virgin Aggregate CBEM (800kPa-1.5°)	13.7	0.6476	0.6918	3.6-04	2.46E-05	1.92-04
	13.7	0.7675	0.8216	4.26-04	3.01E-05	2.28-04
	15.6	0.8027	0.8955	4.46-04	5.16E-05	2.49-04
	14.3	**0.7393**	**0.8021**	**4.11**-04	**3.49E-05**	**2.23-04**

5.15 Fatigue Characteristics of Selected CBEMs at Different Stress Levels

An attempt was made to briefly characterise the fatigue responses of the CBEMs produced in this chapter. This entailed determining the number of load cycles to failure ($N_{failure}$ or N_f) and the critical point i.e. point at which when the number of cycles is divided by vertical deformation, it reaches its highest value ($N_{critical}$ or N_c). Two low stress levels of 100kPa and 200kPa were employed in the NAT (DDABF-2003 ITFT). Figure 5.27 and Table 5.24 both detail the properties. It is worth noting that at a stress level of 100kPa, virgin aggregate CBEMs had almost the same fatigue response as RAP CBEMs at a stress level of 200kPa. Overall, the RAP CBEMs performed better than the virgin aggregate CBEMs in fatigue response. The result reported here is just an indicative exercise on the responses of the CBEMs studied here however, further investigations were carried out in order to fully characterise the materials. The results of the extensive investigation are detailed in Chapter 6 of this thesis. The results of the fuller investigations are applied to the structural design of pavements in Chapter 7.

5.16 Implication of the Initial Study on Some Selected CBEMs

A limited study on two materials i.e. virgin aggregate CBEMs and RAP (with 20dmm Pen residual binder) CBEMs were briefly conducted. The materials were given similar treatments of compactive effort and void content. The results showed that the virgin aggregate CBEMs were generally good in response to permanent deformation in dry condition but poor in fatigue and stiffness responses. More importantly, they were highly moisture susceptible compared to the RAP CBEMs studied. The RAP CBEMs examined in this trial were generally good in fatigue, and water susceptibility and the ease of compaction (compactability) compared to the virgin aggregate CBEMs.

The implication is that for a CBEM that contains 65% RAP with 20dmm Pen residual binder, 30% 5mm dust and 5% mineral filler, the aged residual binder plays a beneficial role in the mix as it enhances the mechanical properties over that of 95% virgin aggregate and 5% mineral filler CBEMs with the same gradation, bitumen emulsion content and similar treatments.

5.17 Summary

This chapter has discussed the studies that were carried out on CBEM mix design.

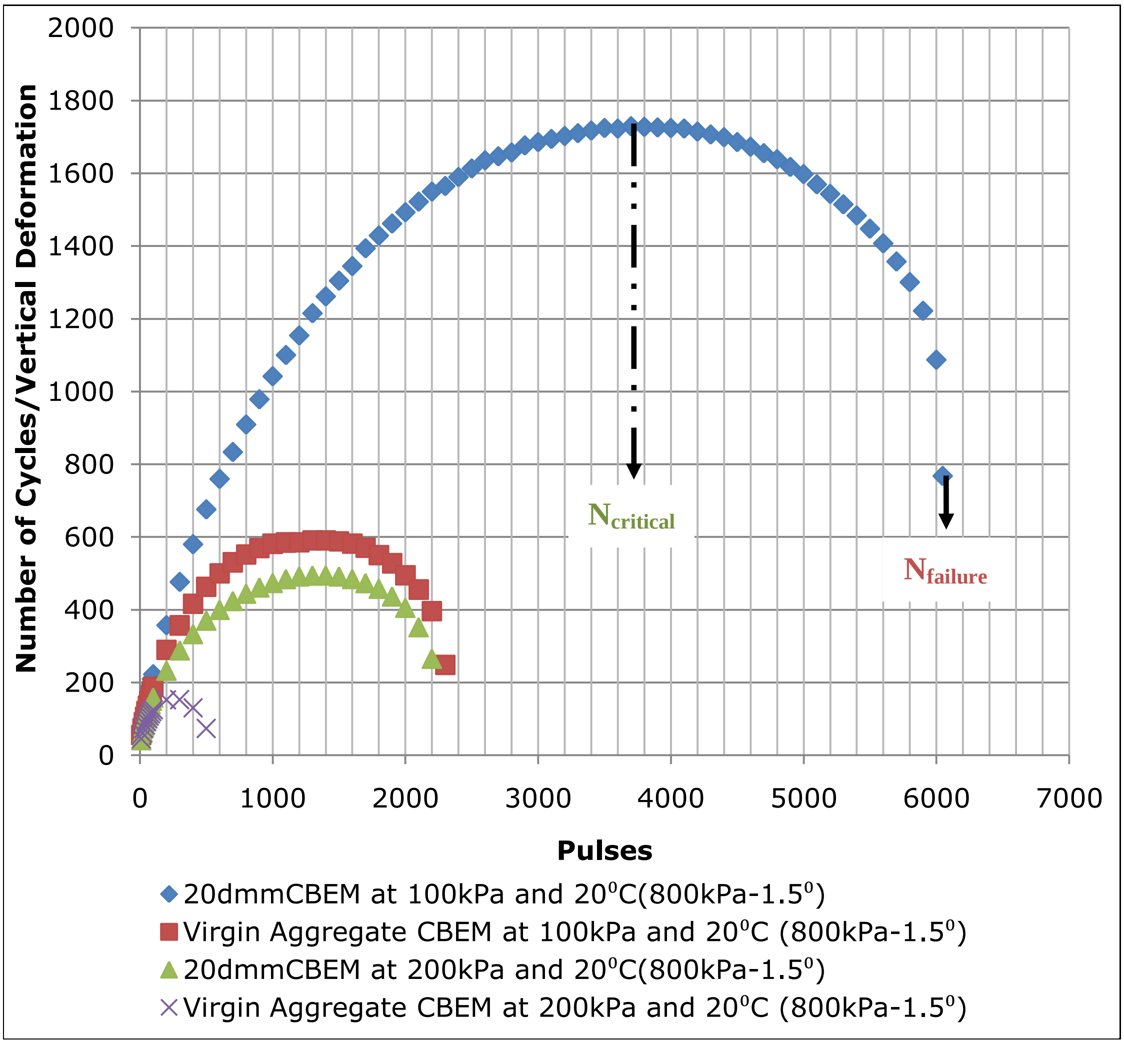


Figure 5.27: Fatigue Characteristics of CBEMs at Different Stress Levels

Table 5.24: Fatigue Properties of CBEMs

Material Description	ITSM (MPa)	Void Content	Test Stress Level (kPa)	N_c	N_f	% N_c (100 * N_c/N_f)
20dmmCBEM @ 200kPa & 20°C(800kPa-1.5°)	2137	15.1	100	3700	6048	61
	2226	11.5	200	1400	2143	65
	2182	13.3	N/A	N/A	N/A	N/A
Virgin Aggregate CBEM @ 200kPa & 20°C (800kPa-1.5°)	2283	12.7	100	1300	2285	57
	2251	13.9	200	300	464	65
	2267	13.3	N/A	N/A	N/A	N/A

Nf = Number of Cycles to failure;, Nc= Number of Cycles to Critical Point

RAP with residual bitumen of 20dmm penetration was used for the exercise. The plan is to use this mix design for the manufacture of CBEMs containing RAPs with varying degrees of severity of ageing. Also, a report on a limited investigation on the mechanical properties of the CBEMs produced during the mix design was also presented. The following general conclusions can be drawn from the studies:

- Mix proportions of 65:30:5 for RAP, 5mm Dust and Filler respectively were found suitable and were adopted for subsequent manufacture of CBEM specimens. The RAP materials used for this mix design contained residual bitumen with penetration value of 20dmm and granite virgin aggregates.
- The Optimum Moisture Content and MDD values were found as 7% and 2222kg/m^3 for the mix studied. The OMC in particular proved useful in the determination of the optimum total fluid content (OTFC) for the mix studied in this work.
- The Optimum Bitumen Emulsion Content (OBEC) and Pre-wetting Water Content were determined as 6.5% and 1.5% respectively making an Optimum Total Fluid Content (OTFC) for the CBEMs to be 8%.
- The Indirect Tensile Stiffness Modulus (ITSM) test proved useful along side the Indirect Tensile Strength (ITS) test as an indicative test for ranking CBEMs during mix design. With ITSM (a non destructive test), materials can be conserved as the same set of specimens tested dry can also be tested for water susceptibility. ITS tests (destructive test) are common for mix designs of this nature.
- Just enough mixing times for both the addition of pre-wetting water and bitumen emulsion are recommended for CBEMs in order to ensure adequate coating and also at such timings, the bitumen emulsion in the mix has not started breaking. All these together facilitate good compactability and stiffness properties for the CBEM. Inadequate mixing time can significantly reduce compactability and stiffness response. A 36.1% reduction in stiffness property as a result of such inadequate timings was observed in this study.
- As observed in this study, limestone filler significantly reduces the water susceptibility of CBEMs. Limestone has been known to be good at inhibiting stripping. However, granite filler gave better dry stiffness properties compared to limestone. Granite aggregate was adopted for subsequent CBEMs mainly because it is often used in Nigeria which is the study area of this work.

- Mixing and the compaction of CBEMs at a temperature of 32ºC gave better results in compactability, stiffness properties and water susceptibility when compared to those similarly done at 20ºC. Such slightly elevated temperature simulates a typical average maximum temperature in the tropics. The two temperatures have been adopted for further studies on the CBEMs. Improvement in stiffness as high as 39.5% was achieved at a temperature of 32ºC over 20ºC in such circumstance.
- As observed in this study, the use of parameters of 600kPa pressure and 1.25º gyration angle on the Cooper Research Gyratory Compactor enhanced the CBEM durability especially the water susceptibility properties compared to other higher compactive efforts such as pressure of 800kPa and gyration angles of 1.5º and 2º. The 600kPa pressure and 1.25º gyration angle parameters were subsequently adopted for the manufacture of all the specimens used in this study.
- Characterisation of the recovered binder from CBEMs cured at 40ºC over 624hrs in the forced draft oven compared to those cured at 60ºC over 96hrs also in the forced draft oven showed very close physico-chemical properties. The results obtained from the two protocols also showed that the bitumen in the mix had not significantly degraded compared to the same properties of the CBEMs characterised prior to curing. Based on the study, curing protocols of 40ºC over 12hrs for early life, 40ºC over 72hrs for intermediate life and 60ºC over 96hrs for fully cured condition have been adopted for further studies on the CBEMs.
- As observed in this limited work, filler type (with the exception of active fillers which were not experimented with in this work) does not influence curing.
- Limited comparative studies on CBEMs containing 100% virgin aggregates, and 65% RAP indicated a better fatigue response by the RAP CBEMs while the virgin aggregate CBEM gave a better performance against rutting. The two results (i.e. fatigue and rutting potential alike) indicate that the residual bitumen in the RAP is contributing to the enhancement of the mechanical–properties of the RAP CBEM. This point is further investigated in Chapter 6 of this thesis.

6 Detailed Investigation of the Mechanical Properties of Cold Bituminous Emulsion Mixtures under Laboratory Simulated Tropical Conditions

6.1 Overview

This chapter describes the investigations carried out on the cold bituminous emulsion mixture (CBEM) designed in the previous chapter. The study involved CBEMs made from different types of RAP materials (i.e. RAPs with different degrees of severity of ageing). This study is deemed appropriate since among other things, proper characterisation of the engineering properties of the materials has been conducted and this is increasingly becoming the standard practice in the industry for ensuring adequate road pavement design. More importantly, knowledge of such performance is desirable under simulated hot tropical conditions, the main focus of this research work. The outcomes, it is believed will help in categorising (ranking) the appropriateness of such severely aged RAPs for cold recycling of road pavements in hot tropical climates.

So as to account for and also evaluate the effect of the RAPs' condition on the studied properties of the CBEMs on a comparative scale, CBEMs with 100% virgin materials were similarly included. Such virgin materials are commonly used for the construction of road base layers in Nigeria though cement stabilised. Meanwhile statistical tools involving both descriptive and inductive analyses were employed in the comparative analyses as is common practice for such studies among researchers (Mubaraki, 2010; Nguyen, 2009; Brennan et al, 2007; Hurley and Prowell, 2006; Tandon et al, 2006; Soleymani et al, 1999; Kandal et al, 1998; Bell et al, 1994; Tayebali et al, 1994; Kandal and Khatri, 1992; and Adedimila, 1986). Tools of this nature are good at ascertaining the significance and influence of the material condition on their properties though within certain confidence levels.

Furthermore, as fatigue and rutting among others have both been identified by researchers as the major failure mechanisms in road pavements (Brown and Brunton, 1986), most structural designs of road pavement require information about the stiffness/resilient modulus properties, fatigue and rutting responses of materials for the road pavement among other things. The results obtained for these tests have proved useful in this regard as some of the results were fed into the Structural Design and Pavement Modelling described in Chapter 7 of this thesis.

6.2 Experimental Programme

The experimental programme involved assessment of CBEMs for compactability, stiffness modulus, fatigue response and rutting potential (permanent deformation). This programme was also extended to cover resilient modulus testing in order to properly account for the early life properties of the CBEMs. At such early life, CBEMs are still evolving and are thus not fully bound materials. Five types of materials in all were tested during the investigation. A full description of these materials is detailed in the succeeding section.

The durability of the materials in respect of water susceptibility, and their temperature susceptibility were also studied. Durability of bituminous pavements is often affected by environmental factors (Needham, 1996). The stiffness and rutting responses of such materials under the individual or combined effects of water and temperature are good indicative tests for assessing durability.

6.3 Materials and Procedure for Specimen Manufacture

The procedure described here is applicable for in-plant cold processed (recycled) bituminous emulsion mixtures. The materials used for specimen manufacture for the investigations reported in this chapter include:

- RAP with residual binder of 5dmm, 10dmm, 15dmm and 20dmm
- Granite 5mm Dust
- Granite Filler
- Granite Aggregates of various sizes
- Nymuls CP 50 – A proprietary bitumen emulsion with residual bitumen of 48dmm penetration
- Demineralised water

Although the process of specimen manufacture is properly detailed in the previous chapter, Table 6.1 gives a summary of the procedure followed in the production of all the specimens that were tested in this chapter. Four CBEMs containing 65% RAP with residual bitumen as follows were prepared:

- 5dmm penetration (5dmm-CBEM)
- 10dmm penetration (10dmm-CBEM)
- 15dmm penetration (15dmm-CBEM)

Table 6.1: Step by Step Approach to CBEM Specimens Preparation

<table>
<tr><td colspan="3">1.Materials for Cold Bituminous Emulsion Mixture (CBEM)</td></tr>
<tr><td>RAP and Granite Aggregates</td><td colspan="2">Proportion (20mm DBM Gradation)
65:30:5 – RAP : Dust : Filler
95:5 – Aggregate : Filler</td></tr>
<tr><td>Fluid Content</td><td>Binder: Nymuls CP 50 (Cationic Bitumen Emulsion)
Content: 6.5% of aggregate mass</td><td>Pre-Wetting Water: Demineralised Water
Content: 1.5% of aggregate mass</td></tr>
<tr><td>Batching</td><td colspan="2">Each batch contains enough materials for 4 cores at 910g per sample</td></tr>
<tr><td colspan="3">2. Mixing</td></tr>
<tr><td>Material Conditioning prior to mixing</td><td colspan="2">- Materials (aggregate mix, bitumen emulsion in air tight container and demineralised water) preconditioned over night at the desired mixing and compaction temperatures (20ºC and 32ºC)
- Set Sun and Planet Mixer also to the desired temperature over night (or 3hrs to the time of mixing)
- Precondition prepared moulds at the required temperature</td></tr>
<tr><td>Fluid Addition and Mixing Technique</td><td colspan="2">- Add the prescribed amount of pre-wetting water to mix in the mixing basin.
- Mix continuously for 60s and stop the mixer. Stir mix manually using a spatula. Mix again for 60s
- Add the prescribed bitumen emulsion content to pre-wetted mix
- Mix for 60s and stop the mixer. Also stir manually to obtain homogeneous mix. Complete operation by mixing for 60s more
- Place loose CBEM in moulds and subsequently condition for 30minutes</td></tr>
<tr><td colspan="3">3. Compaction</td></tr>
<tr><td>Moulds</td><td colspan="2">- Prepare 100mm diameter moulds and precondition overnight
- Upon mixing, place 900g of CBEMs in each mould
- Condition for about 30minutes</td></tr>
<tr><td>Compaction Information</td><td colspan="2">- Set angle of gyration to 1.25º
- Compaction pressure to 600kPa
- Target 200 gyrations for good compaction
- Target 100 gyrations for mild compaction</td></tr>
<tr><td>Specimen Size</td><td colspan="2">- Diameter = 100±1mm
- Height = 50±2mm (requires no cutting)</td></tr>
<tr><td colspan="3">4. Curing</td></tr>
<tr><td>Stage 1 of Curing</td><td colspan="2">First 24Hrs after compaction
- Keep in mould (sealed) condition for 24hrs at ambient condition (temperature of about 20º)
- De-mould or extrude specimens carefully after 24hrs
- Check mass</td></tr>
<tr><td>Stage 2 of Curing</td><td colspan="2">Curing in the Forced Draft Oven After 24Hrs of Stage 1 Curing
- Cure @ 40ºC over 12hrs (Early Life)
- Cure @ 40ºC over 72hrs (Intermediate Life)
- Cure @ 60ºC over 96hrs (Fully Cured Condition)</td></tr>
<tr><td colspan="3">5. Handling of Specimens After Curing</td></tr>
<tr><td>Storage After Curing</td><td colspan="2">If specimens are not tested immediately after curing, store in the cold store at 5ºC until needed for testing</td></tr>
<tr><td>Conditioning Prior to Test</td><td colspan="2">- Prior to any dry test (i.e. ITSM) irrespective of whether stored in the cold store or not, place specimens in the conditioning cabinet at the desired test temperature for about 8hrs (preferably over night)
- Specimens for wet tests (for water sensitivity) should be conditioned for 2hrs in water at the desired test temperature before testing</td></tr>
</table>

- 20dmm penetration (20dmm-CBEM)

A control mix of CBEM containing 100% Virgin Aggregates (VA-CBEM) was also prepared. All the specimens produced had the same aggregate gradation and total fluid content (pre-wetting water content + bitumen emulsion content). These allowed for comparative analysis since one of the objectives of this research work is to identify the point at which the residual aged bitumen in the RAPs starts enhancing the engineering properties of the mix. Although some hot mixtures were brought into the picture later for a balanced comparison, virgin aggregate CBEM (VACBEM) has been selected as the control mainly because practitioners in the field tend to favour the use of virgin materials in Nigeria. Performances of such mixtures compared with those containing RAPs will make it clear whether incorporating RAP CBEMs into the pavement structure is of any advantage or not.
By varying the curing conditions, the CBEMs were variously prepared to simulate early life (about 12hrs after being laid), intermediate life (say 6 months after construction) and fully cured condition (up to 6 or 7 years after construction).

6.4 Testing Methods

While the Cooper Research Gyratory Compactor was used for studies on the compactability of CBEMs, the Cooper Research Nottingham Asphalt Tester (NAT) was used for conducting the range of other tests mentioned earlier. A description of the Gyratory Compactor had been given in Chapter 5 and thus needs no further mention here. Compactability was studied by applying high and low compactive efforts of 200 and 100 gyrations on specimens while other factors remained unchanged. The analysis was based on density (air void contents) of the CBEMs.

Meanwhile, the NAT is known to be widely used for tests on bituminous materials because of its versatility in aiding mixture design and specification in practical situations within the highway industry (Brown, 1995). More importantly, this inexpensive facility provides a user friendly interface for conducting the tests even on a routine basis. It is suitable for testing cores (100mm or 150mm diameter) manufactured in the laboratory or those cored from road pavements. Some of the modes of testing in the NAT are:

- Indirect Tensile Stiffness Modulus (ITSM) test (strain or stress controlled)
- Indirect Tensile Fatigue Test (ITFT)
- Repeated Load Axial test (RLAT)
- Vacuum Repeated Load Axial Test (VRLAT)
- Indirect Tensile Resilient Modulus Test (ITRMT)

In addition to these traditional modes of testing, the Resilient Modulus Test using the VRLAT mode can also be conducted on the NAT. This is achievable as a result of the new developments on the NAT's actuator and coupled with the availability of the more versatile and recently developed Universal Test Software. Except for the ITRMT, all these mentioned modes of testing on the NAT were used in this study.

Two NATs specifically were used in this investigation - the 'Old' and the NU14. The old version was used for most of the ITSM tests, ITFT, RLAT and VRLAT carried out in this study. The NU14 is a new version of the NAT and was used for the Resilient Modulus Test, though with an external vacuum pump. The NU14 is a servo-pneumatic machine with maximum loading capacity of about 20kN. The machine has an advantage in that its actuator is capable of applying both compressive and tensile forces and in a manner that allows for control over the load waveform and enables loads to be cycled between the two modes of force application at high frequencies (CRT, 2010). Some of the other special features of the machine as stated by CRT (2010) are:

- Digitally controlled servo-pneumatic system
- Cycling at frequencies up to 70Hz
- Built in displacement transducer for precise position control
- Control & Data Acquisition system with precision 16 bit digital servo-control
- Transducers available for modulus, permanent deformation and triaxial tests
- Temperature controlled cabinet (-10ºC to +60ºC, ±0.2ºC)

Meanwhile, the 4 main components of the NAT as described by Ibrahim (1998) are:

- Main test Frame placed in a temperature controlled cabinet
- Interface Unit for the acquisition data and control of test
- A (servo-) pneumatic system connected with the interface unit and the actuator mounted above the test frame for controlling the applied load
- A computer unit

Figure 6.1 shows the 'Old' and the new NU14 NATs that were used in this work. Figure 6.2 shows the four components of the NAT as described earlier.

6.5 The Theory of the Indirect Tensile Test Mode

This section gives a quick description of the theory behind the Indirect Tensile Test mode. This is important for a proper understanding of the principles behind the ITSM test and the ITFT alike which were extensively used in this investigation.

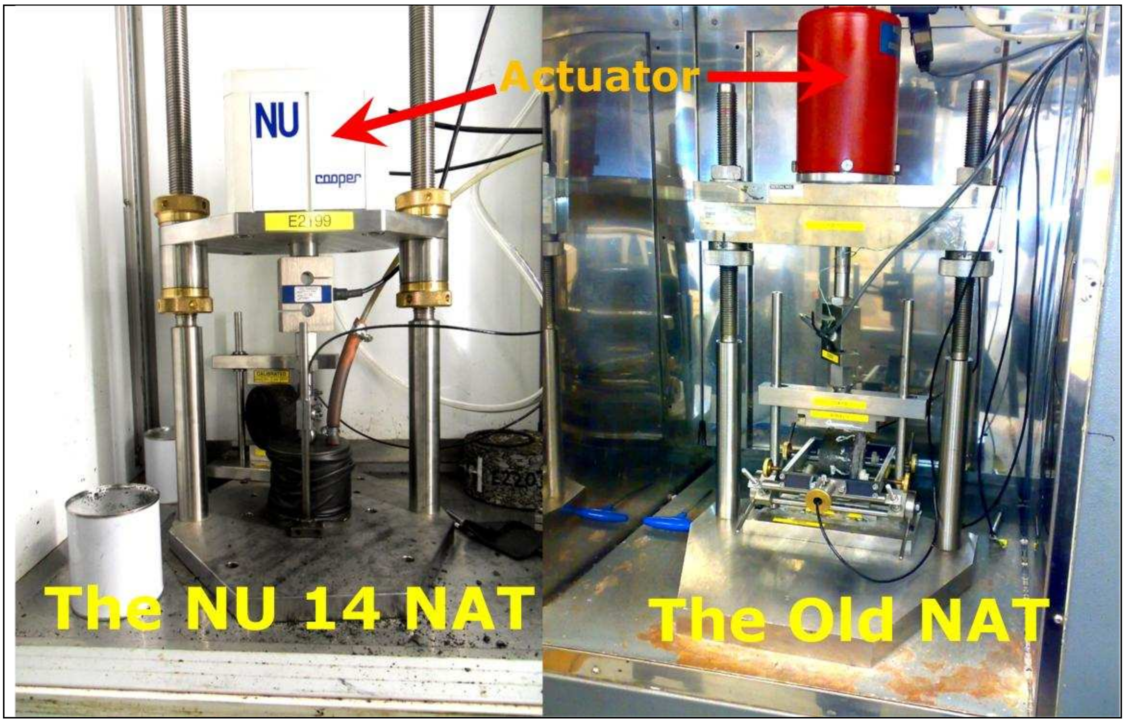


Figure 6.1: The Nottingham Asphalt Tester Test Frames

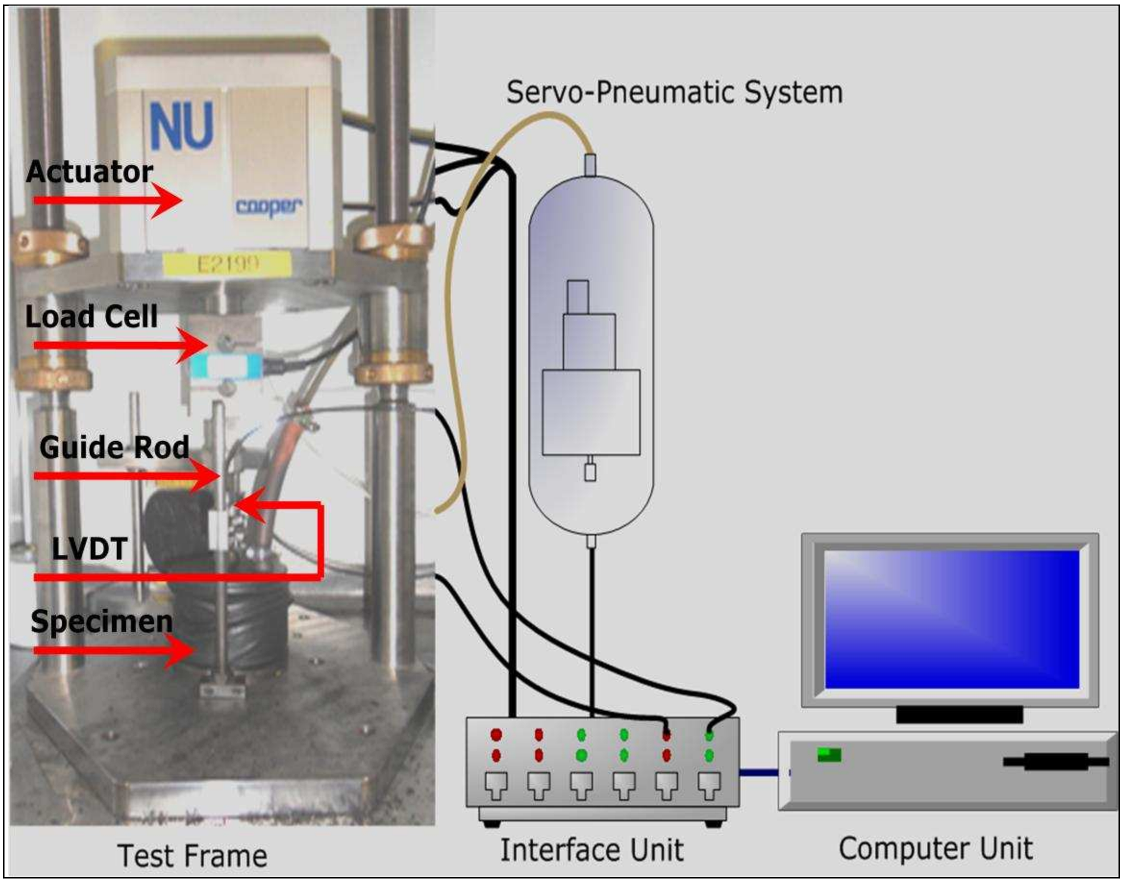


Figure 6.2: The Main Components of a NAT (Shown in the VRLAT Mode)

Sunarjono (2008) reported that in the indirect tensile test mode, the compression load is applied across the vertical diameter of a cylindrical specimen and it results in a biaxial stress distribution in the specimen as shown in Figure 6.3. This diagram indicates that both a vertical compressive stress (σ_{vx}) and a horizontal tensile stress (σ_{hx}) are induced on the horizontal diameter of the specimen and the magnitudes of the stresses vary along the diameter. However, they both reach maximum value at the centre of the specimen. He opined that by measuring the horizontal deformation, the values of both the maximum strain and the stiffness modulus of the specimen can be calculated. The calculations are however based on the following assumptions stated by Read (1996) as follows:

- The specimen is subjected to plane stress conditions (σ_z=0).
- The material is linear elastic.
- The material is homogeneous and behaves in an isotropic manner.
- Poisson's ratio for the material is known.
- The vertical load (P) is applied as a line loading.

With all these conditions satisfied, Read and Whiteoak (2003) stated that the stress conditions are in consonance with the theory of elasticity. The theory shows 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:

$$\sigma_{hx} = \frac{2P}{\pi.d.t} \qquad 6.1$$

$$\sigma_{vx} = \frac{-6P}{\pi.d.t} \qquad 6.2$$

Their averages can further be expressed as follows:

$$\overline{\sigma_{hx}} = \frac{0.273P}{d.t} \qquad 6.3$$

$$\overline{\sigma_{vx}} = \frac{-P}{d.t} \qquad 6.4$$

Where:

d= specimen diameter

t = specimen thickness

P = applied compression or vertical 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

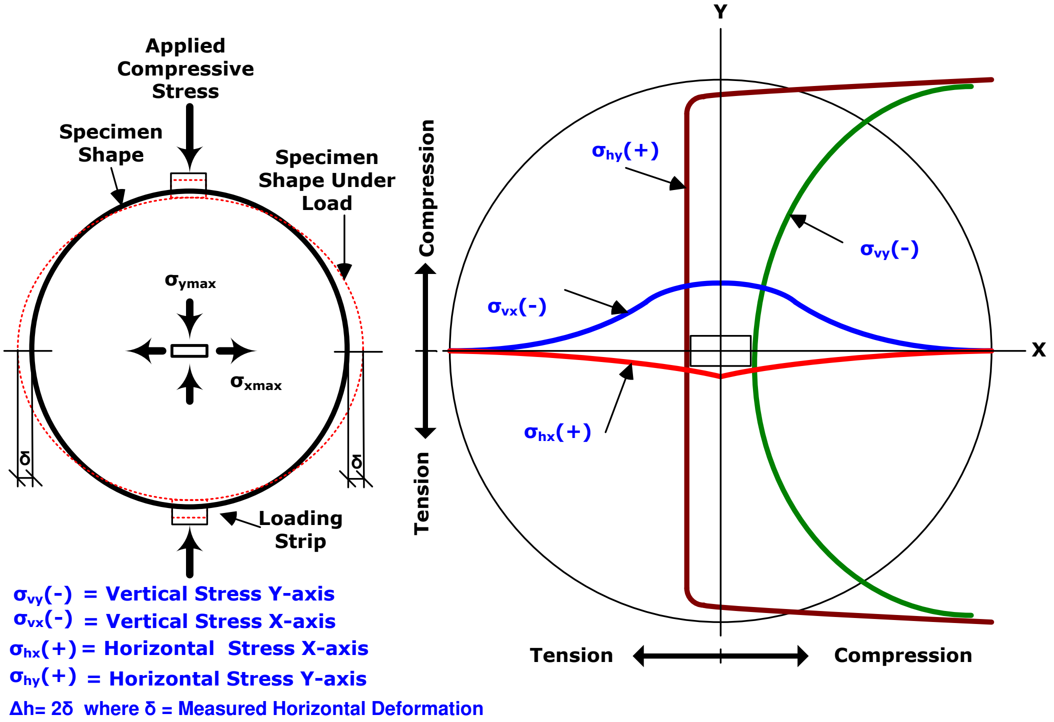


Figure 6.3: Biaxial Stress Distribution under Compression Load in the Indirect Tensile Test Mode

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

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

Therefore, for the principal stresses in an elemental section subjected to biaxial stress conditions, the horizontal tensile strain is:

$$\overline{\varepsilon_{hx}} = \frac{\overline{\sigma_{hx}}}{S_m} - \upsilon\frac{\overline{\sigma_{vx}}}{S_m} \qquad 6.5$$

$$\overline{\varepsilon_{hx}} = \frac{0.273P}{S_m.d.t} + \frac{\upsilon P}{S_m.d.t} \qquad 6.6$$

Horizontal deformation is determined from:

$$\Delta\text{h} = \overline{\varepsilon_{hx}}.d \qquad 6.7$$

$$\Delta\text{h} = \frac{0.273P}{S_m.t} + \frac{\upsilon P}{S_m.t} \qquad 6.8$$

Stiffness Modulus of the material therefore can be calculated thus:

$$S_m = \frac{P\,(0.273+\upsilon)}{\Delta\text{h}.t} \qquad 6.9$$

Where:

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

υ = Poisson's ratio (assumed)

S_m = Stiffness Modulus

Δh = horizontal deformation (measured)

Read and Whiteoak (2003) further stated that by simple linear elastic stress analysis:

$$\varepsilon_{hx(max)} = \frac{\sigma_{\text{hx(max)}}}{S_m} - \upsilon\frac{\sigma_{\text{vx(max)}}}{S_m} \qquad 6.10$$

Substituting equation 6.1 and 6.2 into 6.10 yields:

$$\varepsilon_{hx(max)} = \frac{2\text{P}}{S_m.\pi.d.t} + \frac{\upsilon 6P}{S_m.\pi.d.t} \qquad 6.11$$

Substituting equation 6.1 into 6.11 yields:

$$\varepsilon_{hx(max)} = \frac{\sigma_{\text{hx(max)}}}{S_m}(1+3\upsilon) \qquad 6.12$$

Equation 6.12 is used for calculating maximum tensile strain ($\varepsilon_{hx(max)}$) at the centre of the specimen. This strain is required for the indirect tensile fatigue test. It follows from the equation that stiffness modulus and the maximum tensile stress values are needed in order to calculate the tensile strain in the fatigue test.

6.6 Explanations on Statistical Tools used for Analysis of Results

This section gives brief descriptions of the statistical tools that were used in analysing the results obtained from the experimental investigation conducted and presented in this chapter.

6.6.1 Normality test

Park (2008) advised that it is important to check the pattern of distribution before analysing data. The results of the normality test will guide in choosing the appropriate test (parametric or non-parametric), for analysing the data. Two tests, graphical and numerical, are available for checking the normality of data. Park (2008) opined that numerical tests are more objective and the most popular methods are the Kolmogorov-Smirnov test (K-S test) and the Shapiro-Wilk test (W-S test). When the distribution is normal, parametric tests are used for further analysis and if otherwise, non-parametric tests are employed.

The One-Sample Kolmogorov-Smirnov (Normality) Test (in the SPSS 16) was used in this chapter. The SPSS 16 manual states that the test procedure compares the observed cumulative distribution function with a specified theoretical distribution, which may be normal, uniform, Poisson or exponential. The Kolmogorov-Smirnov Z is computed from the largest difference (in absolute value) between the observed and theoretical cumulative distribution functions. This goodness-of-fit test tests whether the observations could reasonably have come from the specified distribution.

The output of the test gives sample size, normal parameters (mean and standard deviation), most extreme differences, Kolmogorov-Smirnov Z and the two-tailed significance of the test statistic which is the p-value. The p-value ranges from zero to one. If $p>0.05$, the observed cumulative distribution compares with or is not significantly different from the specified theoretical distribution and therefore the distribution is normal. However, if $p<0.05$, the difference between the two distributions is significant and therefore the cumulative distribution is not normally distributed (Mubaraki, 2010).

6.6.2 Multiple regression analysis

Multiple regression analysis was conducted in this chapter using STATA software. Multiple regression analysis is a technique that estimates a single regression model with the prediction of the dependent (y) variable from a set of independent (x) variables (SADLP, 2010). Multiple regressions assume that data sample follow a normal distribution and that the relationship is linear. The method is particularly important when dealing with large number of variables that express common information. For example the dependent variable "y" is assumed to be a function of a set of "k" independent variables x1, x2, x3… Xk, and this yields a regression equation thus:

$$Y = a + b1x1 + b2x2 + \ldots.. + bkxk \qquad 6.13$$

Each "b" is a partial slope coefficient i.e. when all other independent variable is equal to zero, or held constant, in other words, each "b" coefficient is the slope of the relationship between that particular independent variable "x" and the dependent variable "y". For example, the "b1" coefficient in equation 6.13 refers to the slope between "x1" and the independent variable when all other variables in the equation, x2, x3, etc equal to zero. Multiple regression analysis therefore allows for stating relationships between two main variables while controlling for other factors – also known as partial effects (SADLP, 2010).

For example, multiple regression analysis were useful in ascertaining the relationship between air void content and stiffness modulus of the CBEMs studied in this chapter i.e. VACBEM, 5dmmCBEM, 10dmmCBEM, 15dmmCBEM and 20dmmCBEM while controlling for the effect of the five CBEM types. Within this grouped data, each CBEM type was taken as a subgroup such that from equation 6.13, VACBEM was coded as "x1", 5dmmCBEM was coded as "x2", etc in the regression model. This method was similarly useful for taking into account the preparation method used for the CBEMs i.e. 20ºC, 100 gyrations 20ºC, 200 gyrations, etc as discussed in this chapter.

6.7. Compactability of CBEMs

A reasonable level of compactability is desirable for CBEMs in order to guard against rutting and problems associated with durability of asphalt mixtures. Thus the compactability of the CBEMs was studied by applying two compactive efforts of 100 and 200 gyrations. All other conditions apply as stated in Table 6.1. The air void content (density) was used for the analyses as it is a good measure of the degree of compaction and in essence compactability of the material.

6.7.1 The effect of CBEM composition, temperature of preparation and compactive effort on compactability post compaction

An investigation involving the five CBEMs, prepared under conditions simulating post compaction was conducted. Post compaction is assumed to be the three stages of, (1) immediately after compaction (extruded specimens after curing in mould for 24hrs), (2) early life (curing of the extruded cores in the forced draft oven over a period of 12hrs at 40ºC in an unsealed condition after the stage 1 curing in the mould) and (3) intermediate life conditions (same with early life condition with curing in the force draft oven extended beyond 12hrs to 72hrs). In order to account for the effect of compactive effort, temperature of preparation of specimens and CBEM type, the following parameters were considered:

Number of gyrations	: 100 and 200 gyrations
Mixing and Compaction Temperature	: 20ºC and 32ºC

Tables 6.2, 6.3 and 6.4 show the result of the One-Sample Kolmogorov-Smirnov (Normality) Tests conducted on the measured air voids of the CBEMs for conditions simulating immediately after compaction, early life and intermediate life respectively. The results of the normality tests indicate that all the measurements were normally distributed as all the values in Asymp. Sig. (2-tailed) row were greater than 0.05. Since the distribution is normal, therefore the results can be presented using descriptive statistics. The tables similarly contain the respective values for the mean and standard deviation of the measured air voids. The standard deviation values were generally low which indicate low variation in the observed air void contents, and they do not follow a specific trend. The mean values of the air voids were used in plotting Figures 6.4 and 6.5. Figure 6.4 shows the mean air void contents for the five CBEMs immediately after compaction. The figure clearly shows that the 20dmmCBEM had the best air void contents i.e. lowest values with the VACBEM having the highest values for preparation temperature of 32ºC. Although the air voids were generally lower than 16% at this level, the figure shows that the CBEMs prepared at 20ºC were less compact compared to those prepared at 32ºC. At preparation temperature of 20ºC, the 20dmmCBEM indicated the lowest air void contents (though with values higher than those observed at 32ºC) while the 10dmmCBEM had the highest values. The figure shows that the compactive effort of 200 gyrations gave better

Table 6.2: One-Sample Kolmogorov-Smirnov (Normality) Test for Air Voids at Simulated Post Compaction Air Voids

Applied Treatment (Gyrations, Temp.°C)	Parameters		Simulated Post Compaction Air Voids				
			VA CBEM	5dmm CBEM	10dmm CBEM	15dmm CBEM	20dmm CBEM
100, 20	Sample Size		8	8	8	8	8
	Normal Parameters[a]	Mean	14.605	14.525	15.664	13.748	11.913
		SD	0.991	0.981	0.983	1.317	1.210
	Kolmogorov-Smirnov Z		0.529	0.494	0.644	0.521	0.442
	Asymp. Sig. (2-tailed)		0.942	0.968	0.801	0.949	0.990
200, 20	Sample Size		8	24	24	8	24
	Normal Parameters[a]	Mean	14.265	13.864	14.589	12.279	10.675
		SD	1.301	0.878	0.645	2.078	1.464
	Kolmogorov-Smirnov Z		0.839	0.561	0.445	0.741	0.721
	Asymp. Sig. (2-tailed)		0.481	0.912	0.989	0.643	0.675
100, 32	Sample Size		24	24	24	24	24
	Normal Parameters[a]	Mean	13.144	12.315	11.548	9.123	8.275
		SD	1.335	1.019	1.453	1.033	1.537
	Kolmogorov-Smirnov Z		0.566	0.522	0.777	0.689	0.676
	Asymp. Sig. (2-tailed)		0.906	0.948	0.582	0.730	0.750
200, 32	Sample Size		88	108	108	88	108
	Normal Parameters[a]	Mean	12.533	11.478	9.822	8.139	7.041
		SD	1.552	1.053	1.265	1.359	1.154
	Kolmogorov-Smirnov Z		0.543	0.556	0.781	0.564	0.820
	Asymp. Sig. (2-tailed)		0.930	0.917	0.575	0.909	0.512

a. Test distribution is Normal.

Table 6.3: One-Sample Kolmogorov-Smirnov (Normality) Test for Air Voids at Simulated Early Life Condition of CBEMs

Specimen Condition	Parameters		VA CBEM	5dmm CBEM	10dmm CBEM	15dmm CBEM	20dmm CBEM
Early Life Condition Air Voids (200 gyrations, 32°C)	**Sample Size**		32	32	32	32	32
	Normal Parameters[a]	Mean	17.045	15.218	12.876	12.276	10.775
		SD	1.575	0.916	0.997	0.906	1.083
	Kolmogorov-Smirnov Z		0.775	0.787	0.976	0.496	0.698
	Asymp. Sig. (2-tailed)		0.585	0.565	0.297	0.966	0.715

a. Test distribution is Normal.

Figure 6.4: One-Sample Kolmogorov-Smirnov (Normality) Test for Air Voids at Intermediate Life Condition of CBEMs

Applied Treatment (Gyrations, Temp.)	Parameters		Final Air Voids				
			VA CBEM	5dmm CBEM	10dmm CBEM	15dmm CBEM	20dmm CBEM
100, 20	Sample Size		4	4	4	4	4
	Normal Parameters[a]	Mean	18.660	18.681	19.918	17.243	15.900
		SD	0.770	1.095	0.798	0.882	1.270
	Kolmogorov-Smirnov Z		0.444	0.566	0.408	0.829	0.437
	Asymp. Sig. (2-tailed)		0.989	0.906	0.996	0.497	0.991
200, 20	Sample Size		4	4	4	4	4
	Normal Parameters[a]	Mean	19.054	18.391	18.251	15.821	14.500
		SD	1.504	0.943	0.455	0.761	0.497
	Kolmogorov-Smirnov Z		0.567	0.656	0.586	0.473	0.340
	Asymp. Sig. (2-tailed)		0.905	0.782	0.883	0.979	1.000
100, 32	Sample Size		12	12	12	12	12
	Normal Parameters[a]	Mean	17.336	16.464	15.621	13.168	12.275
		SD	0.912	0.661	0.892	0.823	1.019
	Kolmogorov-Smirnov Z		0.530	0.565	0.539	0.536	0.622
	Asymp. Sig. (2-tailed)		0.941	0.907	0.933	0.936	0.834
200, 32	Sample Size		12	12	12	12	12
	Normal Parameters[a]	Mean	16.495	15.717	14.418	12.572	11.125
		SD	1.443	0.637	0.836	1.269	0.991
	Kolmogorov-Smirnov Z		0.775	0.498	0.911	0.639	0.682
	Asymp. Sig. (2-tailed)		0.585	0.965	0.378	0.809	0.741

a. Test distribution is Normal.

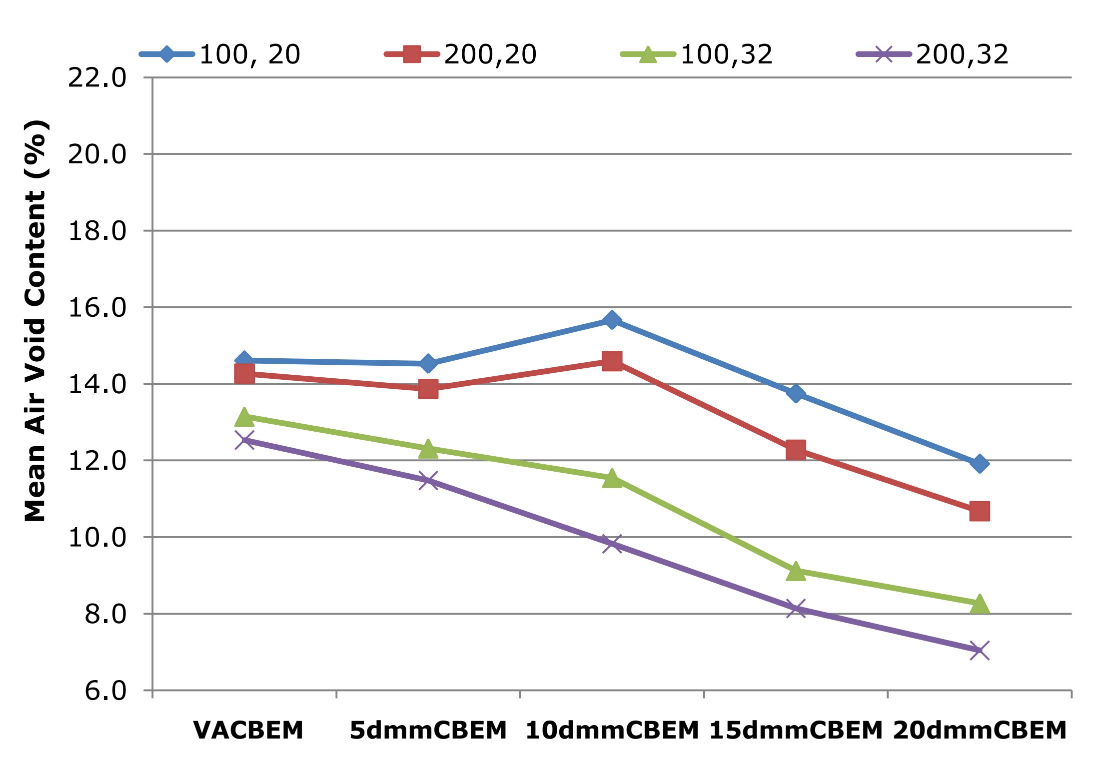


Figure 6.4: Mean Air Void Contents of CBEMs Immediately after Compaction

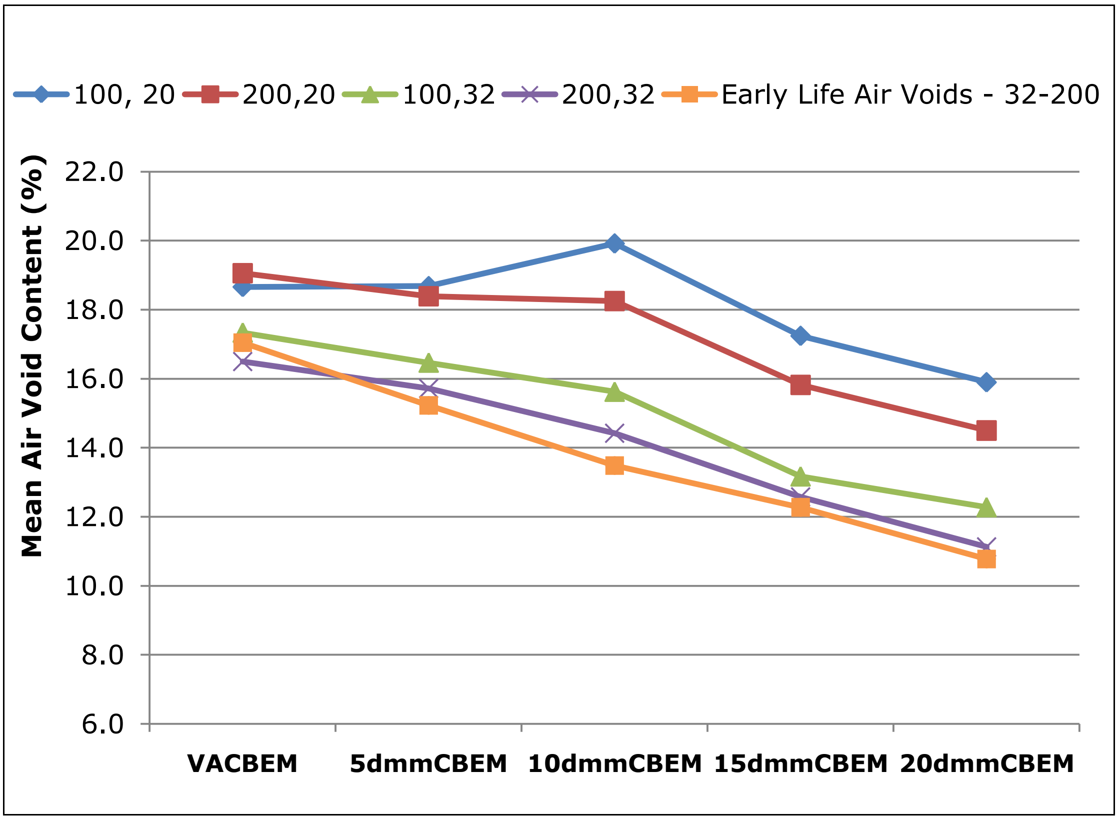


Figure 6.5: Mean Air Void Contents of CBEMs at Simulated Early and Intermediate Lives

response compared to 100 gyrations.

No doubt, the mixing and compaction temperature of 32ºC was a major factor for the enhancement of CBEMs prepared at that temperature. It is very likely that at such temperature, the severely aged residual bitumen in the RAPs was 'excited'/softened or possibly rejuvenated thus becoming less viscous. This in effect aided better lubrication during compaction resulting in a higher level of compaction as evidenced in the air void content values. This effect could also have demobilised some of the RAP fines which hitherto were glued to the coarse RAPs by the residual aged binder in the RAP. It implies thus that at such temperature, the CBEMs in effect had more lubricants than the VACBEM. Another important factor that could have contributed to the lower air void contents of the CBEMs prepared at 32ºC was the viscosity of the bitumen emulsion at such temperature. The Redwood II viscosity test conducted on the bitumen emulsion indicated values of 5.14s and 15s for temperatures of 30ºC and 20ºC respectively.

Figure 6.5 shows the mean air void contents of the CBEMs at simulated early and intermediate lives. The results similarly indicated that preparation (mixing and compaction) temperature and compactive effort both play important roles in ensuring compactability of CBEMs. However, mixing and compaction temperature had greater influence on compactability compared to compactive effort. For example, the mean air void contents for the combination 32ºC - 100 gyrations were generally lower (better) than those observed for 20ºC - 200 gyrations. This observation was irrespective of RAP type in CBEM, with 20dmmCBEM having the lowest air void contents and the 5dmmCBEM, the highest content of air void.

Furthermore, the observed trend in Figure 6.5 is similar to that of Figure 6.4, however, the mean air void contents were generally higher than those previously observed in Figure 6.4. This as result of further loss of moisture contents in the CBEMs. The mean moisture contents at different curing conditions and the standard deviation are detailed in Table 6.5 while Figures 6.6 and 6.7 are the graphical representations of mean moisture contents at early and intermediate life conditions, and the fully cured conditions respectively. This might not be a true picture of what obtains on site. As these results indicate, within the first few hours of laying the material, a significant portion of the moisture in the CBEM evaporates (about 71% of total moisture content), which subsequently raises the level of the air void contents. This is a weak point for this laboratory simulation since ordinarily, within a short time after the construction, vehicles will start trafficking the road pavement. Loads from such vehicular traffic would aid further

Table 6.5: Moisture Contents of CBEMs at Different Curing Conditions

Specimen Condition	Moisture Content (%)									
	VACBEM		5dmm CBEM		10dmm CBEM		15dmm CBEM		20dmm CBEM	
	M	SD	M	SD	M	SD	M	SD	M	SD
EL (32°C, 200 gyrations)	1.0	0.2	1.1	0.2	1.4	0.2	1.3	0.3	1.2	0.5
IL (20°C, 100 gyrations)	0.3	0.3	0.3	0.2	0.3	0.2	0.5	0.2	0.6	0.2
IL (20°C, 200 gyrations)	0.6	0.2	0.3	0.1	0.5	0.3	0.5	0.1	0.6	0.2
IL (32°C, 100 gyrations)	0.4	0.2	0.6	0.2	0.7	0.3	0.8	0.3	0.8	0.2
IL (32°C, 200 gyrations)	0.5	0.2	0.6	0.2	0.9	0.3	0.8	0.3	0.7	0.2
FC (20°C, 100 gyrations)	0.2	0.0	0.3	0.1	0.6	0.3	0.4	0.1	0.5	0.0
FC (20°C, 200 gyrations)	0.2	0.2	0.5	0.2	0.5	0.2	0.6	0.3	0.5	0.2
FC (32°C, 100 gyrations)	0.4	0.3	0.5	0.2	0.7	0.2	0.8	0.2	0.7	0.3
FC (32°C, 200 gyrations)	0.4	0.2	0.6	0.2	0.7	0.3	0.7	0.3	0.6	0.2

Note: M= Mean; SD= Standard Deviation; EL = Early Life; IL = Intermediate Life; FC = Fully Cured

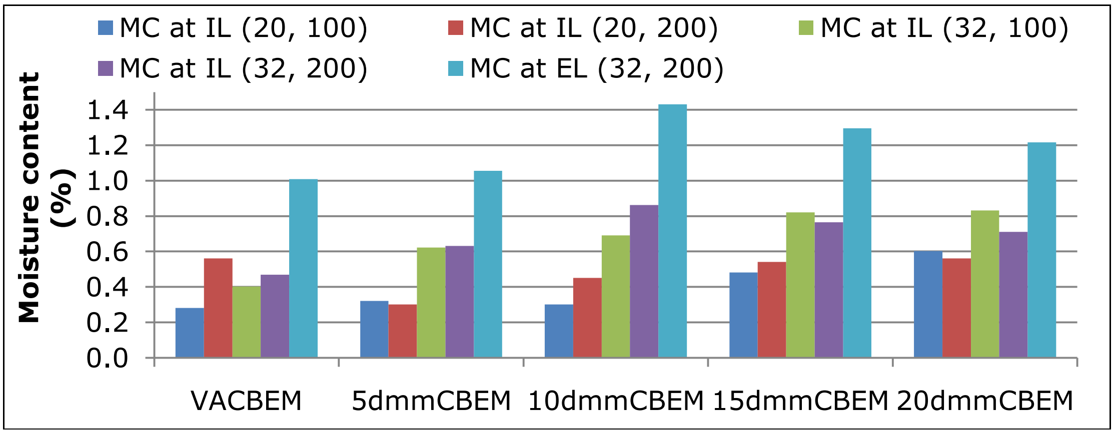


Figure 6.6: Moisture Contents of CBEMs at Early and Intermediate Life Conditions

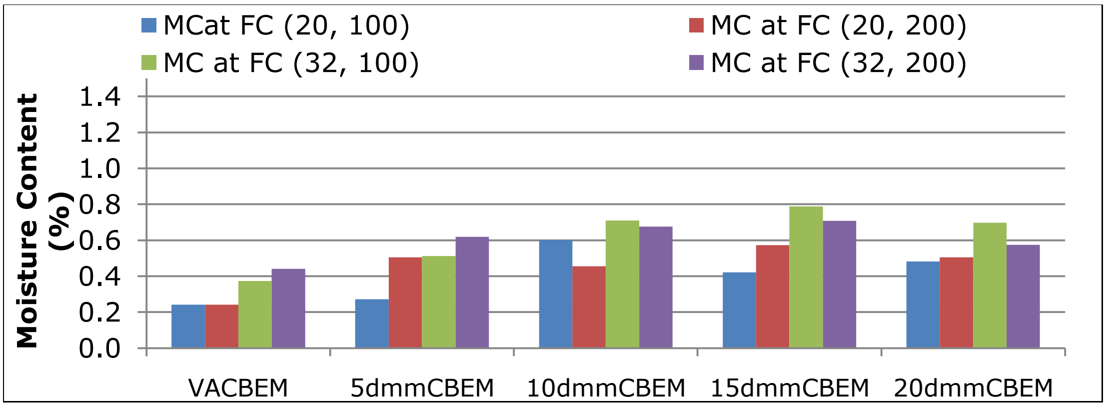


Figure 6.7: Moisture Contents of CBEMs at Fully Cured Conditions

consolidation in the road pavement with an accompanying reduction in air void contents. The specimens studied here did not receive further compaction after manufacture. Serfass et al (2009) in a study in which initial air voids of CBEMs were compared with those of later life found a reduction in air voids (though not significant) in road pavements.

The trends in Figures 6.6 and 6.7 show that air void content affects the rate of moisture loss in the CBEMs. The higher the initial air void content, the higher the rate of moisture loss from the CBEM. In this regard, the VACBEM showed the highest loss of moisture content. Ojum (2009) in a related work observed similar trend.

6.7.2 The effect of CBEM composition, temperature of preparation and compactive effort on compactability of fully cured specimens

Similar studies to those carried out in the preceding section were also conducted on specimens conditioned to simulate fully cured condition. With all other conditions for specimen preparation as previously done in the preceding section held, the following were also investigated at this level along with the effect of material type:

Number of gyrations	: 100 and 200 gyrations
Mixing and Compaction Temperature	: 20ºC and 32ºC
Curing Protocol	: 60ºC over 96hrs

Again, before any analysis was conducted, the results of the measurements for the air void contents were also tested for normality. The results are as indicated in Table 6.6. The One-Sample Kolmogorov-Smirnov (Normality) Test showed that the measurements were similarly normally distributed (all the values in Asymp. Sig. (2-tailed) row were greater than 0.05). Similarly, the standard deviation values were generally low which indicate low variation in the observed air void contents, and they do not follow a specific trend as was observed in the previous section. Figure 6.8 shows that the mean air void contents of the CBEMs at mixing and compaction temperature of 32ºC for fully cured condition followed similar trend as observed in the preceding section. However, the trend of the air voids of the RAP CBEMs compared to those of the VACBEM at the mixing and compaction temperature of 20ºC were not completely consistent with the observations made in the preceding section. This observation was most obvious for those compacted using the compactive effort of 200 gyrations as there was no significant difference from those compacted using 100 gyrations.

Table 6.6: One-Sample Kolmogorov-Smirnov (Normality) Test for Air Voids at Fully Cured Condition of CBEMs

Applied Treatment (Gyrations, Temp.)	Parameters		Final Air Voids				
			VA CBEM	5dmm CBEM	10dmm CBEM	15dmm CBEM	20dmm CBEM
0100, 20	Sample Size		4	4	4	4	4
	Normal Parameters[a]	Mean	19.583	19.009	19.615	18.642	16.503
		SD	1.166	1.061	1.031	1.128	1.049
	Kolmogorov-Smirnov Z		0.330	0.442	0.584	0.484	0.358
	Asymp. Sig. (2-tailed)		1.000	0.990	0.885	0.973	1.000
200, 20	Sample Size		4	20	20	4	20
	Normal Parameters[a]	Mean	18.125	17.967	18.887	17.275	15.126
		SD	0.227	0.747	0.635	2.358	1.414
	Kolmogorov-Smirnov Z		0.600	0.627	0.583	0.563	0.583
	Asymp. Sig. (2-tailed)		0.864	0.826	0.886	0.909	0.886
100, 32	Sample Size		12	12	12	12	12
	Normal Parameters[a]	Mean	17.583	16.512	15.765	13.453	12.734
		SD	1.576	1.036	1.616	0.888	1.621
	Kolmogorov-Smirnov Z		0.602	0.616	0.594	0.805	0.554
	Asymp. Sig. (2-tailed)		0.862	0.843	0.872	0.535	0.919
200, 32	Sample Size		44	64	64	44	64
	Normal Parameters[a]	Mean	16.654	15.692	14.223	12.197	11.624
		SD	1.679	1.012	1.089	1.152	1.041
	Kolmogorov-Smirnov Z		0.613	0.610	0.619	0.515	0.459
	Asymp. Sig. (2-tailed)		0.846	0.850	0.838	0.954	0.984

a. Test distribution is Normal.

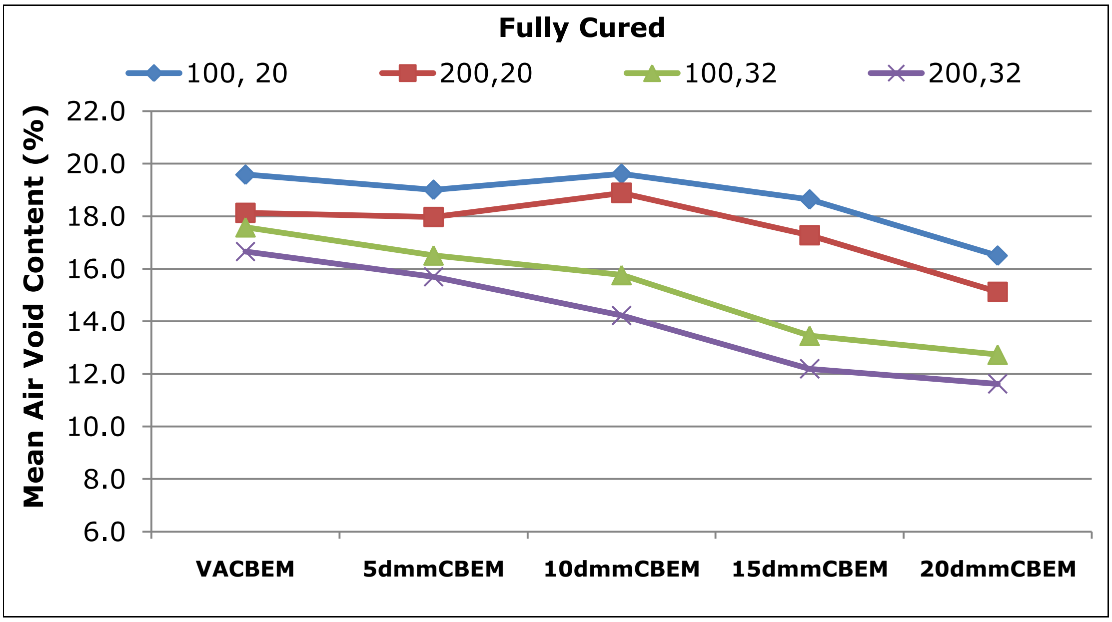


Figure 6.8: Mean Air Void Contents of CBEMs at Simulated Fully Cured Condition

These results clearly imply that apart from better compactability being attainable as observed in the preceding section, the consistency and reproducibility of similar air void contents in CBEMs mixed and compacted at 32ºC are uniform and significantly better compared to those mixed and compacted at 20ºC. The results also confirmed that there is little difference in compactability of CBEMs achieved when compactive effort of 100 gyrations or 200 gyrations is applied compared to the effect of mixing and compaction temperatures 32ºC of 20ºC which is significant. The 10dmmCBEM performed poorest in compactability at 20ºC.

The trends observed in this study for the air voids suggest that initial air void content (at the point of construction) directly impacts (affects) the path of later in-service air void contents and similarly moisture contents of CBEMs. In particular, the moisture contents at the fully cured condition (though very low now i.e. ≤ 0.8%) suggest that the CBEMs might not totally become dry. Serfass et al (2003) argued that moisture content of cold mixes has often been found to be between 0.5 and 1.5% in road bases in temperate climate. Although the VACBEM showed the lowest value for moisture content at fully cured condition, it is believed that with further consolidation of the CBEMs in-service, the accompanying reduction in air void contents will reduce the rate of moisture loss since contact with air and heat (agents of evaporation) by the CBEM is further reduced. On average, the CBEMs containing severely aged RAPs performed significantly better than those of the VACBEM in compactability as earlier observed in the previous sections.

6.8 The Indirect Tensile Stiffness Modulus (ITSM) Test

Stiffness modulus gives an indication as to the expected performance of the respective bituminous layers of a road pavement in service. The performance in this regard is with respect to the ability of the layer to effectively spread traffic loading safely without imposing significant stresses/strains on the underlying layers. Therefore knowledge of stiffness property of any layer in such road pavements is required for adequate structural design. This property, for bituminous mixtures is strongly dependent on the temperature and frequency (period) of loading (Read and Whiteoak, 2003).

The Indirect Tensile Stiffness Modulus test is the common means of measuring this property of bituminous mixtures in the UK. In fact the ITSM test has been reported as the most frequently conducted test in the NAT (Wu, 2009). Apart from being inexpensive, the test is simple and can be quickly conducted compared to other traditional means of testing the stiffness modulus of bituminous mixtures. Brown (1995) reported that up to 100 specimens can be tested for ITSM in a working day. Experience from the studies reported in this chapter confirms such claims as at some point about 120 specimens were successfully completed in about 9hrs. Another advantage of this test is that it allows cylindrical specimens, usually either 100 or 150mm in diameter to be tested. The test specimen does not need to be long, typically 30 to 80mm, which is convenient when coring pavements. The load to be applied is lower than that required, for example in a direct compression test. The test also offers a convenient means of applying tensile stress under compressive loading (Brown, 1995).

ITSM tests are regarded as non-destructive tests and are normally conducted using the BS DD 213: 1993 as a guide. Plate A in Figure 6.9 shows the ITSM testing mode in the NAT. A summary of the test protocol from the mentioned standard is detailed in Table 6.7. It is worth noting however that, the target horizontal deformation used in this study was 3 μm due to the fragile nature of the specimens. Santagata et al (2009) used a target of 2 μm in their work and found it very useful for cold mixtures. All specimens tested in this study were 100±2mm in diameter and 50±2mm thick. Also, tests were conducted at 30ºC and 40ºC as well as the standard of 20ºC. In addition, the standard requires that all specimens are tested twice on diameters 90º±10º apart and the measurements are averaged and recorded as the result, though with the condition that such measurements do not differ by more than 10%. This was followed in this study except for occasions where specimens were too fragile. Prior to testing, all specimens must have been brought to the required test temperature in the conditioning cabinet (for about 7hrs). During the test, once information on deformation (measured by two Linear Variable Differential Transformers - LVDTs), load, specimen dimensions and Poisson’s ratio are available, equation 6.9 applies for the stiffness modulus.

6.8.1 Stiffness Modulus Response and CBEM Composition

Measurements were made for the specimens at the early life, intermediate life and fully cured conditions. The indirect tensile stiffness modulus (ITSM) responses as presented here were measured at 20ºC after curing for the respective cases discussed. Before delving into the investigation in full, an attempt was made to check the effectiveness of the chosen curing protocols which simulate early life (12hrs at 40ºC), intermediate life (72hrs at 40ºC) and fully cured (96hrs at 40ºC) conditions. The short analysis was based only on CBEMs mixed and compacted at

Table 6.7: Summary of ITSM Testing Protocol (BS DD 213: 1993)

Parameter	Protocol
Rise time (ms)	124 ± 4
Target Horizontal Deformation (μm)	5 ± 2 (for 100mm diameter) 7 ± 2 (for 150mm diameter)
Load (N)	Commensurate to target deformation for strain controlled test
Pulse Duration (s)	3
Wave Shape	Haversine
Number of Conditioning Pulses	5
Test Temperature (ºC)	20±0.5
Poisson's Ratio	0.35(representative for most bituminous mixtures)
Specimen Thickness (mm)	30-80

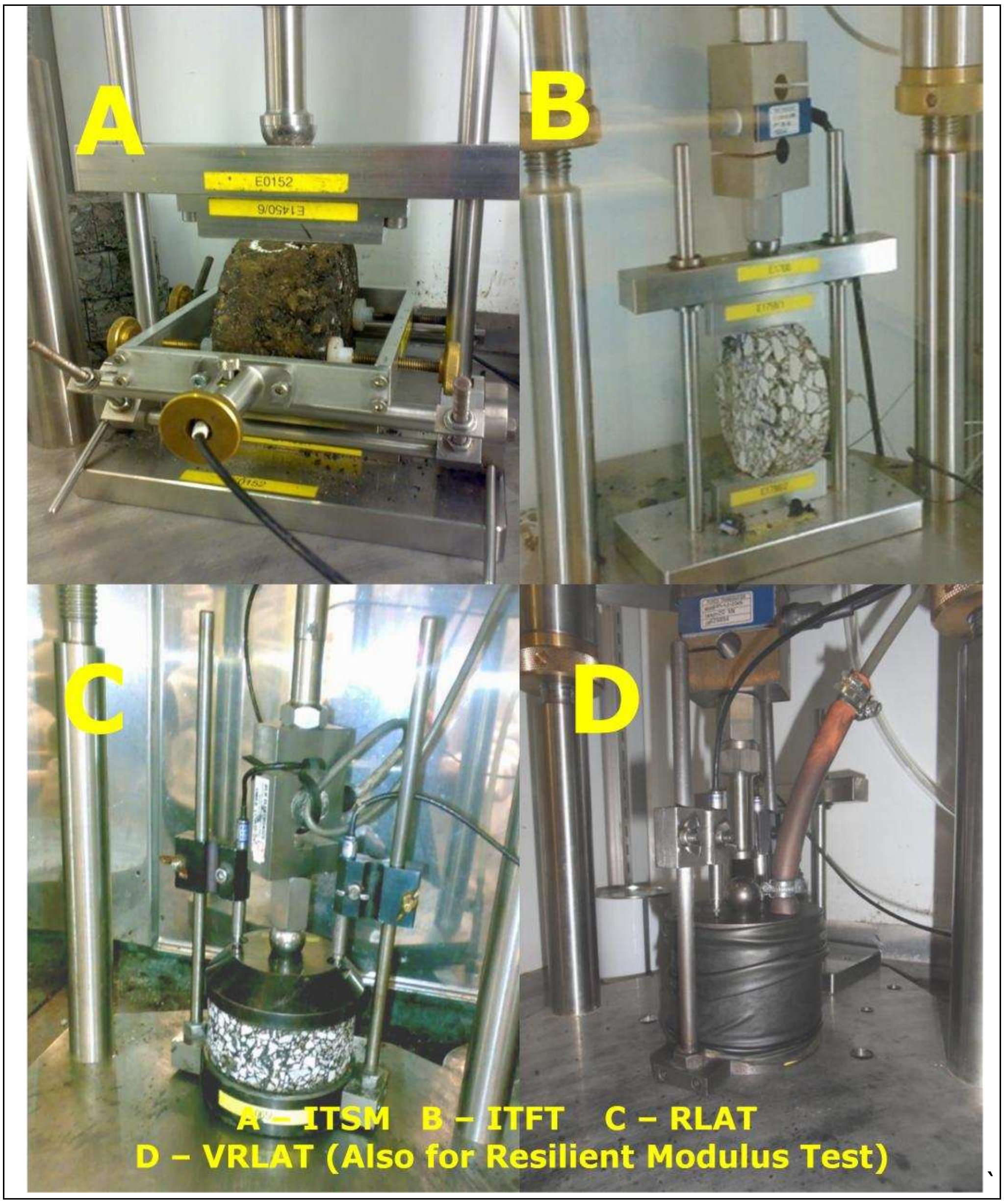


Figure 6.9: Different Modes of Testing in the NAT

32ºC using a compactive effort of 200 gyrations. All the observed measurements were tested for normality. The statistics for the measurements were also ascertained. The results of the One-Sample Kolmogorov-Smirnov (Normality) Test on the measured ITSM values are as detailed in Table 6.8 while some of the statistics are as detailed in Table 6.9. Figure 6.10 shows the effect of the curing protocols on the ITSM responses of the CBEMs. The result of the normality test indicated that the measurements were normally distributed since all the values in Asymp. Sig. (2-tailed) row were greater than 0.05. The higher this value the better the goodness of fit. The figure shows a logical trend since it generally indicates that the longer the time of curing the higher the ITSM. The steeper trend observed for 60ºC 96hrs compared to that of 40ºC 12hrs and 40ºC 72hrs is attributable to the change in temperature. These protocols are here assumed to be good enough to simulate the desired conditions.

6.8.1.1 ITSM and CBEM composition at early life condition

The ITSM values were measured for all the five CBEMs produced under the simulated early life condition. As done previously, all the observed measurements were tested for normality. The results of the One-Sample Kolmogorov-Smirnov (Normality) Test on the measured ITSM values for simulated early life condition are as detailed in Table 6.10. The result indicated that the measurements were similarly normally distributed since all the values in Asymp. Sig. (2-tailed) row were greater than 0.05. The Table contains also the values for the standard deviation and means of ITSM for five CBEMs. More importantly, the results generally indicate that the ITSM test was good enough at ranking the CBEMs in the expected logical order at this early life condition.

Figure 6.11 and Table 6.11 show the results and the percentage improvements in ITSM of the CBEMs over the VACBEM at simulated early life condition. They are in agreement with earlier findings reported on compactability. The other CBEMs performed better than the VACBEM. 5dmmCBEM had the least percentage improvement of 8.8% while 20dmmCBEM had the highest of 32%. As the penetration of the residual binder in the severely aged RAP increases, there is a corresponding increase in ITSM. The trend in Figure 6.11 however suggests that the ITSM values might plateau just after the 20dmmCBEM as previously observed for compactability of the CBEMs in the early life. This gives an indication that there is probably a correlation between the compactability of the CBEMs and the corresponding ITSM value. Such relationship exits for HMA (Tayebali et al, 1994). This is investigated in this chapter.

Table 6.8: One-Sample Kolmogorov-Smirnov (Normality) Test for ITSM of CBEMs at Simulated Early Life, Intermediate Life and Fully Cured Conditions

CBEM Type	Parameters		40^0C for 12Hrs	40^0C for 72Hrs	60^0C for 96Hrs
VA CBEM ITSM @ 20^0C	Sample Size		32	12	44
	Normal Parameters[a]	Mean	1645	2464	2882
		SD	194	345	567
	Kolmogorov-Smirnov Z		0.440	0.561	0.560
	Asymp. Sig. (2-tailed)		0.990	0.911	0.913
5dmm CBEM ITSM @ 20^0C	**Sample Size**		32	12	64
	Normal Parameters[a]	Mean	1790	2180	2548
		SD	131	206	288
	Kolmogorov-Smirnov Z		0.579	0.479	0.744
	Asymp. Sig. (2-tailed)		0.891	0.976	0.636
10dmm CBEM ITSM @ 20^0C	**Sample Size**		32	12	64
	Normal Parameters[a]	Mean	1877	2386	3109
		SD	201	237	480
	Kolmogorov-Smirnov Z		0.696	0.599	0.665
	Asymp. Sig. (2-tailed)		0.718	0.865	0.769
15dmm CBEM ITSM @ 20^0C	**Sample Size**		32	12	44
	Normal Parameters[a]	Mean	2056	2365	3607
		SD	157	182	501
	Kolmogorov-Smirnov Z		0.730	0.669	0.492
	Asymp. Sig. (2-tailed)		0.661	0.762	0.969
20dmm CBEM ITSM @ 20^0C	**Sample Size**		32	12	64
	Normal Parameters[a]	Mean	2172	2499	3934
		SD	171	226	345
	Kolmogorov-Smirnov Z		0.643	0.588	0.880
	Asymp. Sig. (2-tailed)		0.803	0.880	0.421

Table 6.9: Statistics for the Effect of Curing on ITSM (20^{0}C) of CBEMs Manufactured at 32^{0}C/200 gyrations

CBEM Type	Parameters	40^{0}C for 12Hrs	40^{0}C for 72Hrs	60^{0}C for 96Hrs
VA CBEM ITSM @ 20^{0}C	Sample Size	32	12	44
	Std. Error of Mean	34.265	99.456	85.542
	Minimum	1286	1671	1938
	Maximum	2131	2810	3924
5dmm CBEM ITSM @ 20^{0}C	Sample Size	32	12	64
	Std. Error of Mean	23.184	59.430	35.939
	Minimum	1560	1760	1710
	Maximum	2147	2517	3159
10dmm CBEM ITSM @ 20^{0}C	Sample Size	32	12	64
	Std. Error of Mean	35.545	68.509	59.996
	Minimum	1539	2004	2212
	Maximum	2373	2879	4178
15dmm CBEM ITSM @ 20^{0}C	Sample Size	32	12	44
	Std. Error of Mean	27.747	52.449	75.576
	Minimum	1761	1951	2409
	Maximum	2320	2645	4591
20dmm CBEM ITSM @ 20^{0}C	Sample Size	32	12	64
	Std. Error of Mean	30.234	65.156	43.138
	Minimum	1898	2149	3133.00
	Maximum	2654	2920	4805.00

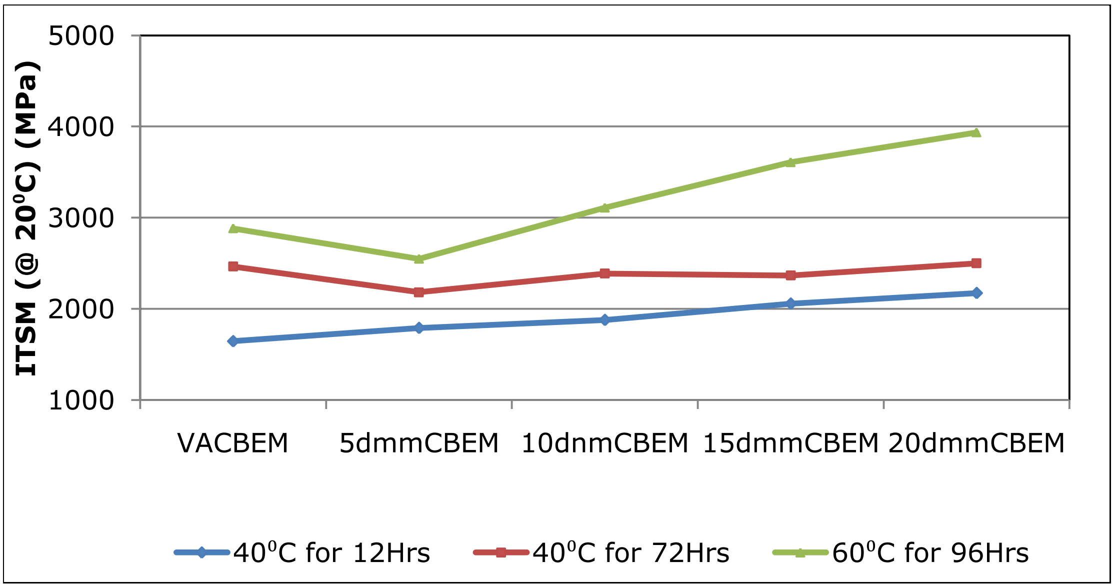


Figure 6:10: The Effect of Curing Protocols on CBEMs

Table 6.10: One-Sample Kolmogorov-Smirnov (Normality) Test for ITSM at Simulated Early Life Condition of CBEMs

Specimen Condition	Parameters		VA CBEM	5dmm CBEM	10dmm CBEM	15dmm CBEM	20dmm CBEM
ITSM @ Early Life	Sample Size		32	32	32	32	32
	Normal Parameters[a]	Mean	1645	1790	2022	2056	2172
		SD	194	131	222	157	171
	Kolmogorov-Smirnov Z		0.440	0.579	0.523	0.730	0.643
	Asymp. Sig. (2-tailed)		0.990	0.891	0.948	0.661	0.803

a. Test distribution is Normal.

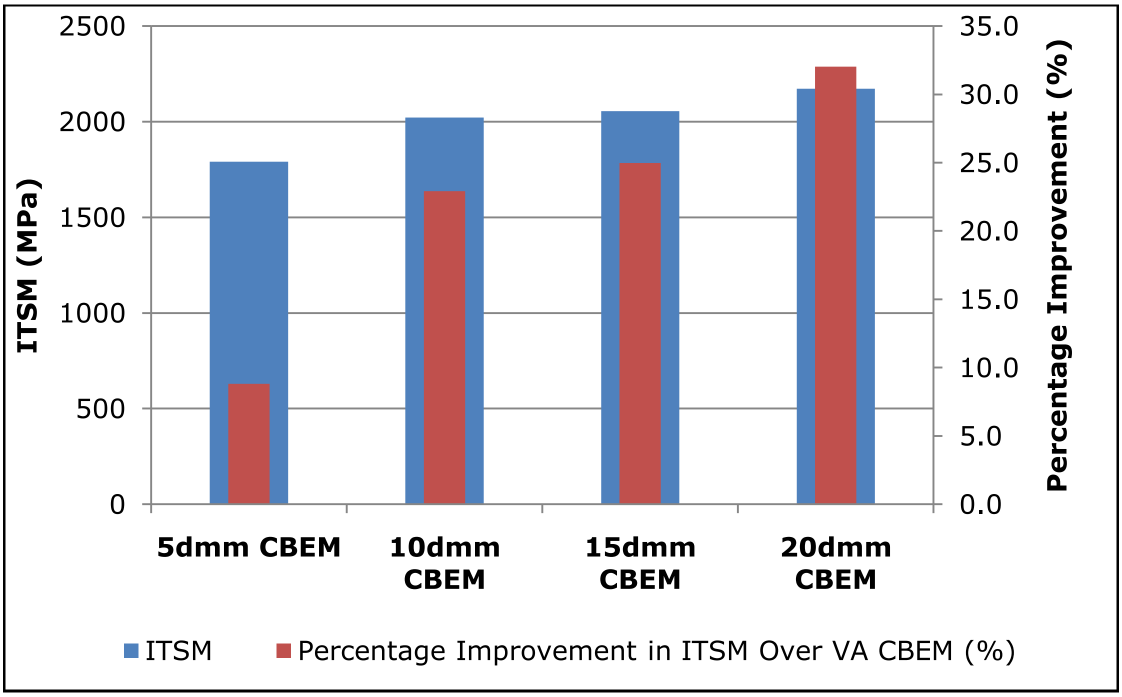


Figure 6.11: ITSM of CBEMs and their Respective Percentage Improvements over the VACBEM at Early life

Table 6.11: Percentage Improvements in ITSM of other CBEMs over the VACBEM at Simulated Early Life Condition

Material Type	5dmm CBEM	10dmm CBEM	15dmm CBEM	20dmm CBEM
ITSM (MPA)	1790	2022	2056	2172
Percentage Improvement in ITSM Over VA CBEM (%)	8.8	22.9	25.0	32.0

6.8.1.2 ITSM and CBEM composition at intermediate life condition

All the specimens prepared for the intermediate life condition were similarly tested for ITSM. The study here afforded the opportunity to investigate the effect of compactive effort and temperature of mixing and compaction on the ITSM responses of CBEMs as was done for compactability.

The results of the One-Sample Kolmogorov-Smirnov (Normality) Test on the measured ITSM values for simulated intermediate life of CBEMs are detailed in Table 6.12. The results again indicated that the measurements were normally distributed as all the values in Asymp. Sig. (2-tailed) row were greater than 0.05. The table similarly contains the standard deviation and mean values of the distribution. Figure 6.12 shows the ITSM responses of the CBEMs at the simulated intermediate life. The figure indicates that, the ITSM was not good enough to rank the CBEMs at the intermediate life condition as there is no clear trend observed compared to the early life condition. However, the figure shows that all the CBEMs mixed and compacted at 32ºC have ITSM values clearly above 2000MPa (irrespective of the compactive effort applied) which indicates that the CBEMs are progressively evolving when compared to those values observed for early life.

Table 6.13 and Figure 6.13 both detail the percentage improvements in ITSM of RAP CBEMs over those for the VACBEM at simulated intermediate life condition. Figure 6.13 in particular corroborates the earlier observation of lack of clear trend. The VACBEM performed better in all cases with the exception of the 20dmmCBEM. The 5dmmCBEM and 15dmmCBEM only indicated better performances at the mixing and compaction temperature of 20ºC and compactive effort of 200 gyrations. 10dmmCBEM generally was poorer than VACBEM as observed in the figure. It was also observed from Figure 6.12 that the ITSM responses at the mixing and compaction temperature of 20ºC with the low compactive effort of 100 gyrations were higher than those for higher compactive effort of 200 gyrations for the VACBEM and 5dmmCBEM. The only reasonable explanation is that at such operational temperature, the optimum degree of compactness of the mix facilitated by the presence of fluid (moisture and emulsion) was reached at very near 100 gyrations under aggregate-fluid-aggregate interaction in the matrix, with the 5dmm RAP behaving just as the virgin aggregates i.e. black rock. Black rock is a RAP in which the residual binder in it is regarded as inactive or not contributing to the properties of the new mix. Beyond 100 gyrations, aggregate-aggregate interaction plays a predominant role in the matrix, and with further compaction, defragmentation of the coarse aggregates in the mix starts to occur. This ordinarily will alter and accordingly

Table 6.12: One-Sample Kolmogorov-Smirnov (Normality) Test for ITSM at Simulated Intermediate Life Condition of CBEMs

Applied Treatment (Gyrations, Temp.)	Parameters		ITSM				
			VA CBEM	5dmm CBEM	10dmm CBEM	15dmm CBEM	20dmm CBEM
100, 20	Sample Size		4	4	4	4	4
	Normal Parameters[a]	Mean	1982	1869	1459	1773	2128
		SD	404	214	169	75	245
	Kolmogorov-Smirnov Z		0.424	0.533	0.351	0.519	0.541
	Asymp. Sig. (2-tailed)		0.994	0.939	1.000	0.951	0.932
200, 20	Sample Size		4	4	4	4	4
	Normal Parameters[a]	Mean	1527	1623	1483	1817	2335
		SD	277	123	93	203	331
	Kolmogorov-Smirnov Z		0.429	0.560	0.433	0.480	0.543
	Asymp. Sig. (2-tailed)		0.993	0.913	0.992	0.975	0.930
100, 32	Sample Size		12	12	12	12	12
	Normal Parameters[a]	Mean	2398	2079	2201	2362	2472
		SD	476	172	305	268	180
	Kolmogorov-Smirnov Z		0.493	0.922	0.950	0.533	0.838
	Asymp. Sig. (2-tailed)		0.968	0.363	0.327	0.939	0.484
200, 32	Sample Size		12	12	12	12	12
	Normal Parameters[a]	Mean	2464	2180	2386	2365	2472
		SD	345	206	237	182	206
	Kolmogorov-Smirnov Z		0.561	0.479	0.599	0.669	0.550
	Asymp. Sig. (2-tailed)		0.911	0.976	0.865	0.762	0.923

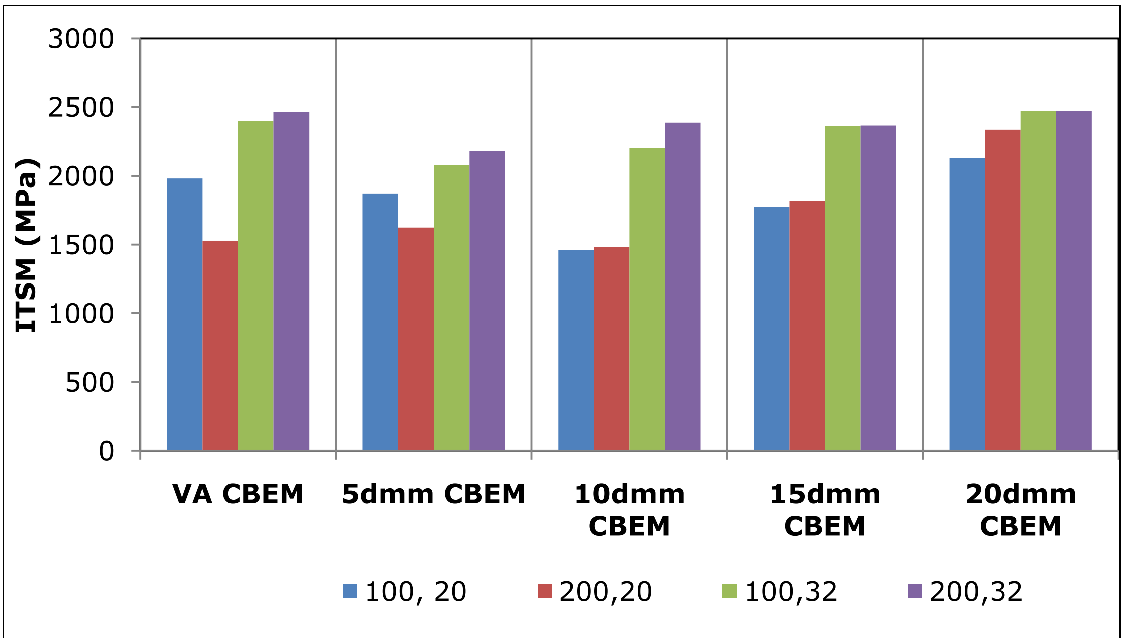


Figure 6.12: ITSM of CBEMs at Simulated Intermediate Life Condition

Table 6.13: Percentage Improvements in ITSM of other CBEMs over the VACBEM at Simulated Intermediate Life Condition

Applied Treatment (Gyrations, Temp.)	5dmm CBEM	10dmm CBEM	15dmm CBEM	20dmm CBEM
100, 20	-5.7	-26.4	-10.5	7.4
200,20	6.3	-2.8	19.0	52.9
100,32	-13.3	-8.2	-1.5	3.1
200,32	-11.5	-3.2	-4.0	0.3

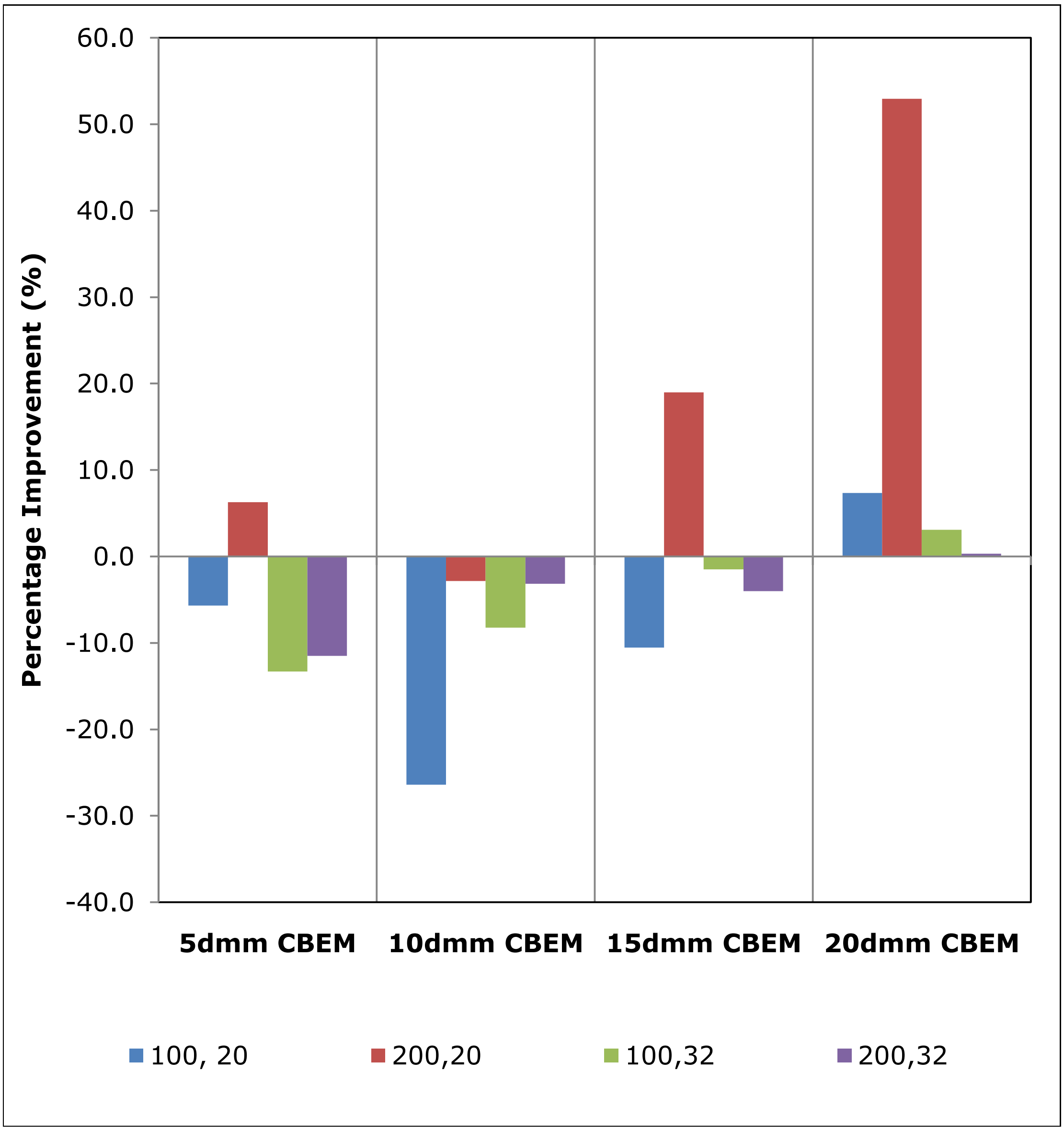


Figure 6.13: Percentage Improvement of other CBEMs in ITSM over VA CBEM at Simulated Intermediate Life Condition

weaken the skeleton of the matrix responsible for strength and stiffness.

The ITSM responses of the 10dmmCBEM, 15dmmCBEM and 20dmmCBEM at these mixing and compaction temperatures with similar compactive efforts were logical though with a significant difference noticed in the latter of the three. This suggests that the 10dmm, 15dmm and 20dmm RAPs were not completely acting as black rock at these temperatures, with improvement as penetration value of the residual bitumen increases. These observations which are probably attributable to the fact that the CBEMs are still evolving at this stage confirm the views expressed by other researchers (Ibrahim, 1998; and Thanaya, 2003) on CBEMs that they are complex materials.

6.8.1.3 ITSM and CBEM composition at fully cured condition

A further investigation though similar in procedure to that of the preceding section was conducted on fully cured specimens. The results of the One-Sample Kolmogorov-Smirnov (Normality) Test on the measured ITSM values for simulated fully cured condition of CBEMs are detailed in Table 6.14. The results indicate a normal distribution for the measurements as all the values in Asymp. Sig. (2-tailed) row were greater than 0.05. Figure 6.14 shows the general ITSM responses of the CBEMs at fully cured condition. The trend of ITSM responses as observed in the figure is clearer here than was in the preceding section. All ITSM responses seem to be logical here (irrespective of mode of manufacture). Table 6.15 and Figure 6.15 show a similar trend for percentage improvements of RAP CBEMs in ITSM over the VACBEM. As observed here, the VACBEM was superior in ITSM response compared to the 5dmmCBEM. Also, there is an indication of better performance over the 10dmmCBEM and 15dmmCBEM by the VACBEM generally at the mixing and compaction temperature of 20ºC irrespective of the compactive effort applied.

Although the 20dmmCBEM significantly improved here compared to the intermediate life ITSM responses, the figure indicates the best response as expected at mixing and compaction temperature of 32ºC but at the lower compactive effort of 100 gyrations. Meanwhile, as more bars are now on the positive side of percentage improvement over the VACBEM when compared to those of the preceding section, it therefore suggests that the CBEM is still improving with increasing binder penetration and only further research validated by field results can confirm where this ultimately stops.

Table 6.14: One-Sample Kolmogorov-Smirnov (Normality) Test for ITSM at Simulated Fully Cured Condition of CBEMs

Applied Treatment (Gyrations, Temp.)	Parameters		ITSM				
			VA CBEM	5dmm CBEM	10dmm CBEM	15dmm CBEM	20dmm CBEM
100, 20	Sample Size		4	4	4	4	4
	Normal Parameters[a]	Mean	2110	1631	1729	1931	2620
		SD	355	315	39	185	288
	Kolmogorov-Smirnov Z		0.811	0.264	0.628	0.749	0.527
	Asymp. Sig. (2-tailed)		0.527	1.000	0.825	0.630	0.944
200, 20	Sample Size		4	20	20	4	20
	Normal Parameters[a]	Mean	2174	1838	1764	2006	2826
		SD	424	232	129	185	283
	Kolmogorov-Smirnov Z		0.538	0.689	0.755	0.388	0.632
	Asymp. Sig. (2-tailed)		0.935	0.730	0.619	0.998	0.819
100, 32	Sample Size		12	12	12	12	12
	Normal Parameters[a]	Mean	2620	2235	2714	3043	3663
		SD	660	248	344	308	324
	Kolmogorov-Smirnov Z		0.567	0.595	0.570	0.415	0.547
	Asymp. Sig. (2-tailed)		0.904	0.870	0.901	0.995	0.926
200, 32	Sample Size		44	64	64	44	64
	Normal Parameters[a]	Mean	2882	2548	3108	3607	3934
		SD	567	288	480	501	345
	Kolmogorov-Smirnov Z		0.560	0.744	0.665	0.492	0.880
	Asymp. Sig. (2-tailed)		0.913	0.636	0.769	0.969	0.421

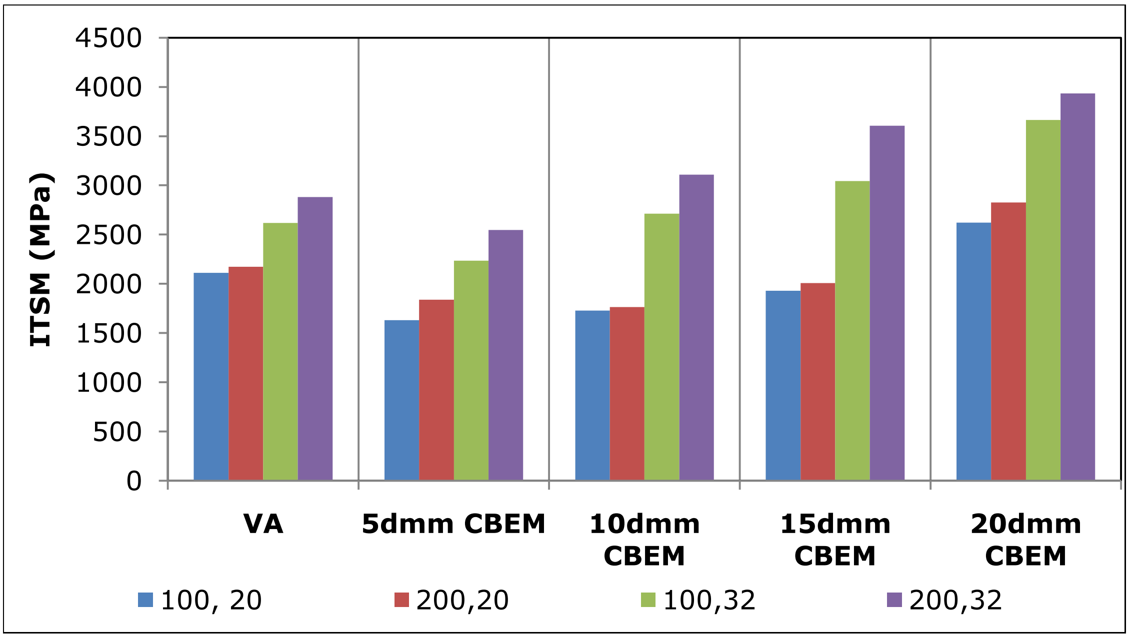


Figure 6.14: ITSM of CBEMs at Simulated Fully Cured Condition

Table 6.15: Percentage Improvements in ITSM of RAP CBEMs over the VA CBEM at Simulated Intermediate Life Condition

Applied Treatment (Gyrations, Temp.)	5dmm CBEM	10dmm CBEM	15dmm CBEM	20dmm CBEM
100, 20	-22.7	-18.1	-8.5	24.2
200,20	-15.5	-18.9	-7.7	30.0
100,32	-14.7	3.6	16.2	39.8
200,32	-11.6	7.9	25.2	36.5

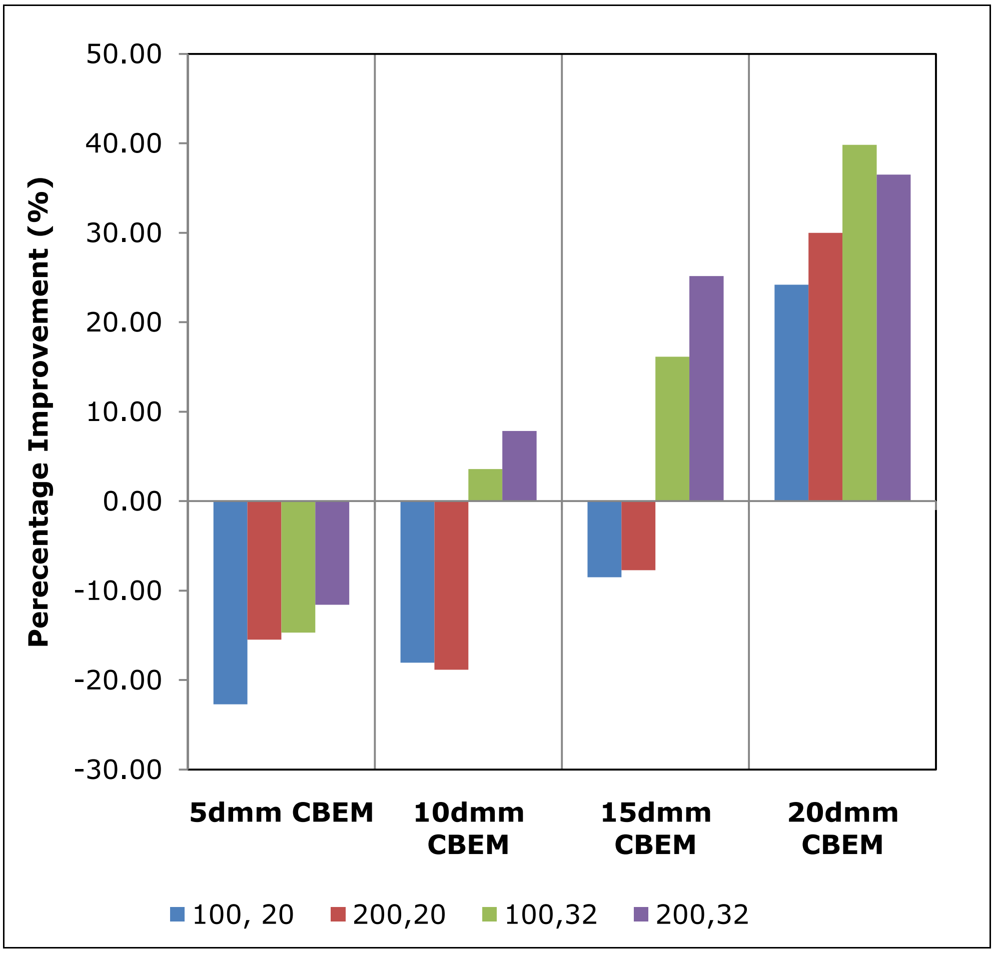


Figure 6.15: Percentage Improvement of RAP CBEMs in ITSM over VACBEM at Simulated Fully Cured Condition

The observed trend again confirms that CBEMs are complex materials. The ITSM responses indicate that they are in some cases not clearly predictable especially when severely aged RAPs such as the 5dmm and 10dmm RAPs are involved. Although the ITSM test clearly ranked the CBEMs in early life as earlier observed, since the trends observed for subsequent simulated life conditions were in some cases not clear, characterisation of the mechanical performance of CBEMs by the ITSM test should be complemented with other tests to gain more knowledge on the behaviour of the materials. Thus the good performance of VACBEM as observed in the preceding section (intermediate life condition) and even at this fully cured condition over some of the RAP CBEMs should be interpreted with caution. The very high air void contents consistently observed for VACBEM in sections 6.7.1 and 6.7.2 of this chapter compared to the RAP CBEMs obviously predisposes it to problems associated with durability in bituminous mixtures.

6.8.2 Temperature and ITSM response of CBEMs

Ascertaining the temperature susceptibility of bituminous materials is required in order to have a good knowledge of such materials in service under extreme conditions. Naturally bitumen is temperature susceptible i.e. becomes brittle at low temperatures and gets increasingly less viscous and ready to flow at high temperatures. While low temperatures make such materials susceptible to cracking they are prone to rutting at high temperatures. Under such hot conditions too, oxidative ageing is enhanced especially when the mixture is highly porous. A properly constituted mixture however should be able to weather through these extremes without necessarily reducing the overall structural integrity of the road pavement.

An attempt was thus made to ascertain the thermal susceptibilities of the five CBEMS. Only CBEMs that simulate intermediate life and fully cured conditions were considered. Materials simulating early life were excluded due to their fragile nature. The study here examined the effects of CBEM material type, mixing and compaction temperature and compactive effort on the stiffness properties of the materials. Four specimens were studied at each observation level. In order to reduce variations, the same sets of specimens were studied at each of the chosen testing temperatures of 20ºC, 30ºC and 40ºC. These testing temperatures have been considered appropriate based on the pavement temperature prediction model suggested earlier in Chapter 5 i.e.

$$T_{surf} = T_{air} - 0.00618\ lat^2 + 0.2289\ lat + 24.4 \qquad 6.14$$

$$T_d = T_{surf}\ (1 - 0.063d + 0.007\ d^2 - 0.0004d^3) \qquad 6.15$$

Where

T_{air} = Average yearly ambient air temperature (32ºC for Max and 21ºC for Min; Nigerian conditions, BBC Weather, 2008)

T_{surf} = Temperature at the surface of pavement (in ºF)

T_d = Temperature at the desired depth in pavement (in ºF)

d = Desired depth in pavement (in inches)

lat = Latitude of location (7.4º for Ibadan, Nigeria)

At the stated temperatures for T_{air} (maximum and minimum alike) and a pavement depth of 150mm (taking an overlay of 50mm, and a road base of 200mm for example),

T_{surf} maximum = 57.76ºC
T_{surf} minimum = 46.76ºC
$T_{d(150mm)}$ maximum = 41.76ºC
$T_{d(150mm)}$ minimum = 33.2ºC

These were rounded to 30ºC and 40ºC for ease. A temperature of 20ºC was included being the standard for ITSM tests.

Table 6.16 details a summary of the ITSM results along with the respective average air void contents of the specimens. Figures 6.16 to 6.19 are relevant here. Figures 6.16 and 6.18 detail the ITSM responses of CBEMs conditioned to simulate intermediate life while Figures 6.17 and 6.19 depict the ITSM responses for those conditioned to simulate fully cured. Table 6.17 details the percentage loss in ITSM of CBEMs due to temperature increase.

The relevant figures for intermediate life show that the ITSM responses for the CBEMs were generally below 2500MPa irrespective of testing temperature. This observation is not unexpected as the CBEMs are still evolving at this point. The CBEMs did not follow a distinct trend at intermediate life when compared to those that had been fully cured. Also the effect of mixing and compaction temperature is not as clear as observed for the fully cured condition.

ITSM responses tested at 20ºC for fully cured CBEMs mixed and compacted at 32ºC were generally above 2500MPa (with the exception of the 5dmmCBEM) irrespective of the compactive effort applied as observed in Figures 6.17 and 6.19. However, the responses dropped to around 1500MPa for ITSM tests

Table 6.16: Summary of ITSM Results for Temperature Susceptibility of CBEMs

CBEM Condition	Testing Condition	VA CBEM	5dmm CBEM	10dmm CBEM	15dmm CBEM	20dmm CBEM
I.L. (32,100)	ITSM @20ºC (MPa)	2424	2071	2280	2387	2448
	ITSM @30ºC (MPa)	1298	1058	1265	1486	1248
	ITSM @40ºC (MPa)	591	483	512	631	501
	Ave. Air voids (%)	17.6	16.5	15.1	12.9	12.9
I.L. (20,100)	ITSM @20ºC (MPa)	1982	1869	1459	1773	2127
	ITSM @30ºC (MPa)	852	934	705	1007	1109
	ITSM @40ºC (MPa)	378	386	317	388	467
	Ave. Air voids (%)	18.7	18.7	19.9	17.2	15.9
I.L. (32, 200)	ITSM @20ºC (MPa)	2428	2190	2383	2378	2510
	ITSM @30ºC (MPa)	1195	1108	1307	1260	1333
	ITSM @40ºC (MPa)	495	442	444	585	493
	Ave. Air voids (%)	15.7	15.8	14.1	12.5	11.5
I.L. (20,200)	ITSM @20ºC (MPa)	1526	1623	1483	1817	2335
	ITSM @30ºC (MPa)	576	682	778	1014	1124
	ITSM @40ºC (MPa)	313	317	298	377	455
	Ave. Air voids (%)	19.1	18.4	18.3	15.8	14.5
F.C. (32,100)	ITSM @20ºC (MPa)	2704	2267	2752	3085	3665
	ITSM @30ºC (MPa)	1389	1306	1576	1920	1663
	ITSM @40ºC (MPa)	550	441	606	694	702
	Ave. Air voids (%)	17.2	16.3	15.9	13.3	13.0
F.C. (20,100)	ITSM @20ºC (MPa)	2110	1631	1729	1930	2620
	ITSM @30ºC (MPa)	1108	723	944	1094	1337
	ITSM @40ºC (MPa)	451	378	391	430	581
	Ave. Air voids (%)	19.6	19.0	19.6	18.6	16.5
F.C. (32, 200)	ITSM @20ºC (MPa)	2860	2584	3216	3557	3900
	ITSM @30ºC (MPa)	1502	1374	1732	1839	1816
	ITSM @40ºC (MPa)	644	482	738	752	667
	Ave. Air voids (%)	17.7	15.3	13.3	12.6	11.9
F.C. (20,200)	ITSM @20ºC (MPa)	2174	1814	1787	2006	2826
	ITSM @30ºC (MPa)	1046	926	994	1137	1404
	ITSM @40ºC (MPa)	463	436	408	413	555
	Ave. Air voids (%)	18.1	18.1	18.7	17.3	15.3

*Note: **F.C.** = Fully Cured; **I.L.** = Intermediate Life*
Numbers in parenthesis = mixing and compaction temperature in ºC, number of gyrations.

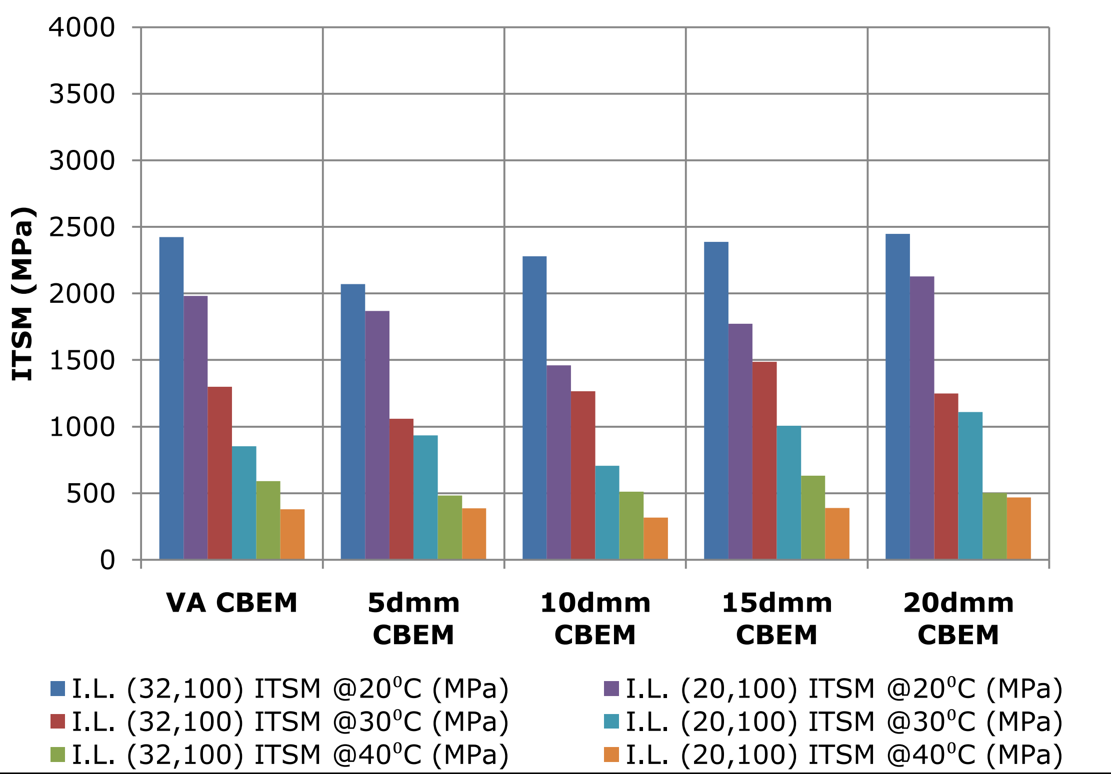


Figure 6.16: A Comparison of ITSM Responses of CBEMs Manufactured at Mixing and Compaction Temperatures of 32°C and 20°C with 100 gyrations - Conditioned to Intermediate Life

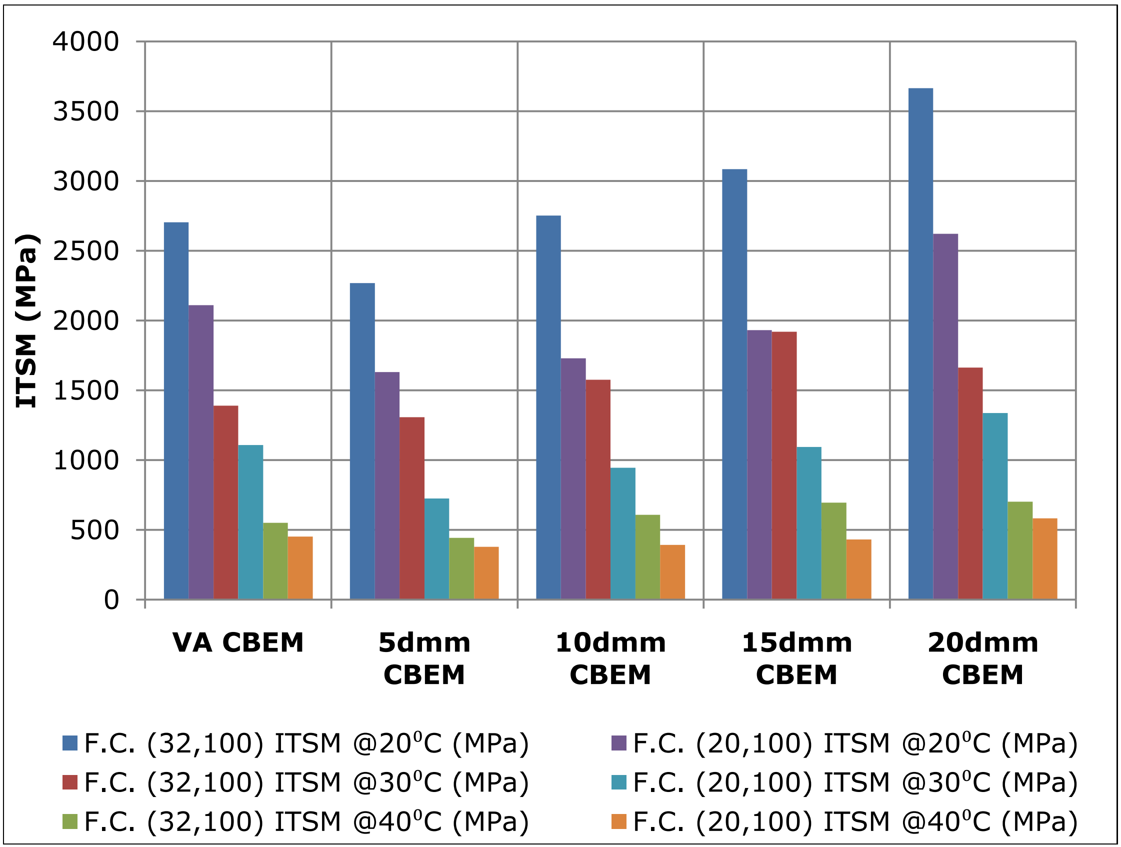


Figure 6.17: A Comparison of ITSM Responses of CBEMs Manufactured at Mixing and Compaction Temperatures of 32°C and 20°C with 100 gyrations - Conditioned to Fully Cured

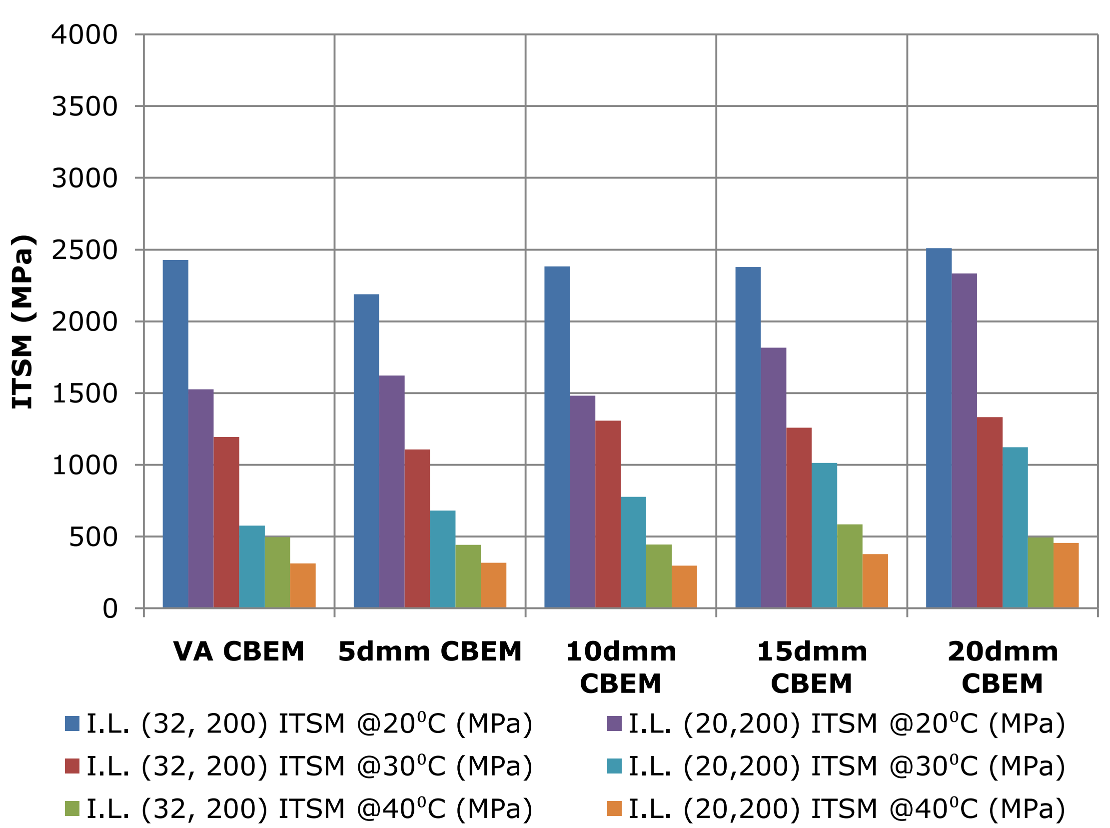


Figure 6.18: A Comparison of ITSM Responses of CBEMs Manufactured at Mixing and Compaction Temperatures of 32°C and 20°C with 200 gyrations - Conditioned to Intermediate Life

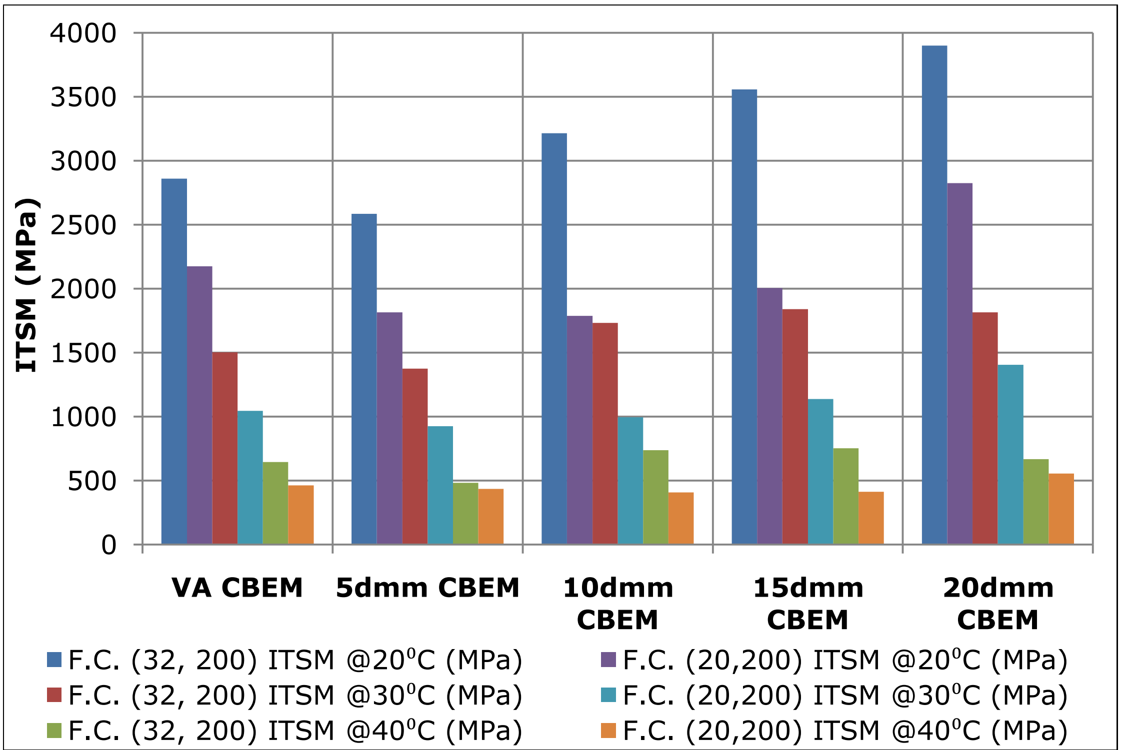


Figure 6.19: A Comparison of ITSM Responses of CBEMs Manufactured at Mixing and Compaction Temperatures of 32°C and 20°C with 200 gyrations - Conditioned to Fully Cured

Table 6.17: Percentage Loss in ITSM of CBEMs due to Temperature Increase

CBEM Condition	Percentage Loss Description (%)	VA CBEM	5dmm CBEM	10dmm CBEM	15dmm CBEM	20dmm CBEM
I.L. (32,100)	Percentage Loss in ITSM from 20ºC to 30ºC	46.5	48.9	44.5	37.8	49.0
	Percentage Loss in ITSM from 20ºC to 40ºC	75.6	76.7	77.6	73.6	79.5
I.L. (20,100)	Percentage Loss in ITSM from 20ºC to 30ºC	57.0	50.0	51.7	43.2	47.9
	Percentage Loss in ITSM from 20ºC to 40ºC	80.9	79.4	78.3	78.1	78.0
I.L. (32, 200)	Percentage Loss in ITSM from 20ºC to 30ºC	50.8	49.4	45.2	47.0	46.9
	Percentage Loss in ITSM from 20ºC to 40ºC	79.6	79.8	81.4	75.4	80.4
I.L. (20,200)	Percentage Loss in ITSM from 20ºC to 30ºC	62.3	58.0	47.5	44.2	51.9
	Percentage Loss in ITSM from 20ºC to 40ºC	79.5	80.5	79.9	79.2	80.5
F.C. (32,100)	Percentage Loss in ITSM from 20ºC to 30ºC	48.6	42.4	42.7	37.8	54.6
	Percentage Loss in ITSM from 20ºC to 40ºC	79.7	80.5	78.0	77.5	80.9
F.C. (20,100)	Percentage Loss in ITSM from 20ºC to 30ºC	47.5	55.7	45.4	43.3	49.0
	Percentage Loss in ITSM from 20ºC to 40ºC	78.6	76.8	77.4	77.7	77.8
F.C. (32, 200)	Percentage Loss in ITSM from 20ºC to 30ºC	47.5	46.8	46.2	48.3	53.4
	Percentage Loss in ITSM from 20ºC to 40ºC	77.5	81.4	77.0	78.9	82.9
F.C. (20,200)	Percentage Loss in ITSM from 20ºC to 30ºC	51.9	49.0	44.4	43.3	50.3
	Percentage Loss in ITSM from 20ºC to 40ºC	78.7	76.0	77.2	79.4	80.3

Note: ***F.C.*** *= Fully Cured;* ***I.L.*** *= Intermediate Life*
Numbers in parenthesis = mixing and compaction temperature, number of gyrations

conducted at 30ºC and further down to around 650MPa for tests conducted at 40ºC on the same specimens. The 5dmmCBEM showed the least good performance for the three test temperatures.

The four graphs again confirm the observations made earlier on the superiority of CBEMs prepared at 32ºC compared to those of 20ºC. Irrespective of the CBEM type, the condition of curing and testing temperature for ITSM, CBEMs prepared at 32ºC consistently performed better than those prepared at 20ºC even at ITSM test temperature of 40ºC.

Overall, the figures indicate that all the materials prepared at 32ºC, 100 gyrations have least loss in ITSM which suggests just enough compactive effort along with appropriate temperature for preparation of CBEMs. More importantly, the VACBEM as observed here indicates that it has the highest temperature susceptibility.

6.8.3 Moisture susceptibility of CBEMs

This exercise naturally follows since the temperature susceptibility test at 30ºC and 40ºC did not yield clear enough results for appropriate ranking of the CBEMs. A moisture susceptibility test was considered appropriate since the CBEMs are subjected to the combined effects of temperature and moisture, two of the known agents responsible for altering the durability of bituminous mixtures. More importantly, moisture susceptibility of bituminous materials is generally agreed among researchers as an indicative means of predicting the durability of bituminous mixtures while in service. The presence of moisture in some bituminous mixtures is known to also promote stripping.

The ITSM test just as was done for the temperature susceptibility test was used here. Due to shortness of time and materials, the test was limited to specimens prepared for intermediate life and fully cured conditions at the mixing and compaction temperature of 32ºC. Two protocols were used for the test. The first involved soaking specimens in water at 20ºC for 24hrs followed by testing for ITSM. The ITSM tests were conducted at 20ºC (mild condition) first and then at 30ºC (severe condition) after specimens had been conditioned for 2hrs at that temperature. Prior to soaking, the specimens were tested dry at 20ºC for ITSM. The second protocol similarly involved soaking specimens at 30ºC for 24hrs. These were followed by ITSM testing at 30ºC (severe condition) first and then at 20ºC (mild condition) after the specimens have been conditioned at that temperature for 2hrs. Four specimens were studied at each level of observation making a total of 32 specimens tested for each of the CBEMs. It is worth mentioning that prior to soaking, vacuum saturation of all the specimens was carried out at a pressure of 140mbar for 10 minutes. Meanwhile, from physical observation, the vacuum saturation seemed not to have affected the specimens since none of the specimens bulged or collapsed/disintegrated in the process.

Tables 6.18a and 6.18b detail the ITSM responses for water susceptibility test for Protocols 1 and 2 respectively and the air void content of the CBEMs tested. A close inspection of the two tables alongside Figures 6.20a, 6.20b and 6.20c indicate that the VACBEM has the poorest performance for the severe conditions irrespective of the preparation method though with an exception to materials prepared at 32^{0}C, 200 gyrations in Protocol 1 (Table 6.18a), where it indicates a slightly better ITSM response than the 5dmmCBEM. For Protocol 2 which simulated the recovery of the CBEMs after experiencing a severe condition, the ITSM responses indicated a poor recovery in stiffness for the VACBEM compared to the others except for materials prepared at 32^{0}C, 100 gyrations where it performed reasonably well though only a little better than the 5dmmCBEM. For the two protocols, materials prepared using the higher compactive effort clearly performed better than those with less compactive effort except for the intermediate life in Protocol 1 where the curves were indicating same performance.

Although the two protocols have proved useful by clearly ranking the CBEMs, Protocol 2 appeals more as it also simulates and assesses performance after the recovery process of the CBEMs which is very important for performance in service. However in order to ascertain this, percentage in loss for each individual CBEM was further investigated. The percentage loss was relative to the ITSM response at 20^{0}C (dry).

Tables 6.19a and 6.19b detail the percentage losses in ITSM of the CBEMs as a result of the combined damaging effects of water and temperature. Figures 6.21 to 6.24 show graphically the percentage losses in ITSM values for CBEMs under Protocol 1 for the water susceptibility test when compared to the values obtained at 20^{0}C though tested dry. Figures 6.25 to 6.28 detail similar percentage losses for Protocol 2.

The four figures for Protocol 1 indicate a clear and logical trend for all the CBEMs. For all the conditions investigated here, the VACBEM clearly indicate the worst performance recording the highest losses both for the severe and mild conditions. In fact for the mild conditions, values as high as 52% loss were

Table 6.18a: ITSM Results of Water Susceptibility of CBEMs for Protocol 1

CBEM Condition	Testing Condition	VA CBEM	5dmm CBEM	10dmm CBEM	15dmm CBEM	20dmm CBEM
I.L. (32,100)	At 20⁰C Dry	2368	2089	2169	2360	2474
	At 20⁰C to 20⁰C Wet	1170	1422	1655	2263	2169
	At 20⁰C to 30⁰C Wet	390	422	496	685	661
	Air voids (%)	17.3	16.4	15.7	13.3	11.8
I.L. (32,200)	At 20⁰C Dry	2478	2203	2395	2381	2486
	At 20⁰C to 20⁰C Wet	1147	1430	1687	2251	2315
	At 20⁰C to 30⁰C Wet	358	436	576	638	697
	Air voids (%)	16.6	15.2	14.1	12.2	10.3
F.C. (32,100)	At 20⁰C Dry	2546	2183	2659	3016	3670
	At 20⁰C to 20⁰C Wet	1435	1554	2061	2707	2869
	At 20⁰C to 30⁰C Wet	489	529	813	939	1093
	Air voids (%)	18.7	16.3	16.3	13.5	13.1
F.C.(32,200)	At 20⁰C Dry	2918	2531	3173	3652	3949
	At 20⁰C to 20⁰C Wet	1616	1893	2562	3008	3680
	At 20⁰C to 30⁰C Wet	634	621	866	1238	1419
	Air voids (%)	16.7	16.1	13.4	12.9	12.0

Table 6.18b: ITSM Results of Water Susceptibility of CBEMs for Protocol 2

CBEM Condition	Testing Condition	VA CBEM	5dmm CBEM	10dmm CBEM	15dmm CBEM	20dmm CBEM
I.L. (32,100)	At 20⁰C Dry	2402	2075	2153	2340	2495
	At 30⁰C Dry	948	1108	1079	1352	1426
	30⁰C to 30⁰C Wet	267	391	522	706	624
	30⁰C to 20⁰C Wet	479	908	1089	1486	1502
	Air voids (%)	17.1	16.2	16.0	12.9	12.1
I.L. (32,200)	At 20⁰C Dry	2485	2146	2379	2336	2500
	At 30⁰C Dry	1360	1067	1315	1357	1544
	30⁰C to 30⁰C Wet	273	569	670	777	778
	30⁰C to 20⁰C Wet	607	1173	1372	1550	1811
	Air voids (%)	17.2	16.2	15.0	12.9	11.6
F.C. (32,100)	At 20⁰C Dry	2608	2253	2730	3027	3655
	At 30⁰C Dry	1520	1188	1515	1690	1620
	30⁰C to 30⁰C Wet	588	601	852	1055	1067
	30⁰C to 20⁰C Wet	1485	1373	1778	2264	2603
	Air voids (%)	16.8	16.9	15.0	12.9	12.1
F.C.(32,200)	At 20⁰C Dry	2901	2575	3105	3654	3901
	At 30⁰C Dry	1476	1254	1609	1884	1882
	30⁰C to 30⁰C Wet	658	816	847	1152	1372
	30⁰C to 20⁰C Wet	1359	1641	1966	2319	2859
	Air voids (%)	15.8	15.1	14.3	12.9	11.8

*Note: **F.C.** = Fully Cured; **I.L.** = Intermediate Life*
Numbers in parenthesis = mixing and compaction temperature, number of gyrations

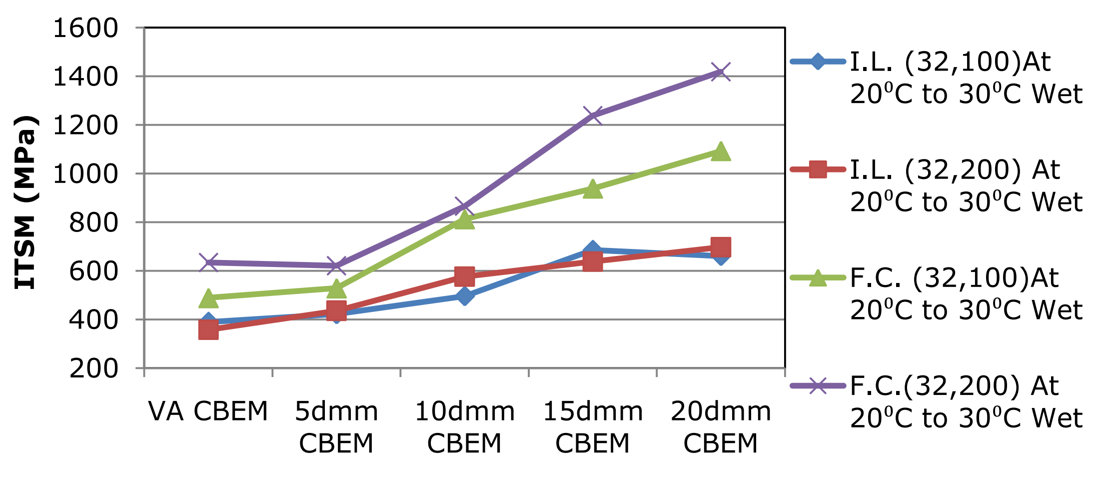


Figure 6.20a: Protocol 1- ITSM Responses for the Severe Condition (ITSM at 30⁰C after soaking for 24hrs at 20⁰C and 2hrs at 30⁰C)

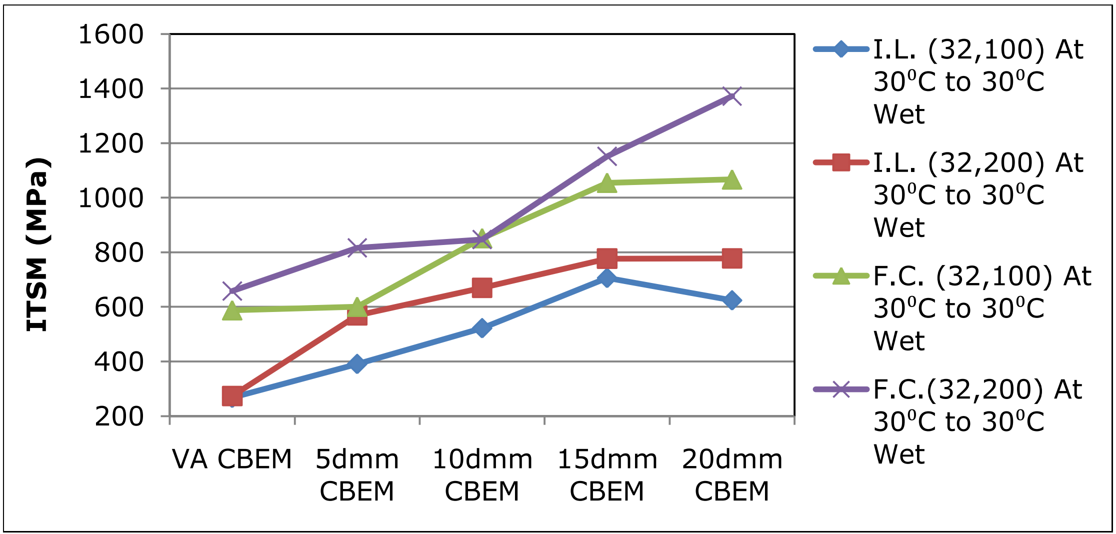


Figure 6.20b: Protocol 2- ITSM Responses for the Severe Condition (ITSM at 30⁰C after soaking for 24hrs at 30⁰C)

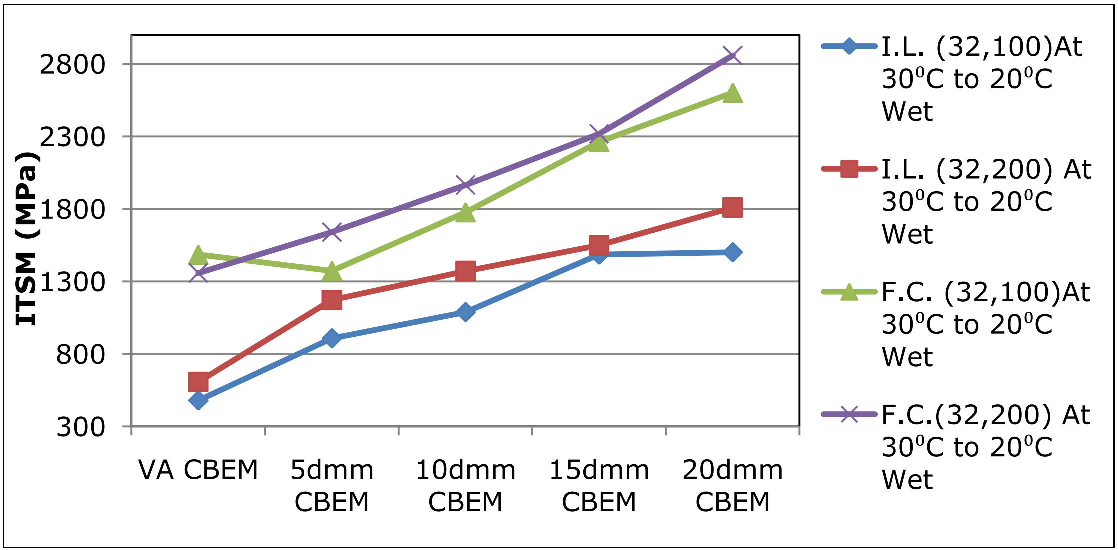


Figure 6.20c: Protocol 2- ITSM Responses for Recovery Path after Exposure to the Severe Condition (ITSM at 20⁰C after soaking for 24hrs at 30⁰C and 2hrs at 20⁰C)

Table 6.19a: Results of Percentage Loss in ITSM due to Water Susceptibility for Protocol 1

CBEM Condition	Percentage Loss Description	VA CBEM	5dmm CBEM	10dmm CBEM	15dmm CBEM	20dmm CBEM
I.L. (32^{0}C, 100)	Percentage Loss in ITSM from 20^{0}C Dry to 20^{0}C Wet	50.6	31.9	23.7	4.1	12.3
	Percentage Loss in ITSM from 20^{0}C Dry to 30^{0}C Wet	83.5	79.8	77.1	71.0	73.3
I.L. (32^{0}C, 200)	Percentage Loss in ITSM from 20^{0}C Dry to 20^{0}C Wet	53.7	35.1	29.6	5.4	6.9
	Percentage Loss in ITSM from 20^{0}C Dry to 30^{0}C Wet	85.6	80.2	75.9	73.2	72.0
F.C. (32^{0}C, 100)	Percentage Loss in ITSM from 20^{0}C Dry to 20^{0}C Wet	43.6	28.8	22.5	10.3	21.8
	Percentage Loss in ITSM from 20^{0}C Dry to 30^{0}C Wet	80.8	75.8	69.4	68.9	70.2
F.C. (32^{0}C, 200)	Percentage Loss in ITSM from 20^{0}C Dry to 20^{0}C Wet	44.6	25.2	19.2	17.6	6.8
	Percentage Loss in ITSM from 20^{0}C Dry to 30^{0}C Wet	78.3	75.5	72.7	66.1	64.1

Table 6.19b: Results of Percentage Loss in ITSM due to Water Susceptibility for Protocol 2

CBEM Condition	Percentage Loss Description (%)	VA CBEM	5dmm CBEM	10dmm CBEM	15dmm CBEM	20dmm CBEM
I.L. (32^{0}C, 100)	Percentage Loss in ITSM from 20^{0}C Dry to 30^{0}C Dry	60.5	46.6	49.9	42.2	42.8
	Percentage Loss in ITSM from 20^{0}C Dry to 30^{0}C Wet	88.9	81.2	75.8	69.8	75.0
	Percentage Loss in ITSM from 20^{0}C Dry to 30^{0}C - 20^{0}CWet	80.1	56.2	49.4	36.5	39.8
I.L. (32^{0}C, 200)	Percentage Loss in ITSM from 20^{0}C Dry to 30^{0}C Dry	45.3	50.3	44.7	41.9	38.2
	Percentage Loss in ITSM from 20^{0}C Dry to 30^{0}C Wet	89.0	73.5	71.9	66.8	68.9
	Percentage Loss in ITSM from 20^{0}C Dry to 30^{0}C - 20^{0}CWet	75.6	45.4	42.3	33.7	27.6
F.C. (32^{0}C, 100)	Percentage Loss in ITSM from 20^{0}C Dry to 30^{0}C Dry	41.7	47.3	44.5	44.2	55.7
	Percentage Loss in ITSM from 20^{0}C Dry to 30^{0}C Wet	77.4	73.3	68.8	65.2	70.8
	Percentage Loss in ITSM from 20^{0}C Dry to 30^{0}C - 20^{0}CWet	43.0	39.0	34.9	25.2	28.8
F.C. (32^{0}C, 200)	Percentage Loss in ITSM from 20^{0}C Dry to 30^{0}C Dry	49.1	51.3	48.2	48.5	51.7
	Percentage Loss in ITSM from 20^{0}C Dry to 30^{0}C Wet	77.3	68.3	72.7	68.5	64.8
	Percentage Loss in ITSM from 20^{0}C Dry to 30^{0}C - 20^{0}CWet	53.2	36.3	36.7	36.6	26.7

*Note: **F.C.** = Fully Cured; **I.L.** = Intermediate Life*
Numbers in parenthesis = mixing and compaction temperature, number of gyrations

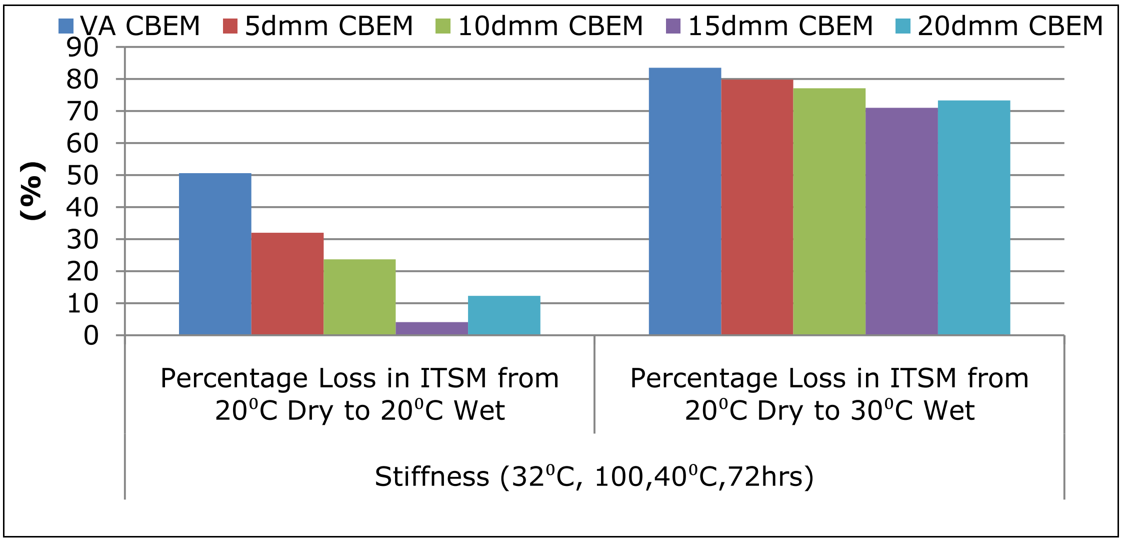


Figure 6.21: Water Susceptibility of CBEMs at Intermediate Life Condition Mixed and Compacted at 32^{0}C, 100 gyrations (Protocol 1)

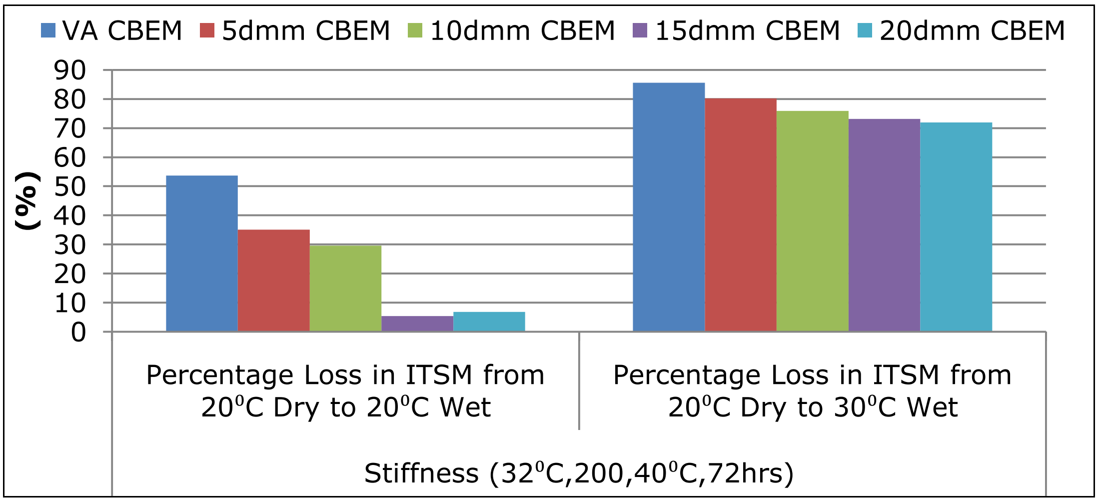


Figure 6.22: Water Susceptibility of CBEMs at Intermediate Life Condition Mixed and Compacted at 32^{0}C, 200 gyrations (Protocol 1)

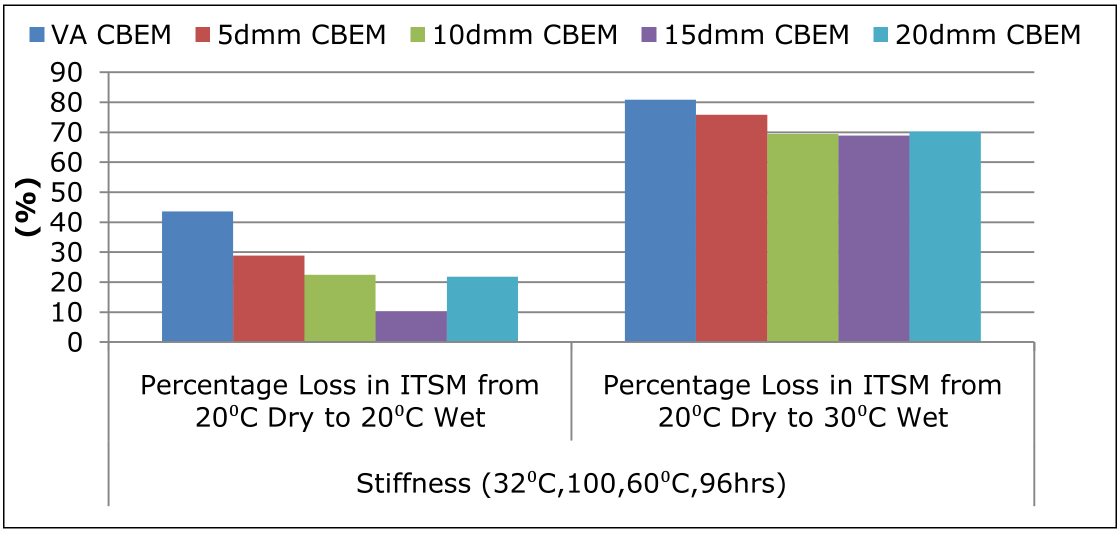


Figure 6.23: Water Susceptibility of CBEMs at Fully Cured Condition Mixed and Compacted at 32^{0}C, 100 gyrations (Protocol 1)

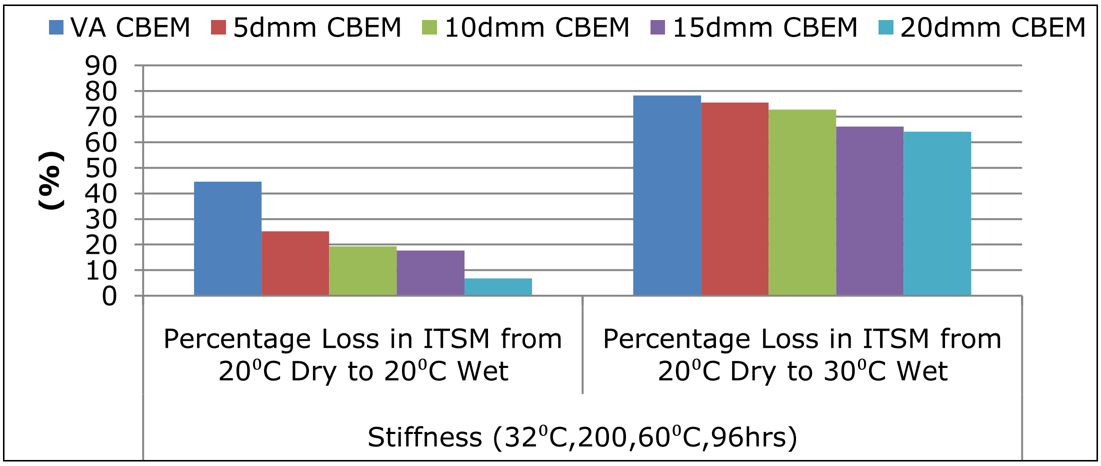


Figure 6.24: Water Susceptibility of CBEMs at Fully Cured Condition Mixed and Compacted at 32°C, 200 gyrations (Protocol 1)

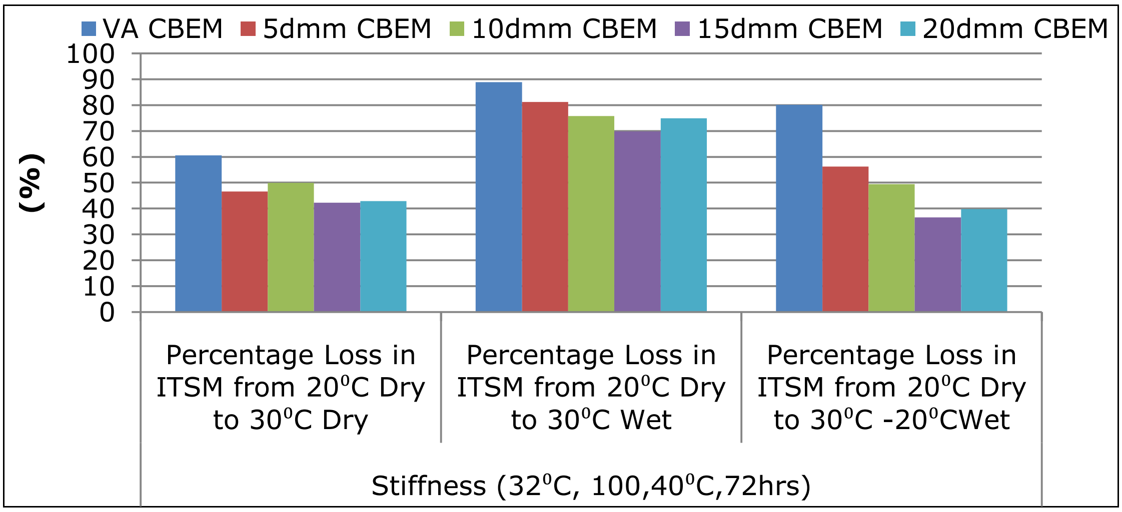


Figure 6.25: Water Susceptibility of CBEMs at Intermediate Life Condition Mixed and Compacted at 32°C, 100 gyrations (Protocol 2)

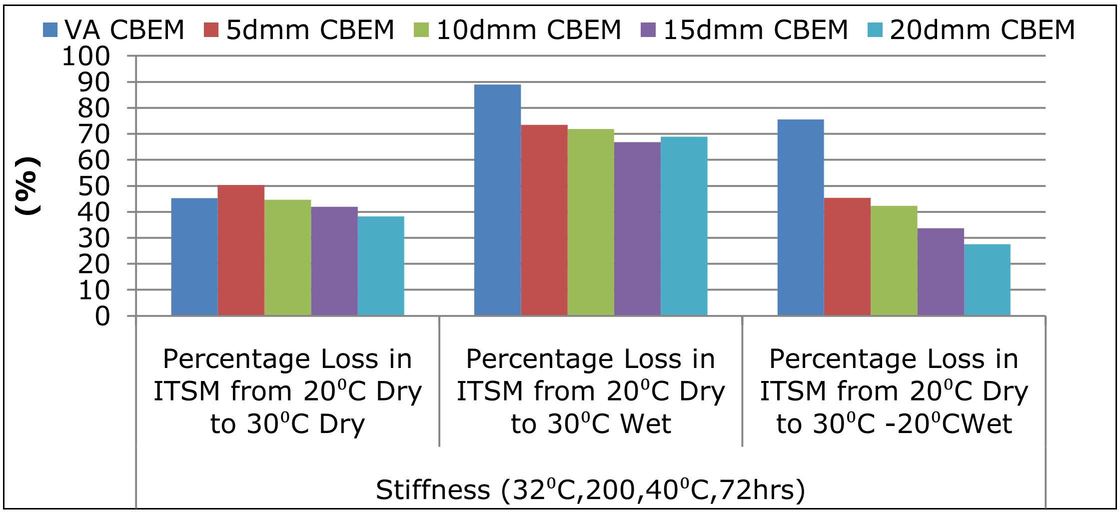


Figure 6.26: Water Susceptibility of CBEMs at Intermediate Life Condition Mixed and Compacted at 32°C, 200 gyrations (Protocol 2)

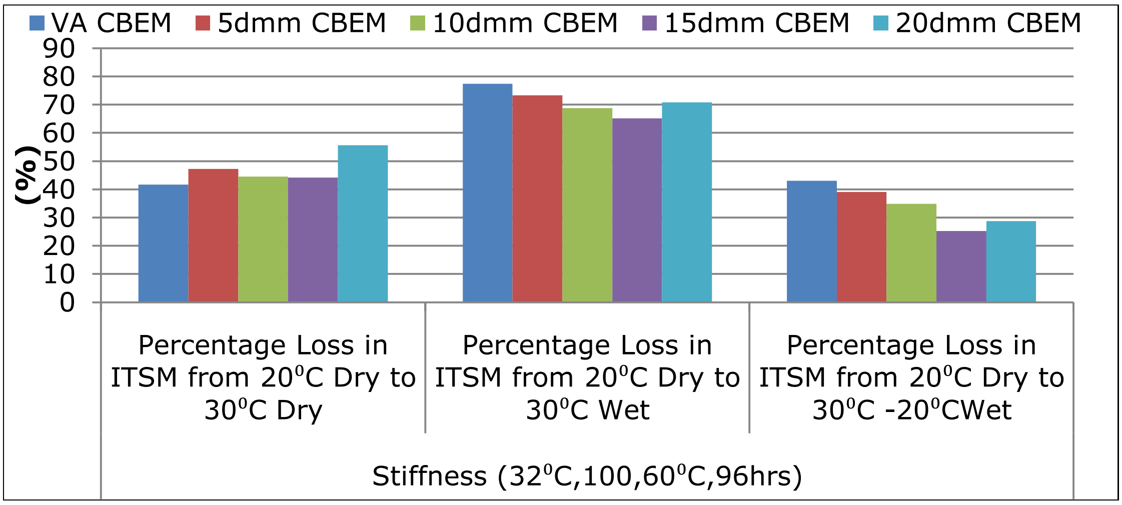


Figure 6.27: Water Susceptibility of CBEMs at Fully Cured Condition Mixed and Compacted at 32^{0}C, 100 gyrations (Protocol 2)

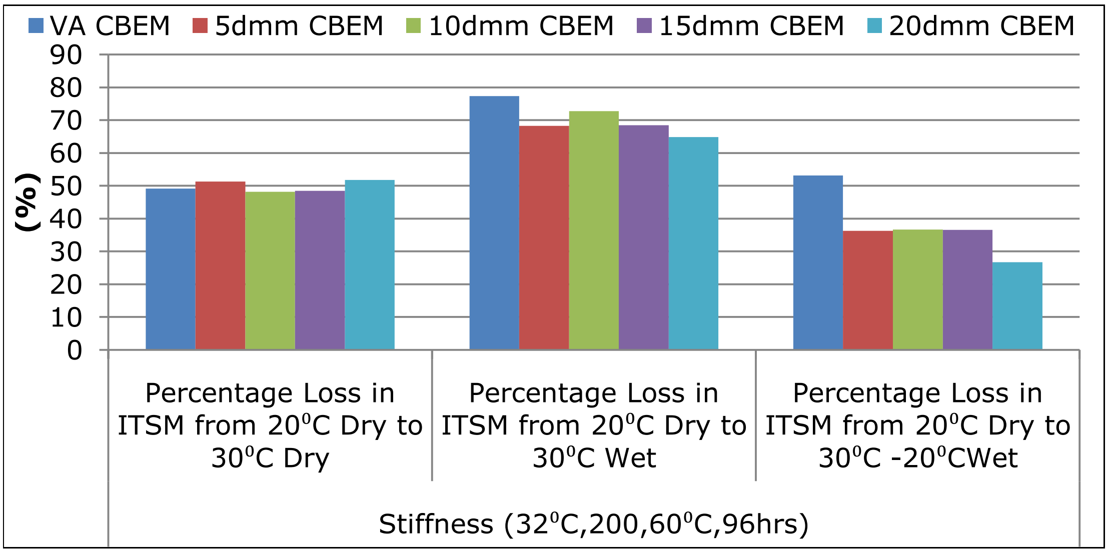


Figure 6.28: Water Susceptibility of CBEMs at Fully Cured Condition Mixed and Compacted at 32^{0}C, 200 gyrations (Protocol 2)

observed for VACBEM, while the 5dmmCBEM which is next in rank recorded a little above 30% loss for the same conditions. For the higher compactive effort, the 20mmCBEM consistently recorded the least loss i.e. the best performance for the severe condition of testing irrespective of the curing condition applied on the CBEMs. Although Figure 6.24 which is for materials prepared at 32^{0}C and 200 gyrations (fully cured) clearly ranked the CBEMs for both the mild and severe conditions of testing in a logical order starting with VACBEM up the range to the 20dmmCBEM, the 15dmmCBEM contrary to logic recorded the overall best performance. The same figure indicated that CBEMs prepared at this level have the best resistance under severe conditions.

The Figures 6.25 to 6.28 for Protocol 2 detail responses for both temperature susceptibility and water damaging effects on the CBEMs. This has been purposely done in order to ascertain the effectiveness of the temperature susceptibility test at being able to rank the materials. It is obvious from the figures that measurements for the temperature susceptibility did not follow a clear order compared to the water damaging test. Although the trend for the severe condition here were not completely consistent with the observations made for CBEMs prepared at 32^{0}C, 200 gyrations, overall again, the VACBEM indicated the least good performance for both the severe and mild conditions (recovery path) with the 15dmmCBEM still having the best performance.

For the recovery process in Protocol 2, the VACBEM achieved a significant improvement for materials prepared at 32^{0}C, 100 gyrations just as the RAP CBEMs. The recovery path for other conditions was generally poor for the VACBEM while it was rather impressive for the RAP CBEMs. The poor performance of the VACBEM could have been partly as a result of its high air void content, while the lower air voids of the RAP CBEMs could have been partly responsible for their better performance. More importantly, it is believed that the residual bitumen in the severely aged RAPs that constitute the RAP CBEMs could possibly have been rejuvenated thus providing a larger volume of active binder. This must have been responsible for the impressive performance observed for these RAP CBEMs in the recovery process and generally in being able to reasonably contain the water damaging effects when compared to the VACBEM. Comparing the results obtained here with the results of the temperature susceptibility tests, it is however likely that those CBEM materials prepared at 20^{0}C might not have been able to contain the water damaging effects as those prepared at 32^{0}C did. While this is an interesting area for future investigations, the two water susceptibility protocols as conducted here have been able to rank the CBEMs compared to the temperature susceptibility tests.

6.8.4 ITSM and air voids

A lot has been done by researchers to ascertain the effect of air void content on the stiffness properties of HMA. Tayebali et al, (1994) in their work observed that with an increase in air void content, the stiffness of HMA is significantly affected. For this reason, it is always required in the industry that air void contents of HMA should be kept low as much as possible though not below 3% to prevent early permanent deformation. However, there is a lack of evidence in literature to confirm whether such a relationship exists for CBEMs. It is believed that a better understanding of this effect for CBEMs will aid better design and use of the material. Therefore an investigation was conducted to ascertain such effects on CBEMs. For this exercise, the five CBEMs previously investigated, conditioned to early life, intermediate life and fully cured were studied. Figure 6.29 shows a general scatter plot of air voids and ITSM measured at 20^0C for all the CBEMs.

Figure 6.30 is a similar scatter plot now coloured to reflect the effect of mixing and compaction temperatures. The red dash line ellipse is for the CBEMs conditioned for early life. As mentioned in the preceding sections, all the CBEMs for early life were produced at 32^0C. The location of CBEMs prepared at 20^0C suggests that materials prepared at 20^0C have high air void contents with correspondingly low ITSM values though the level of significance would have to be ascertained. Figure 6.31 is a similar scatter plot but now coloured to reflect the effect of compactive effort applied on air voids and the corresponding ITSM responses for the CBEMs. Again the red dash line ellipse confirms that the trend enclosed is for the early life CBEMs as black spots which represent compactive effort of 100 gyrations were rarely seen therein. Early life specimens were prepared at 32, 200 gyrations. The scatter plot here, very much unlike the preceding figure, shows a well spread out scatter just as observed for the red spots. This suggests that 100 gyrations compactive effort is not significantly different in effect from 200 gyrations compactive effort, though this needs to be properly ascertained using statistical tools.

Similarly, Figure 6.32 is the same scatter plot but now coloured to show the effect of curing condition i.e. early life, intermediate life and fully cured conditions on the air voids and the corresponding ITSM responses for the CBEMs. The figure confirms the identified trends for early life in the preceding figures. The figure demonstrates that there is a correlation between air void contents and ITSM but

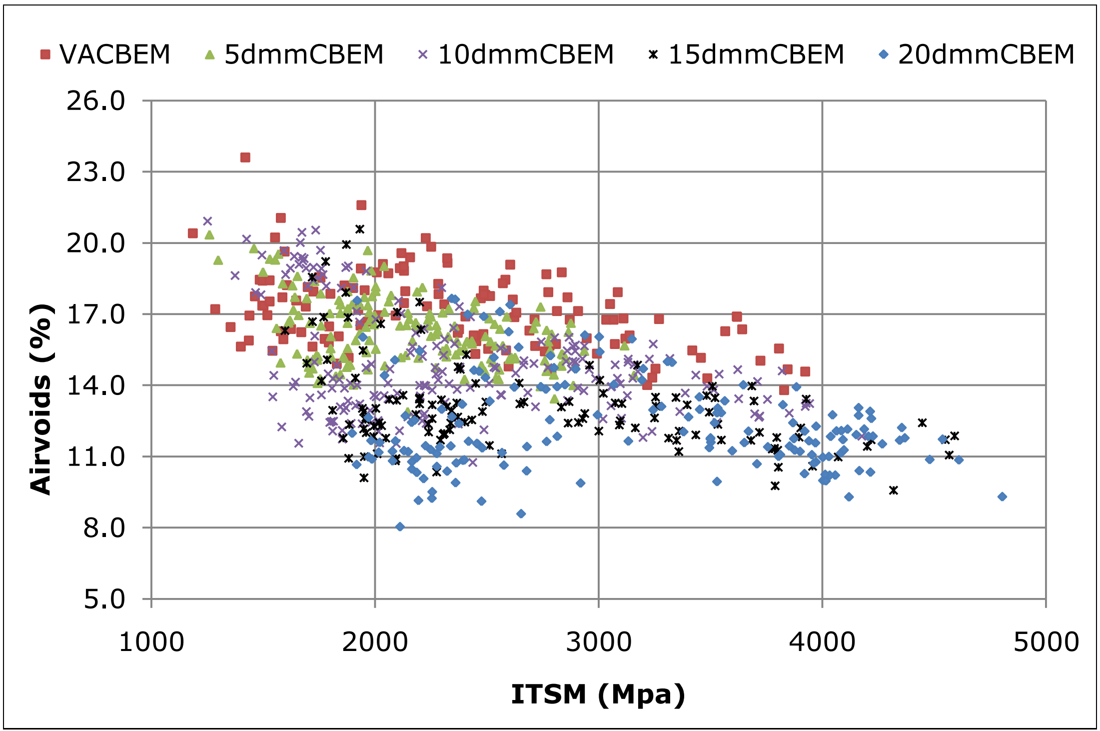


Figure 6.29: Scatter Plot for Air Voids and ITSM @ 20^{0}C of the CBEMs

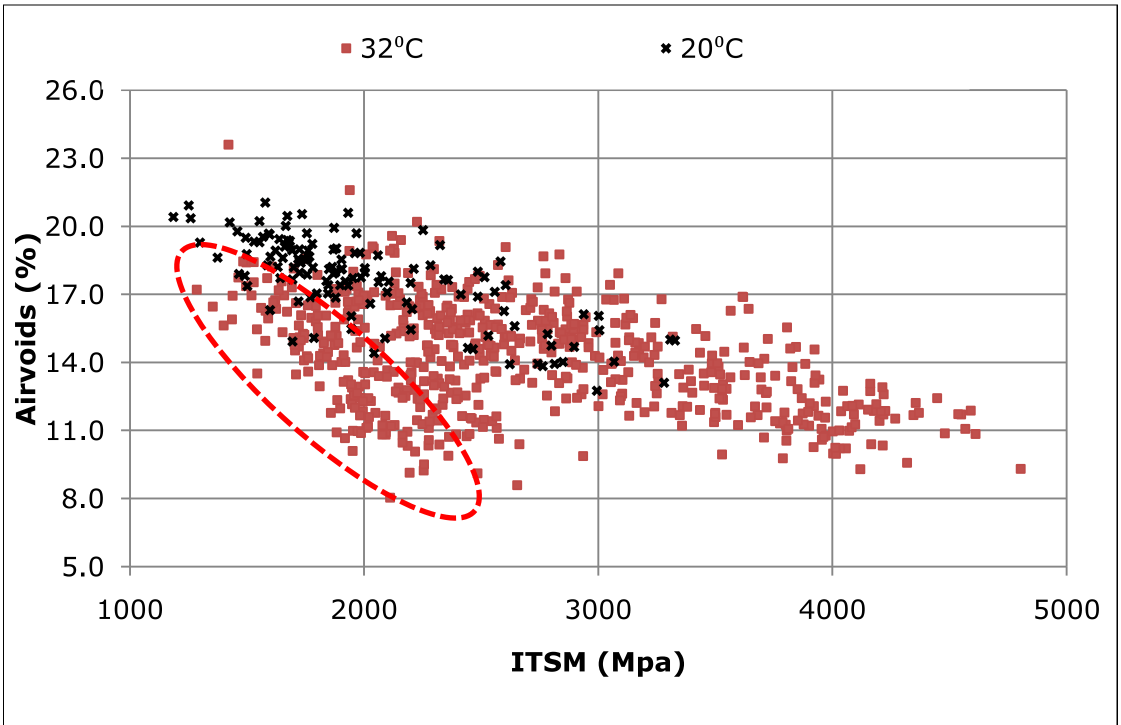


Figure 6.30: Scatter Plot for the CBEMs showing the effect of Mixing and Compaction temperatures on Air Voids and ITSM @ 20^{0}C

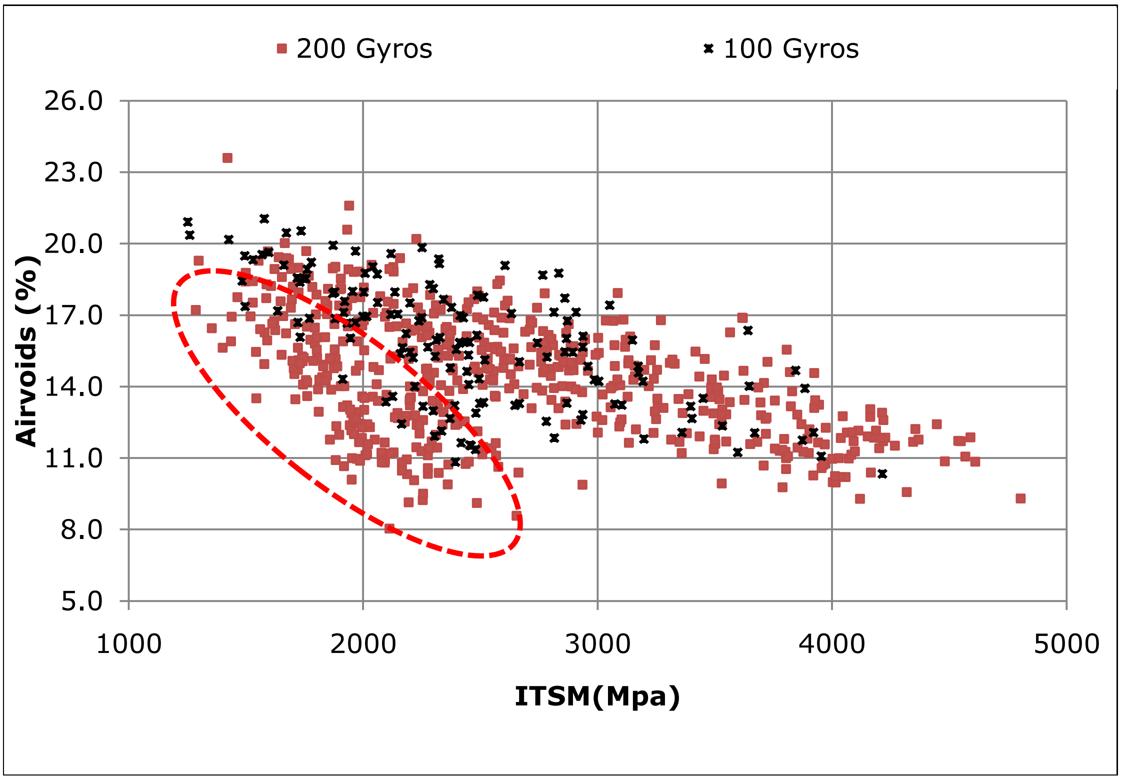


Figure 6.31: Scatter Plot for the CBEMs showing the effect of Compactive Effort on Air Voids and ITSM @ 20^{0}C

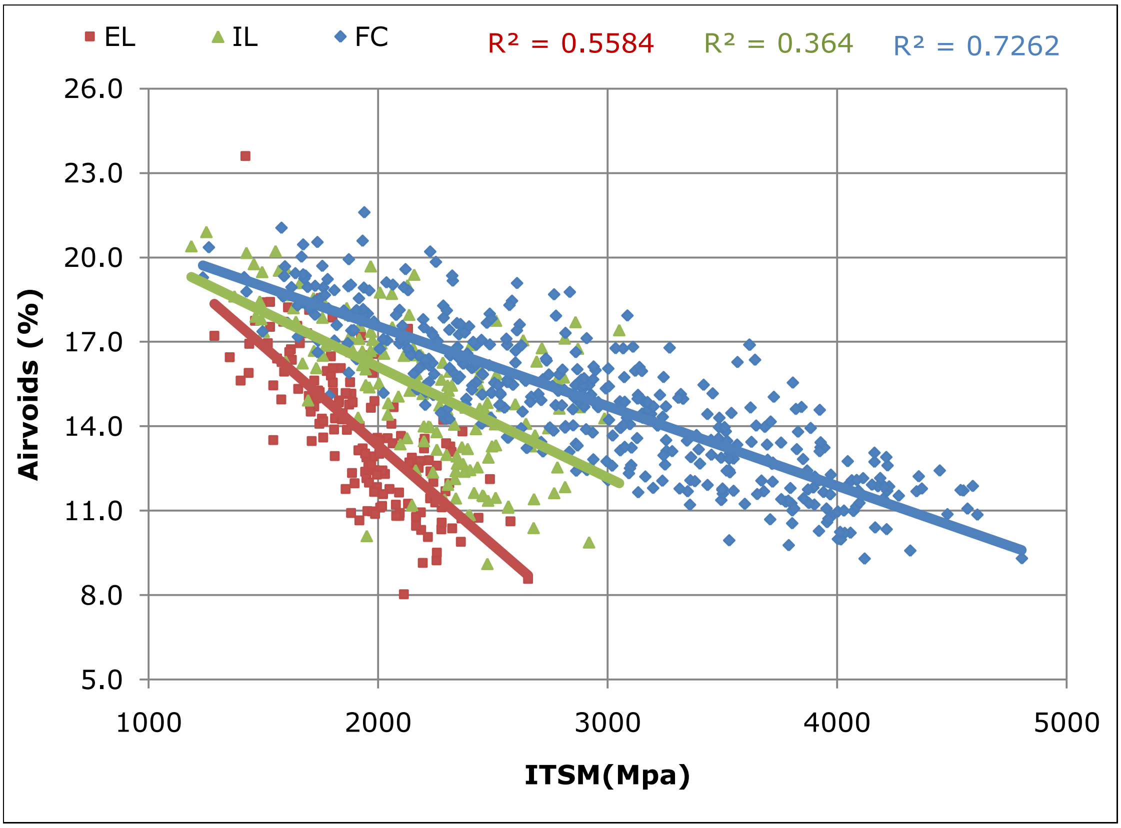


Figure 6.32: Scatter Plot for the CBEMs showing the effect of Curing Conditions on Air Voids and ITSM @ 20^{0}C

strongly influenced by curing condition. The trend in the early life is steeper (negative linear relationship) than that for the intermediate life and fully cured (F.C.) conditions. This steepness in trend due to high moisture contents obviously makes the CBEMs at early life less stiff compared to the fully cured CBEMs. At early life the moisture contents for the CBEMs were generally above 1.0% compared to the fully cured conditions where moisture contents were as low as 0.2% as detailed in Table 6.5.

Figure 6.33 shows the scatter plot of CBEMs produced and conditioned to early life while Figure 6.34 shows the scatter plot of CBEMs produced using similar procedures to those of early life but now fully cured. These two figures demonstrate that there is a correlation between air void content and ITSM but strongly influenced by curing condition. The trend in the early life is steeper than for the intermediate life condition despite having similar ranges of air void contents. These observations therefore call for caution in directly applying the air void – stiffness relationship for HMA to CBEMs.

In order to have a fuller understanding of the air voids – stiffness relationship of CBEMs, a multiple regression analysis as explained in section 6.6.2 of this chapter using STATA was conducted to ascertain the effect of curing state and CBEM material type on such relationship, and if it really exists. While the case of CBEMs produced to simulate early life was straightforward as no variables were involved and normal regression analysis could therefore be used, the cases for CBEMs conditioned to simulate intermediate life (I.L.) and fully cured (F.C.) were different.

So as to have a fairly high sample number for each case and also for brevity, all specimens for these two curing conditions were grouped irrespective of the procedure followed for production of the CBEMs. For each of these respective groups i.e. I.L. and F.C. created for each of the five CBEMs, the CBEMs produced at 20^0C, 100 gyrations were used as the reference variable set in the regression model. Those produced using 20^0C and 200 gyrations, 32^0C and 100 gyrations, 32^0C and 200 gyrations in the mentioned order were taken as the other variables in that order in the regression model. This arrangement considers that these other CBEMs were treated differently compared to the reference variable set. For each of the five CBEMs, 32 specimens were studied for each group of E.L. and I.L. data, while 64, 100, 100, 64 and 100 were studied for the VACBEM, 5dmmCBEM, 10dmmCBEM, 15dmmCBEM and 20dmmCBEM respectively for F.C. Table 6.20 details the results of regression analysis of CBEMs for ITSM versus air voids. Included in the table are values of coefficient, standard error, confidence intervals

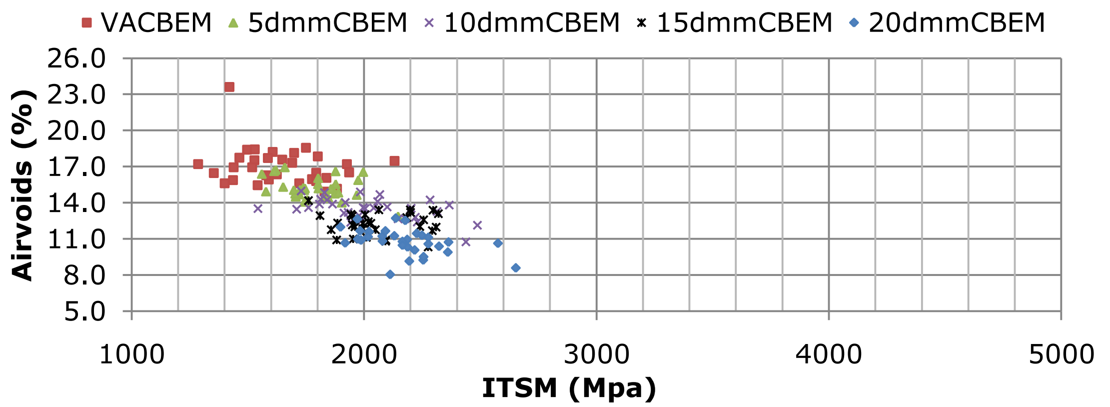


Figure 6.33: Scatter Plot and Trend line Diagram for Air voids and ITSM @ 20^{0}C of CBEMs at Early Life (32^{0}C, 200 gyrations)

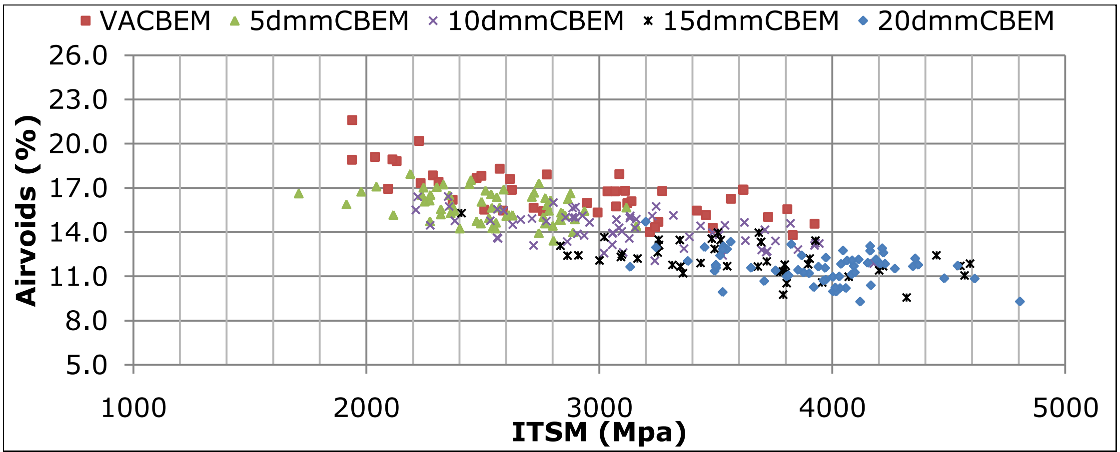


Figure 6.34: Scatter Plot for Air voids and ITSM @ 20^{0}C of CBEMs at Fully Cured Condition (32^{0}C, 200 gyrations)

Table 6.20: Results of Regression Analysis of CBEMs for ITSM versus Air voids

Material Type	CBEM Condition	Coef.	Std Error	t	P>|t|	[95% Conf. Interval]	
VA CBEM	E.L.	-27.746	21.9444	-1.26	0.216	-72.56	17.07
	I.L.	-144.18	58.0065	-2.49	0.019	-263.20	-25.15
	F.C.	-239.49	34.5107	-6.94	0.000	-308.57	-170.40
5dmm CBEM	E.L.	-52.934	24.2781	-2.18	0.037	-102.52	-3.36
	I.L.	-114.81	42.8568	-2.68	0.012	-202.74	-26.87
	F.C.	-134.89	25.4491	-5.30	0.000	-185.41	-84.37
10dmm CBEM	E.L.	-81.312	34.0495	-2.39	0.023	-150.85	-11.77
	I.L.	-168.78	48.8291	-3.46	0.002	-268.97	-68.59
	F.C.	-227.21	30.7590	-7.39	0.000	-288.27	-166.14
15dmm CBEM	E.L.	-10.598	31.4853	-0.34	0.739	-74.90	53.70
	I.L.	2.54559	40.2057	0.06	0.950	-79.95	85.04
	F.C.	-171.64	43.1802	-3.97	0.000	-258.04	-85.23
20dmm CBEM	E.L.	-74.794	25.3845	-2.95	0.006	-126.64	-22.95
	I.L.	-77.638	40.9428	-1.90	0.069	-161.65	6.37
	F.C.	-124.78	24.9833	-4.99	0.000	-174.38	-75.18

E.L. = Early Life *I.L. = Intermediate life* *F.C. = Fully Cured* and results of the significance tests. The values in the coefficient (abbreviated as Coef.) column in the table imply that for each 1% rise in air void content, there is such effect on stiffness modulus. A close inspection of the table shows that all the values except one (15dmmCBEM I.L.) are negative. In most cases, values reduce from E.L. to I.L., I.L. to F.C. This implies that as CBEMs mature (cure) the effect of air voids becomes more relevant. Meanwhile, for the observations to be taken as significant and thus confirm a strong relationship, the values in column P>|t| must be less than 0.05. These results have been coded thus: green highlights in Table 6.20 identify those with weak relationships i.e. P>|t| is greater than 0.05, while the yellow highlights are those below 0.05. These results are represented graphically in Figures 6.35 to 6.37 with 95% confidence intervals. The red bars represent the result of the significance tests.

From these results it is observed that a strong relationship exists between air voids and stiffness for cases 5dmmCBEM and 10dmmCBEM at E.L., I.L., and F.C. The VACBEM and 20dmmCBEM were next strongest. However, the F.C. condition for all the CBEMs indicates the strongest relationship as shown in Figure 6.37. The significance tests for this case returned values of 0 irrespective of CBEM type which implies very strong relationship. The 15dmmCBEM returned the poorest results for the significance tests as indicated in Figures 6.35 and 6.36 for the multiple regression analysis. However in a normal regression analysis which was executed for the 15dmmCBEM data alone to further examine the air void contents and ITSM relationship, the results retuned 0, -107.7298, and 23.14634 for the level of significance, coefficient and standards error respectively for the I.L. condition. The significance observed here is very strong. Similar results were obtained for the 20dmmCBEM. The reason for these conflicting results must have been due to the sample size which is relatively small. Such analyses require robust sample number for good results. None-the-less, the results here indicate generally that as CBEMs cure, the air voids – stiffness relationship gets stronger. From this investigation, this is particularly observed as strong for VACBEM as a 1% rise in air void content could result in loss of ITSM as high as 239.49MPa for fully cured (F.C.) conditions.

6.9 The Indirect Tensile Fatigue Test (ITFT)

As earlier mentioned, fatigue is one of the failure mechanisms of bituminous road pavements and so the fatigue behaviour of bituminous materials is normally characterised to ensure proper design. The Indirect Tensile Fatigue Test (ITFT) is useful in this regard because it is easy and as well fast to conduct compared to

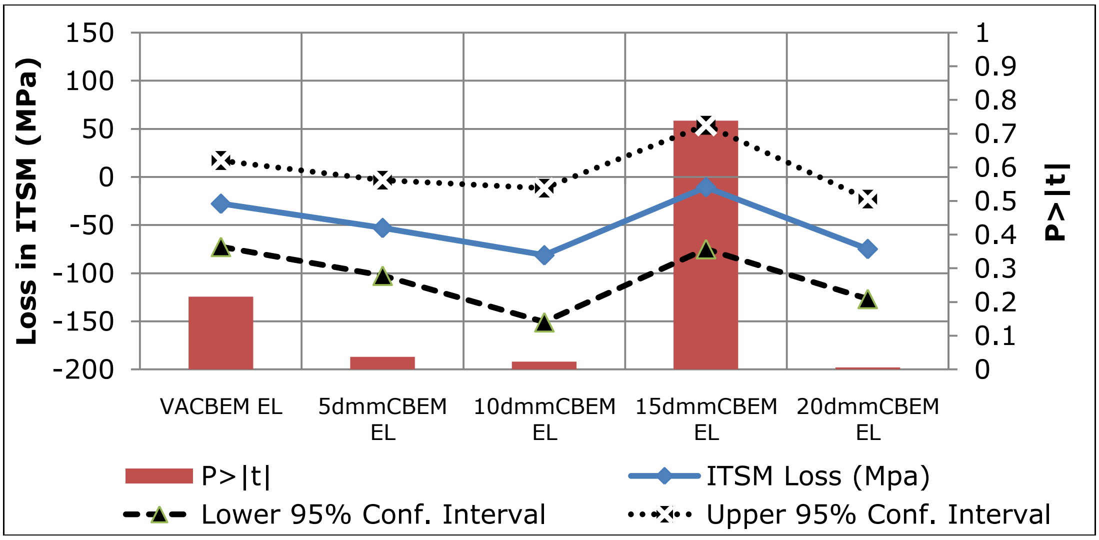


Figure 6.35: Loss in ITSM at Early Life Condition due to a 1% Rise in Air Void Content

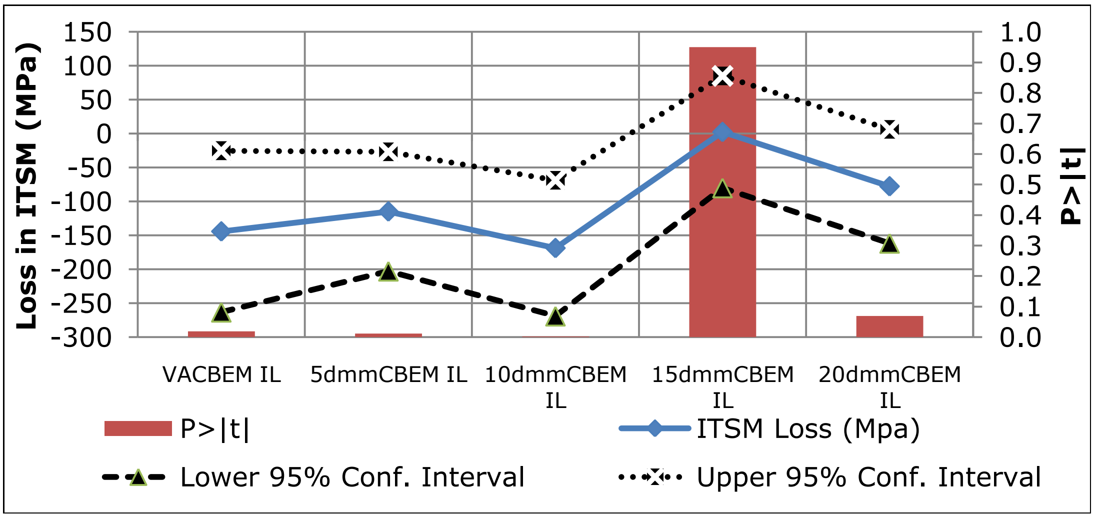


Figure 6.36: Loss in ITSM at Intermediate Life Condition due to a 1% Rise in Air Void Content

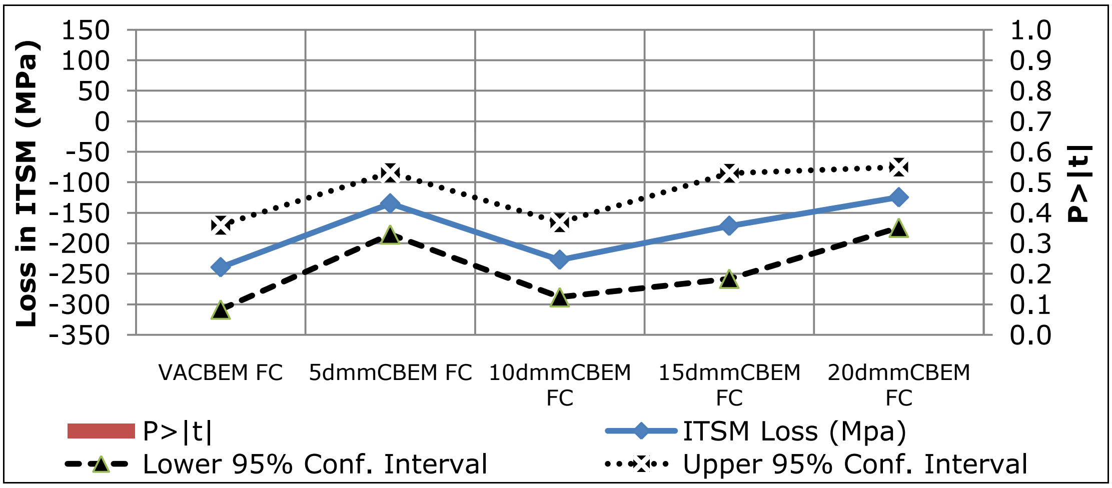


Figure 6.37: Loss in ITSM at Fully Cured Condition due to a 1% Rise in Air Void Content

other fatigue tests i.e. beam fatigue tests. This is mainly because cylindrical specimens either manufactured in the laboratory or cored from the road are also used for the test just as for the ITSM. Plate B in Figure 6.9 shows the ITFT mode of testing in the NAT. As in the ITSM test, every other assumption for specimen and condition of testing holds for the ITFT as outlined by Read (1996), except that load is applied repeatedly along the vertical diameter until the specimen fails i.e. when the loading strip vertically deforms by 9mm. This load induces an indirect tensile stress on the horizontal diameter. Rahman (2004) reported that the magnitude of the stresses varies along the diameter but that they are at a maximum at the centre of the specimen. Thus prior to the ITFT test, an ITSM test in stress controlled mode is conducted on specimens at the required test stress level for the ITFT to determine the horizontal deformation and corresponding ITSM values. Once the test has been done, equations 6.1 and 6.12 are used to determine the maximum stress and maximum strain at the centre of the specimen.

The CBEM specimens used were 50±2mm thick. Tests were conducted at 20^{0}C and 30^{0}C. Trimming was not considered appropriate for such fragile mixtures in order not to compromise the integrity of the specimens. Also for very weak specimen conditions, especially for tests conducted at 30^{0}C, stress levels as low as 300kPa were chosen as the maximum. For the analysis of results, the initial maximum tensile strain at the centre of the specimen ($\varepsilon_{hx(max)}$) for each respective stress level as determined for each specimen using equation 6.12 was subsequently plotted against corresponding number of cycles to failure (N_f). Linear regression analysis using the least square method was then applied to the data. It is common for researchers to fit the curves using the relationship below suggested by Pell (1973):

$$N_f = c \, X \, (\frac{1}{\varepsilon_{hx(\max)}})^m \qquad 6.16$$

Where:

N_f = Number of load applications to failure

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

c, m = factors depending on the composition and properties of the mixture; m is the slope of the fatigue line.

6.9.1 Fatigue responses of CBEMs

For this test, only CBEMs that were fully cured, mixed and compacted at 20^{0}C and 32^{0}C applying compactive effort of 200 gyrations were examined. 15dmmCBEMs were not included in this study. Also the VACBEM prepared at 20^{0}C was not included. Table 6.21 details the fatigue responses of the CBEMs using the equations for strain and cycles to failure and the R-squared values. Equations for 20mmDBM and 28mmDBM 50 studied by Read (1996) were included for comparison though tested at lower temperatures. Meanwhile the R-squared values in the table show a good fitness of the data for the relationship stated in equation 6.16 since all the values were above 0.90. For brevity, only figures in which VACBEM are represented have been plotted here since it is the control mixture. Figures 6.38 and 6.39 show the regressed fatigue lines for the tests conducted at 20^{0}C and 30^{0}C respectively. Figure 6.38 which is for tests conducted at 20^{0}C demonstrates that the 20dmmCBEM has the longest fatigue life of all the CBEMs studied, while the 5dmmCBEM indicated the shortest. This is against logic at face value as researchers have suggested that the material with the stiffest bitumen gives the longest fatigue life (Cooper and Pell, 1974). The explanation for this observation is that the residual bitumen from the emulsion played the prominent role in the fatigue response of the 5dmmCBEM. The activeness of aged residual bitumen was not significant enough to alter the property of the overall active binder in the mix.

Aside from the VACBEM which ordinarily was expected to have displayed the least fatigue life response, the next in rank to the 5dmmCBEM is the 10dmmCBEM. It is likely that a relatively higher portion of the residual aged bitumen from the 10dmm RAP has possibly been rejuvenated compared to the 5dmmCBEM. Therefore, it is logical for the 20dmmCBEM to have demonstrated the longest fatigue life since it has the greatest possible potential for rejuvenation (if any rejuvenation is taking place) as a result of it containing the most active and stiffest bitumen at the temperature of 30^{0}C used for the preparation of the CBEMs. The preparation temperature was probably sufficient to excite a significant portion of the residual aged bitumen in the 20dmm RAP causing it to interact with the residual binder from the emulsion and in the process altering the overall behaviour of the active binder in the mix i.e. becoming stiffer. This order could have changed if the preparation temperature was sufficiently high to excite all the residual aged binders in the 5dmm and 10dmm RAPs. Figure 6.39 for the tests conducted at 30^{0}C shows that the 5dmmCBEM displayed the longest fatigue life while the VACBEM indicated the shortest of all the CBEMs tested. This observation appears logical except for the order of the 10dmmCBEM and

Table 6.21: Fatigue Responses of CBEMs at Fully Cured Condition

Mixture Type	Equation for Strain	Equation for Cycles to Failure	R^2
VA (Mixed and Compacted @ 32⁰C) Tested@ 20⁰C	$1906.1N_f^{-0.338}$	$1.67 \times 10^9\varepsilon^{-2.776}$	0.94
VA(Mixed and Compacted @ 32⁰C) Tested@ 30⁰C	$69995N_f^{-0.916}$	$99856\varepsilon^{-0.989}$	0.91
5dmm (Mixed and Compacted @ 20⁰C) Tested@ 20⁰C	$5862.8N_f^{-0.45}$	$9.13\text{x}10^8\varepsilon^{-2.082}$	0.94
5dmm (Mixed and Compacted @ 32⁰C) Tested@ 20⁰C	$5243.2N_f^{-0.449}$	$1.13\text{x}10^8\varepsilon^{-2.141}$	0.96
5dmm (Mixed and Compacted @ 20⁰C) Tested@ 30⁰C	$6369.9N_f^{-0.453}$	$1.24\text{x}10^8\varepsilon^{-2.101}$	0.95
5dmm (Mixed and Compacted @ 32⁰C) Tested@ 30⁰C	$3217.1N_f^{-0.328}$	$1.7\text{x}10^{10}\varepsilon^{-2.878}$	0.94
10dmm (Mixed and Compacted @ 20⁰C) Tested@ 20⁰C	$9431.8N_f^{-0.559}$	$8.04\text{x}10^6\varepsilon^{-1.719}$	0.96
10dmm (Mixed and Compacted @ 32⁰C) Tested@ 20⁰C	$3296.3N_f^{-0.367}$	$1.77\text{x}10^9\varepsilon^{-2.585}$	0.95
10dmm (Mixed and Compacted @ 20⁰C) Tested@ 30⁰C	$9509.4N_f^{-0.51}$	$4.3\text{x}10^7\varepsilon^{-1.9}$	0.97
10dmm (Mixed and Compacted @ 32⁰C) Tested@ 30⁰C	$7434N_f^{-0.518}$	$1.27\text{x}10^7\varepsilon^{-1.799}$	0.93
20dmm (Mixed and Compacted @ 20⁰C) Tested@ 20⁰C	$2623.1N_f^{-0.377}$	$6.93\text{x}10^8\varepsilon^{-2.557}$	0.96
20dmm (Mixed and Compacted @ 32⁰C) Tested@ 20⁰C	$1942N_f^{-0.32}$	$3.79\text{x}10^9\varepsilon^{-2.803}$	0.90
20dmm (Mixed and Compacted @ 20⁰C) Tested@ 30⁰C	$7581.3N_f^{-0.451}$	$1.24\text{x}10^8\varepsilon^{-2.046}$	0.92
20dmm (Mixed and Compacted @ 32⁰C) Tested@ 30⁰C	$7182.5N_f^{-0.465}$	$1.16\text{x}10^8\varepsilon^{-2.068}$	0.96
20mmDBM (Read,1996)	$1562N_f^{-0.246}$	$9.6\text{x}10^{12}\varepsilon^{-4.065}$	
28mmDBM 50 (Read,1996)	$2595N_f^{-0.255}$	$2.45\text{x}10^{13}\varepsilon^{-3.922}$	

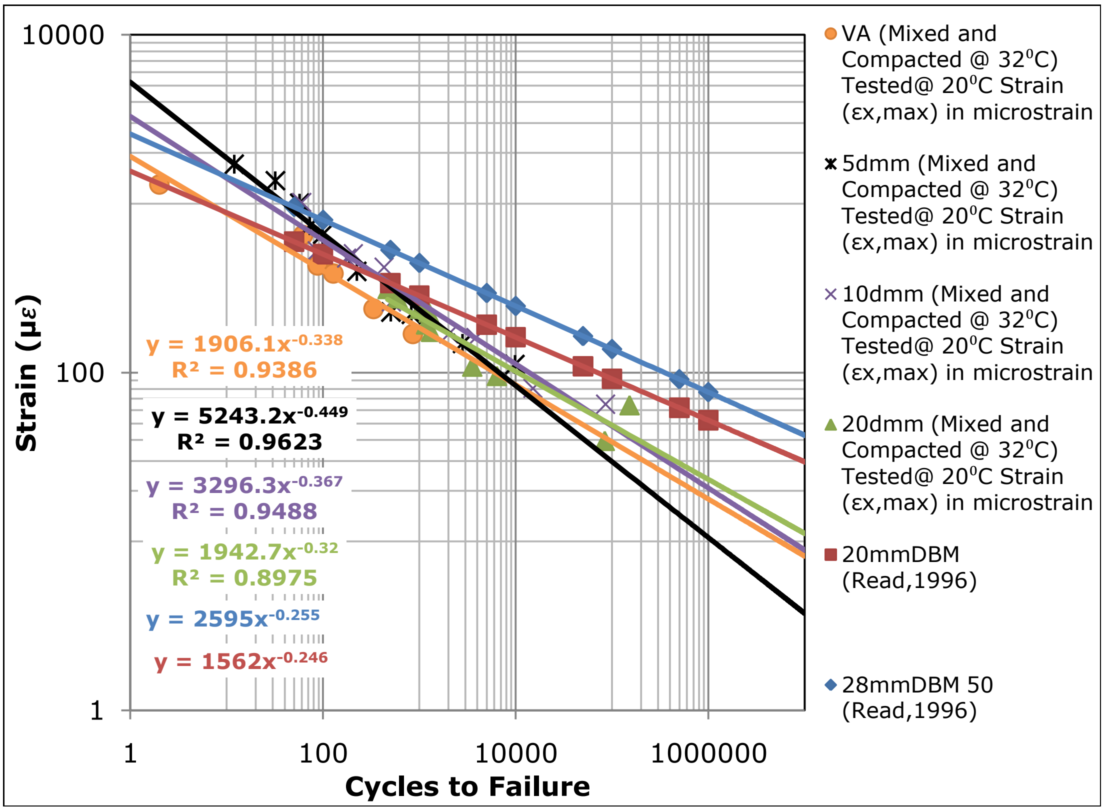


Figure 6.38: Fatigue Responses of CBEMs at Fully Cured Condition Mixed and Compacted @ 32°C, Tested @ 20°C

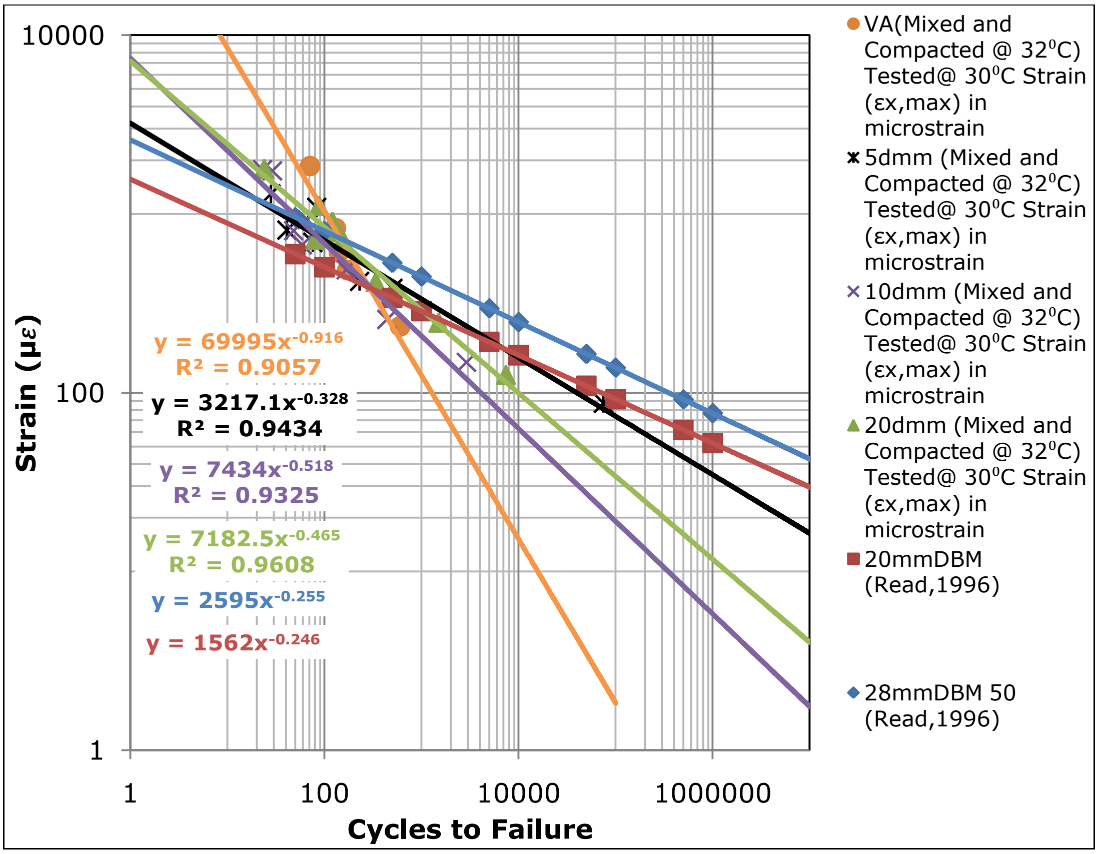


Figure 6.39: Fatigue Responses of CBEMs at Fully Cured Condition Mixed and Compacted @ 32°C, Tested @ 30°C

20dmmCBEM at 20^{0}C and the earlier explanations given. At the test temperature of 30^{0}C, the residual aged bitumen in the 5dmmCBEM is probably now excited or active thus significantly enhancing its fatigue life. These CBEMs were normally conditioned to test temperature before testing. Thus this CBEM should have the overall stiffest active binding medium compared to the other CBEMs. The fatigue life of the VACBEM is short mainly because it has the softest and probably the lowest volume of active binding medium. The figure also indicated that the VACBEM was more stress sensitive compared to the other CBEMs at this test temperature.

However, the fatigue lives of the CBEMs generally were lower than those for the hot mixtures as observed in the said figures. This confirms the general belief that HMA have superior mechanical properties than cold mixtures (Biczysko, 1996; and Carswell, 2004).

In order to have a fuller knowledge of what the equations presented earlier in Table 6.21 signify, the equation for cycles to failure were used to calculate fatigue lives consequent upon reaching strains of 30με, 50με, 100με and 200με, and similarly the equivalent strains using the equations for strain for 1.0E+03, 1.0E+04, 1.0E+05 and 1.0E+06 cycles respectively. Read (1996) suggested that fatigue failure in bituminous mixtures normally occurs in the range 30-200 με. Table 6.22 details fatigue lives of CBEMs at the mentioned strains while Table 6.23 details the corresponding strains for the suggested fatigue lives.

The analysis is restricted to those comparable to the VACBEM (prepared at 32^{0}C), thus only similar colours in the tables are comparable. At 50με, 100με and 200με, the 10dmmCBEM recorded the longest fatigue life while the 5dmmCBEM had the least for 50με. The VACBEM returned the least performance for the 100με and the 200με. Overall, the result at this level indicates that the 10dmmCBEM has the best fatigue life.

For the tests conducted at 30^{0}C, the results in green in Table 6.22 are relevant for the cycles to failure. The 5dmmCBEM in all cases performed best followed by the 20dmmCBEM while the VACBEM had the least good performance. The earlier explanations given for the performances based on the regressed fatigue lines in Figures 6.38 and 6.39 are still relevant here. Therefore, there is an advantage for using RAPs in CBEMs when it comes to fatigue response over those made purely from virgin aggregates. As expected, the HMA in all cases performed better than the CBEMs though the 5dmmCBEM prepared at 32^{0}C and tested at 30^{0}C was close in performance to the 20mmDBM at 200με.

Table 6.22: Fatigue Life of CBEMs at Various Microstrains

CBEM Condition	Fatigue Life @ Various Microstrain			
	30(με)	50(με)	100(με)	200(με)
VA (Mixed and Compacted @ 32°C) Tested@ 20°C	132504	32090	4685	684
VA(Mixed and Compacted @ 32°C) Tested@ 30°C	3457	2086	1051	529
5dmm (Mixed and Compacted @ 20°C) Tested@ 20°C	76755	26498	6259	1478
5dmm (Mixed and Compacted @ 32°C) Tested@ 20°C	77725	26037	5903	1338
5dmm (Mixed and Compacted @ 20°C) Tested@ 30°C	97721	33411	7788	1815
5dmm (Mixed and Compacted @ 32°C) Tested@ 30°C	953441	219186	29816	4056
10dmm (Mixed and Compacted @ 20°C) Tested@ 20°C	23232	9654	2933	891
10dmm (Mixed and Compacted @ 32°C) Tested@ 20°C	268915	71802	11967	1994
10dmm (Mixed and Compacted @ 20°C) Tested@ 30°C	67133	25435	6815	1826
10dmm (Mixed and Compacted @ 32°C) Tested@ 30°C	27955	11152	3205	921
20dmm (Mixed and Compacted @ 20°C) Tested@ 20°C	115807	31367	5330	906
20dmm (Mixed and Compacted @ 32°C) Tested@ 20°C	274327	65528	9389	1345
20dmm (Mixed and Compacted @ 20°C) Tested@ 30°C	117823	41431	10033	2429
20dmm (Mixed and Compacted @ 32°C) Tested@ 30°C	102275	35562	8481	2023
20mmDBM (Read,1996)	9501088	1191127	71166	4252
28mmDBM 50 (Read,1996)	39302430	5297937	349274	23026

Table 6.23: Microstrain of CBEMs at Various Fatigue Lives

CBEM Condition	Microstrain @ Various Cycles to Failure (με)			
	1.0E+03	1.0E+04	1.0E+05	1.0E+06
VA (Mixed and Compacted @ 32°C) Tested@ 20°C	185	85	39	18
VA(Mixed and Compacted @ 32°C) Tested@ 30°C	125	15	2	0
5dmm (Mixed and Compacted @ 20°C) Tested@ 20°C	262	93	33	12
5dmm (Mixed and Compacted @ 32°C) Tested@ 20°C	236	84	30	11
5dmm (Mixed and Compacted @ 20°C) Tested@ 30°C	279	98	35	12
5dmm (Mixed and Compacted @ 32°C) Tested@ 30°C	334	157	74	35
10dmm (Mixed and Compacted @ 20°C) Tested@ 20°C	198	55	15	4
10dmm (Mixed and Compacted @ 32°C) Tested@ 20°C	261	112	48	21
10dmm (Mixed and Compacted @ 20°C) Tested@ 30°C	281	87	27	8
10dmm (Mixed and Compacted @ 32°C) Tested@ 30°C	208	63	19	6
20dmm (Mixed and Compacted @ 20°C) Tested@ 20°C	194	81	34	14
20dmm (Mixed and Compacted @ 32°C) Tested@ 20°C	213	102	49	23
20dmm (Mixed and Compacted @ 20°C) Tested@ 30°C	336	119	42	15
20dmm (Mixed and Compacted @ 32°C) Tested@ 30°C	289	99	34	12
20mmDBM (Read,1996)	286	162	92	52
28mmDBM 50 (Read,1996)	446	248	138	77

In Table 6.23 and for tests conducted at 20⁰C, the results for 1.0E + 03 cycles to failure indicated that the VACBEM had the least strain while the RAP CBEMs returned values generally greater than 200με. This trend was similarly observed for those tested at 30⁰C.

6.9.2 Fatigue responses and the effect of mixing and compaction temperatures of CBEMs

Based on the results of the studies carried out on the fatigue responses of the CBEMs, it was considered appropriate to ascertain the effect of preparation temperature on the fatigue responses. The analysis here was restricted to cycles to failure and thus Figures 6.40 and 6.41 were plotted using the results in Table 6.22.

Figure 6.40 represents CBEMs tested at 30⁰C. The black dashed ellipse captures the performance of the VACBEM which was prepared at 32⁰C. There is no information here about its counterpart prepared at 20⁰C. The 5dmmCBEM prepared at 20⁰C was inferior in performance to its counterpart prepared at 32⁰C. While those prepared at the two temperatures for the 20dmmCBEM showed similar performances, the 10dmmCBEM prepared at 20⁰C was better than its counterpart prepared at 32⁰C for all the strains considered.

The plots in Figure 6.41 are for the tests conducted at 20⁰C. The black dashed line ellipse encloses the results for the VACBEM prepared at 32⁰C. The performance here was average compared to the other CBEMs. At this test level, the effect of the preparation temperature was obvious in the performance of the 10dmmCBEM and 20dmmCBEM as those prepared at 32⁰C performed better than those at 20⁰C. No difference was however noticed for the 5dmmCBEM. The two graphs indicate that the VACBEM and 5dmmCBEM prepared at 32⁰C were significantly sensitive to the test temperature.

Overall, the RAP CBEMs prepared at a mixing and compaction temperature of 32⁰C performed better than those prepared at 20⁰C in fatigue response.

6.9.3 Fatigue responses and temperature susceptibility of CBEMs

The temperature susceptibility of the CBEMs under fatigue response was studied. For brevity, the discussion here is limited to CBEMs mixed and compacted at 32⁰C and the analysis was based on cycles required to reach the earlier stated target strains. The two test temperatures considered were 20⁰C and 30⁰C. For simplicity and easy reference the relevant data was extracted from Table 6.22 and

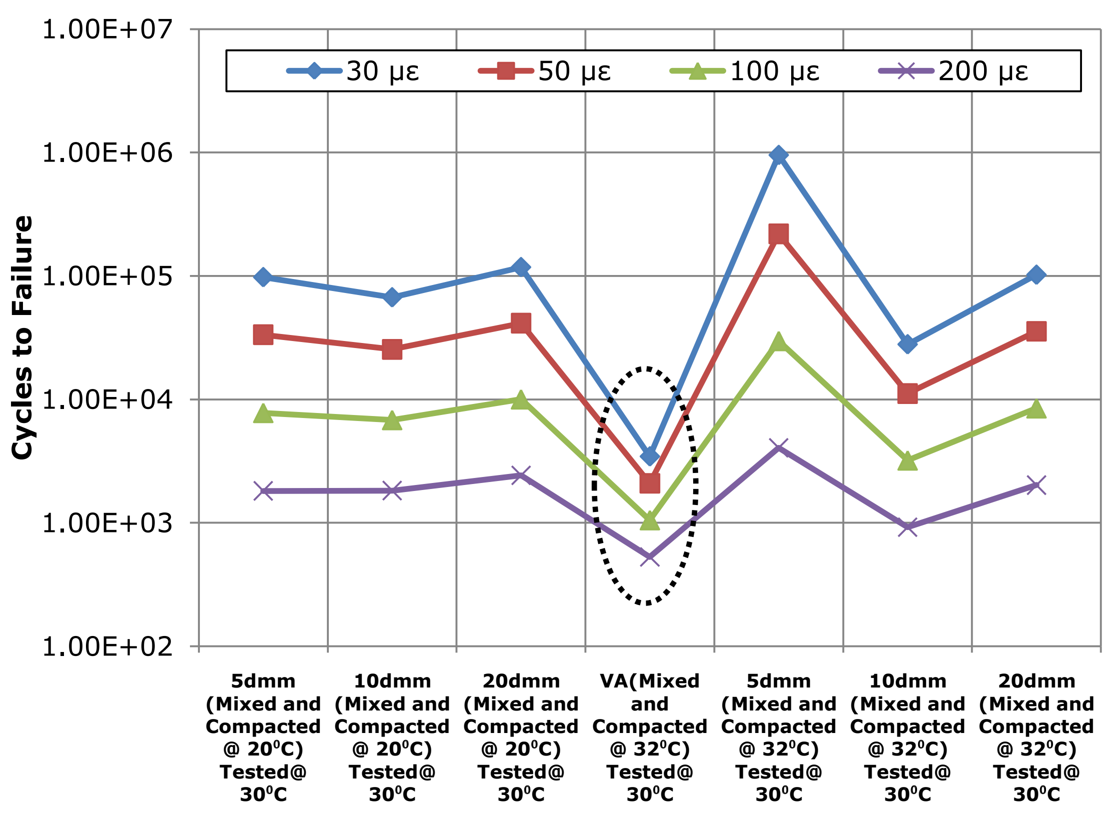


Figure 6.40: The Effect of Mixing and Compaction Temperature on Cycles to Failure of CBEMs Tested at 30°C

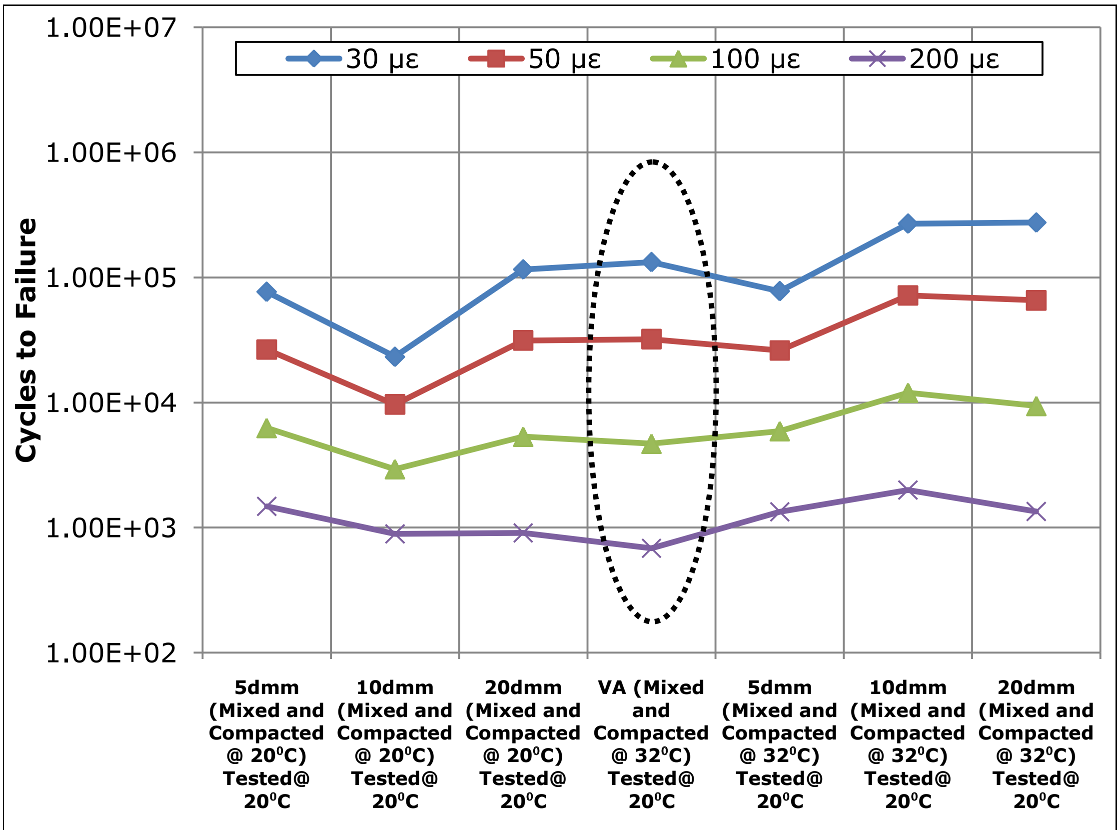


Figure 6.41: The Effect of Mixing and Compaction Temperature on Cycles to Failure of CBEMs Tested at 20°C

condensed to Table 6.24. Figures 6.42 and 6.43 are relevant here.

The plots in the figures indicate that increase in test temperature significantly enhanced the performance of the 5dmmCBEM, while it caused a significant reduction in performance for the VACBEM. These were followed by the 10dmmCBEM and 20dmmCBEM both having a reduction in response due to increase in temperature. The reasons for these observations have been discussed in the preceding sections. Thus for this study, the two most sensitive CBEMs to temperature changes were the VACBEM and 5dmmCBEM, though their responses were different. The behaviour of the 5dmmCBEM here seems to be consistent with its observed response for the ITSM test in section 6.8.2.

Also, considering the behaviour of both the VACBEM and 5dmmCBEM, it seems that fatigue responses of CBEMs were not affected by air void contents although it was earlier established that air voids have a strong correlation with stiffness of CBEMs but depending on the level of curing attained. Generally, these two CBEMs have relatively higher air void contents compared to the other CBEMs as observed in section 6.7.2, Table 6.6. Also they were both inferior in stiffness compared to the other CBEMs as observed in section 6.8.2 and Table 6.16. In HMA, stiffness and air void content alike have been noted to significantly affect fatigue responses (Nguyen, 2009). This was similarly noted for foamed asphalt mixtures (Sunarjono, 2008).

From all indications in this limited study, this might not be completely true for CBEMs. Since this is a limited study on CBEMs prepared at 32^{0}C, 200 gyrations, there is a need to investigate other CBEMs treated differently. Also, other methods of fatigue testing such as the trapezoidal test could be explored.

6.10 The Repeated Load Axial Test (RLAT) – Unconfined Mode of Testing

This test, also known as unconfined creep test is normally conducted to assess the permanent deformation properties of bituminous materials using BS DD 226 as the guide. The test is such that a specimen with similar dimensions to those for ITSM is subjected to repeated load axial pulses that simulate loading conditions in road pavements. Plate C in Figure 6.9 in section 6.8.1 shows the RLAT mode in the NAT. Read and Whiteoak (2003) gave the standard test conditions and requirements for the RLAT as:

Conditioning Stress: 10kPa
Conditioning Period: 600s

Table 6.24: Fatigue Life of CBEMs at Various Strain Levels

Test Temp. (^oC)	Target Strains	Fatigue Life			
		VA CBEM	5dmmCBEM	10dmmCBEM	20dmmCBEM
20	30 με	132504	77725	268915	274327
	50 με	32090	26037	71802	65528
	100 με	4685	5903	11967	9389
	200 με	684	1338	1994	1345
30	30 με	3457	953441	27955	102275
	50 με	2086	219186	11152	35562
	100 με	1051	29816	3205	8481
	200 με	529	4056	921	2023

All specimens mixed and compacted at @ 32℃, 200 gyrations

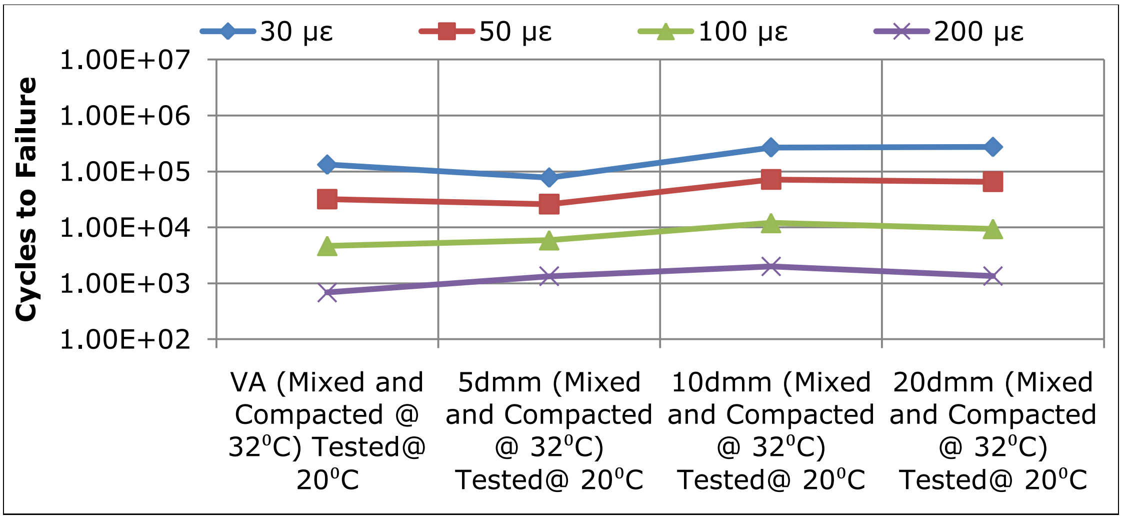


Figure 6.42: Temperature Susceptibility and Fatigue Responses of CBEMs at 20^{o}C

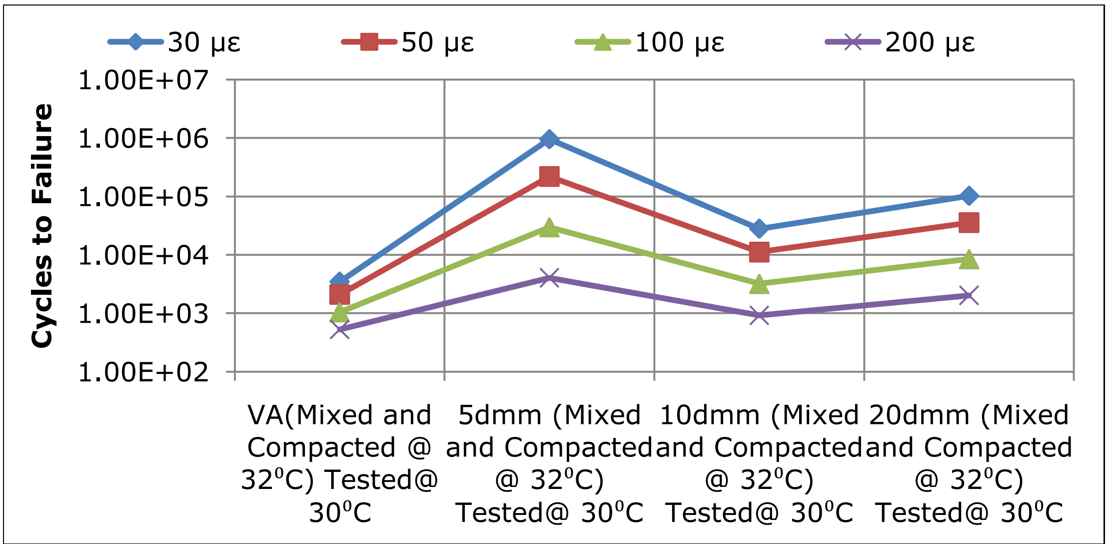


Figure 6.43: Temperature Susceptibility and Fatigue Responses of CBEMs at 30^{o}C

Test Stress:	100kPa
Test Duration:	1800 or 3600cycles
Test Cycle:	Square wave pulse 1s on, 1s off
Test Temperature:	30 or 40^{0}C

In the limited investigation conducted on CBEMs in this work using the RLAT, a test temperature of 40^{0}C and stress levels of 100kPa and 300kPa were used. 1800cycles was adopted for test duration considering the number of specimens and CBEM material types selected for the investigation. For each observation level, 3 specimens were tested and only the CBEMs prepared at 32^{0}C, 200 gyrations were investigated. The quoted strains are averages of the 3 specimens except otherwise stated. Although it is not common to test at 300kPa, it was intentionally done here in order to have a picture of what would have happened if the test was conducted at the standard stress level of 100kPa and for 3600cycles or more and this obviously proved useful since the final axial strain levels for all the studied CBEMs were clearly above 1% with one of the CBEMs getting as far as 5%. Only two effects were studied and it is worth noting that this test is only indicative.

6.10.1 Effect of RAP type on deformation properties

Figure 6.44 shows the plot of axial strain against load cycles for CBEMs prepared at 32^{0}C, 200 gyrations, conditioned to simulate early life and tested at 100kPa. Figure 6.45 is a similar plot but for fully cured specimens. Figures 6.46 and 6.47 respectively are similar to Figures 6.44 and 6.45 but tested at 300kPa. Figure 6.44 also included the results of a vacuum repeated load axial test (VRLAT) conducted on VACBEM at 100kPa (VA-ELV100). The results of the fuller investigation conducted on the VRALT are reported in the succeeding section.

A quick look at the figures shows that the VACBEM gave the best performance irrespective of the test stress level. With the exception of the performance of the 5dmmCBEM in Figure 6.46, all the RAP CBEMs performed fairly well though not in a distinct order for the tests conducted at 100kPa. However Figures 6.46 and 6.47 both seem to have ranked the CBEMs in a logical order excluding the unexpected response of the 5dmmCBEM in Figure 6.46. This result proved testing at 300kPa worthwhile for the short period of 1800cycles. Starting with the best performance, the ranking is in the order VACBEM, 5dmmCBEM, 10dmmCBEM, 15dmmCBEM and 20dmmCBEM respectively. The performance of the RAP CBEMs is not strange as they seem to contain more active binder contributing to the matrix (with the 20dmmCBEM having the greatest portion) than the VACBEM. In the case of the RAP CBEMs the binding medium played a significant role which is reflected in their

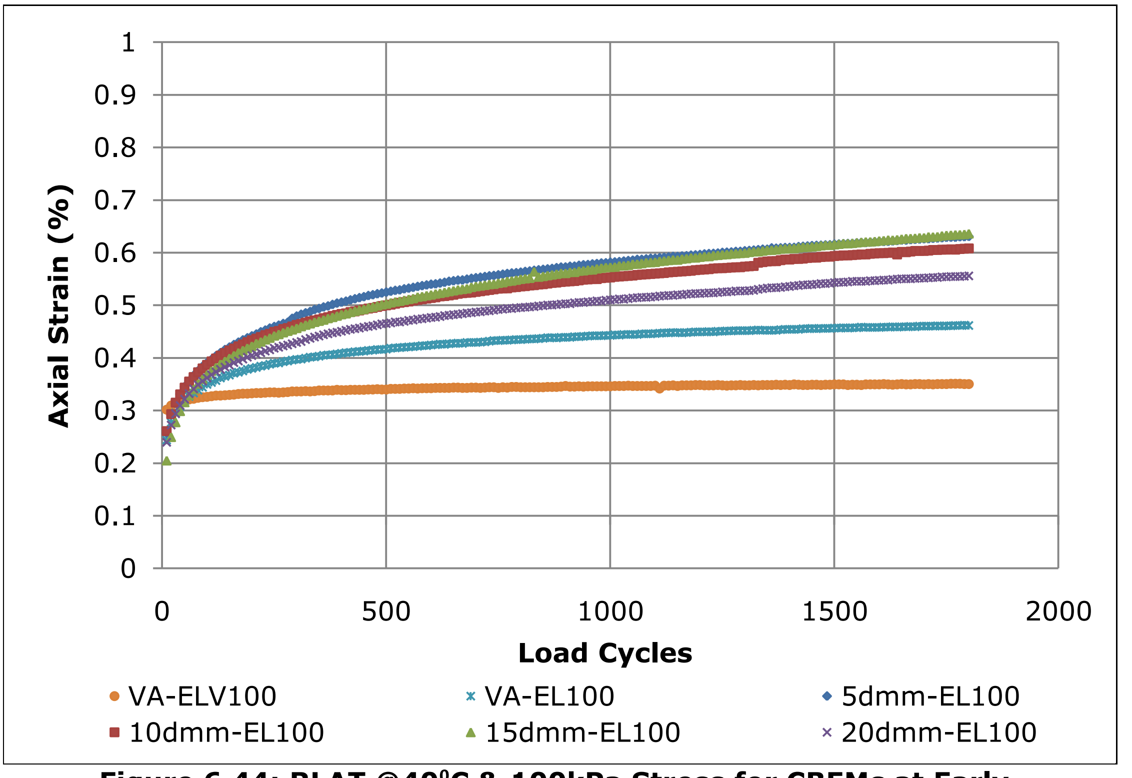


Figure 6.44: RLAT @40^{0}C & 100kPa Stress for CBEMs at Early Life Condition Mixed & Compacted @ 32^{0}C & 200 gyrations

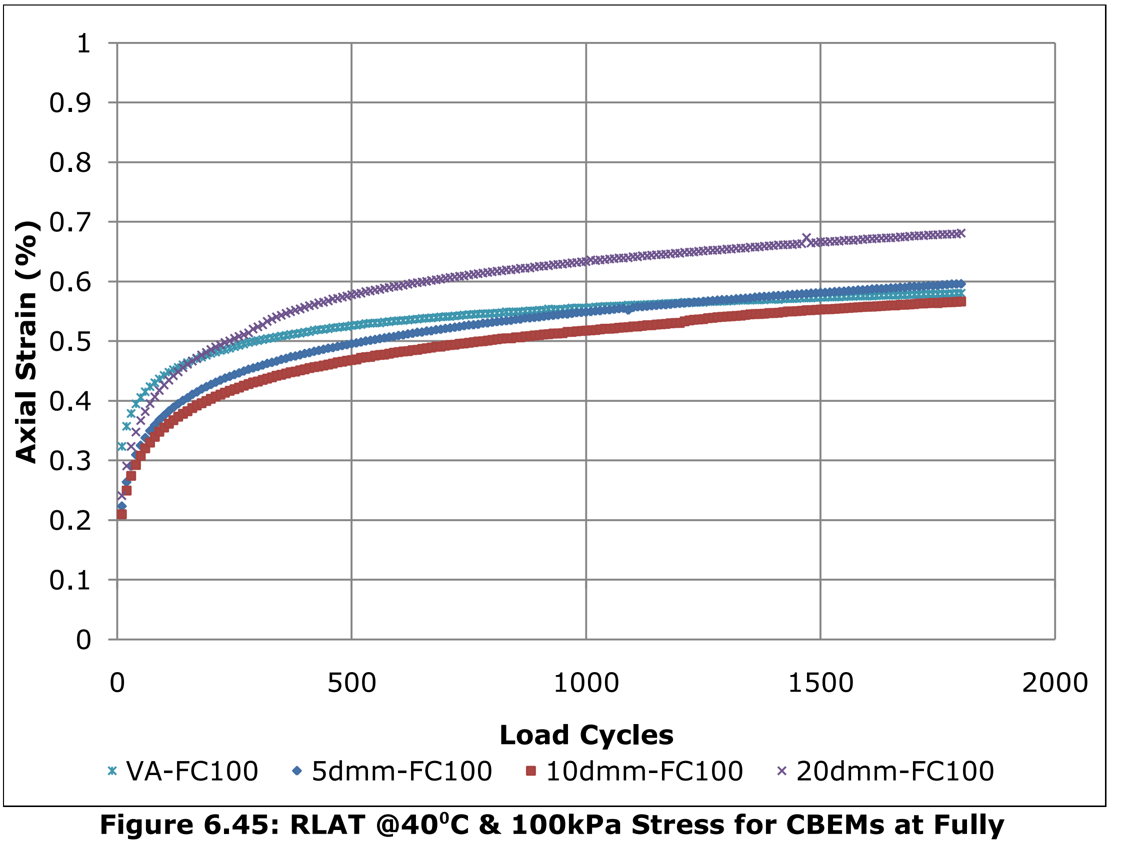


Figure 6.45: RLAT @40^{0}C & 100kPa Stress for CBEMs at Fully Cured Condition Mixed & Compacted @ 32^{0}C & 200 gyrations

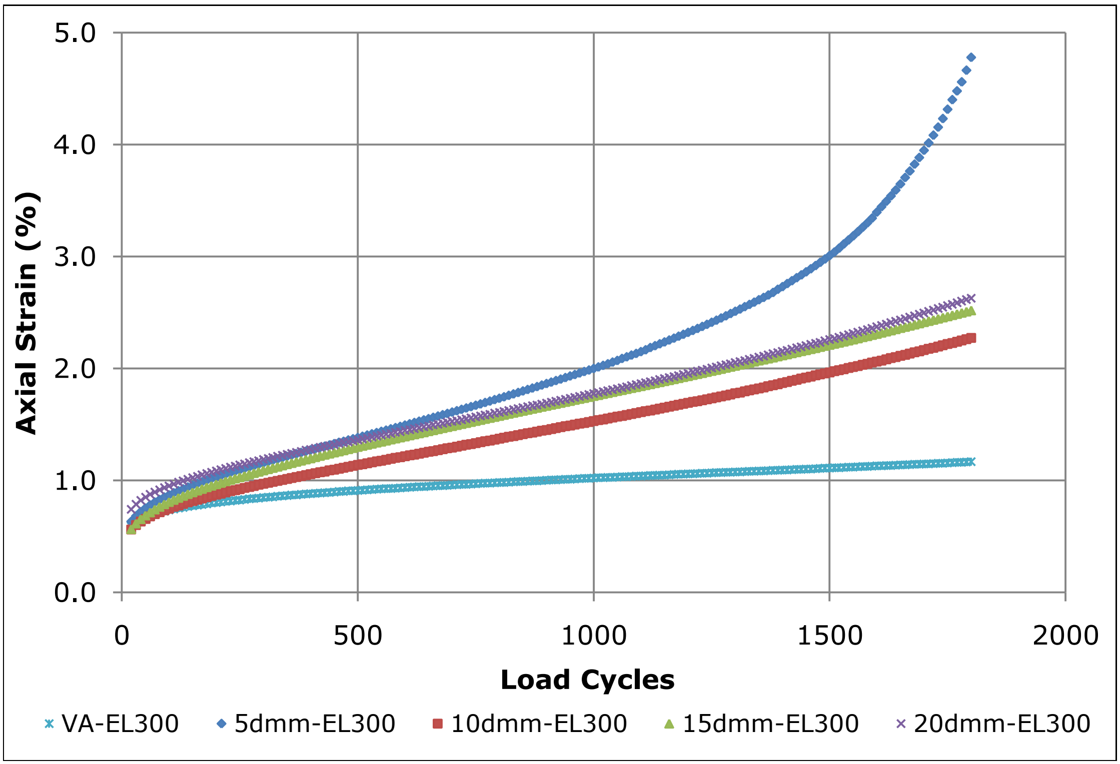


Figure 6.46: RLAT @40^{0}C & 300kPa Stress for CBEMs at Early Life Condition Mixed & Compacted @ 32^{0}C & 200 gyrations

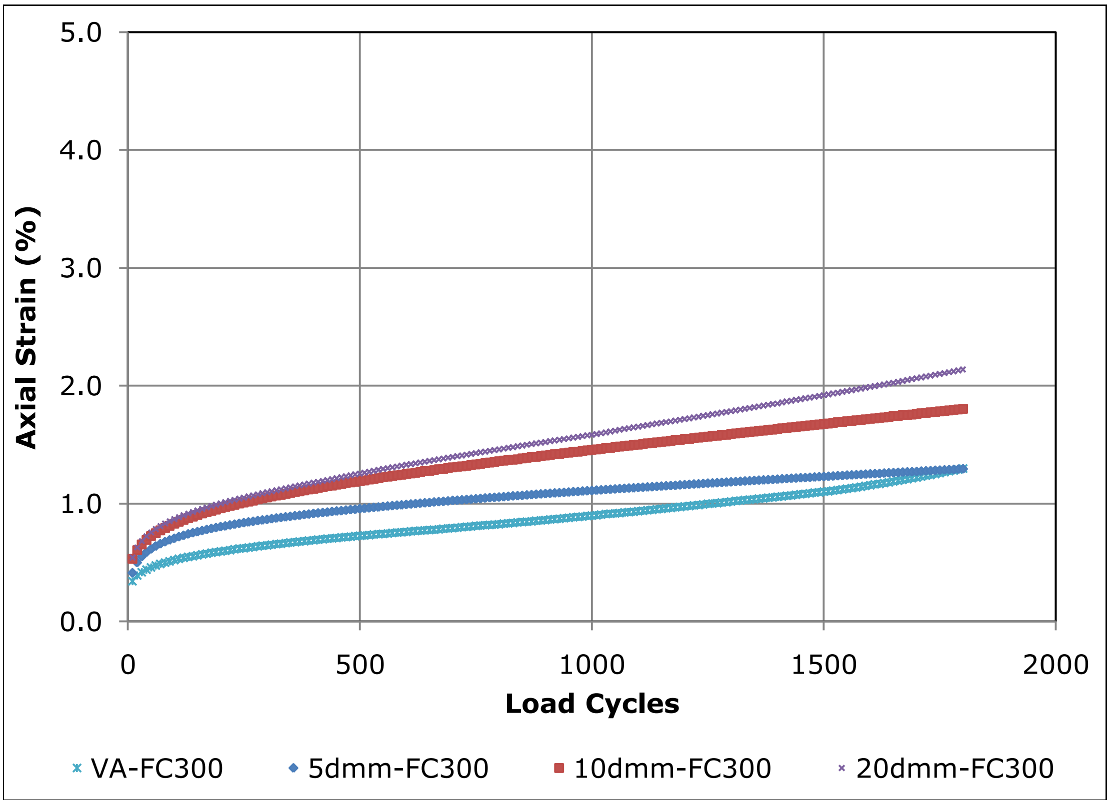


Figure 6.47: RLAT @40^{0}C & 300kPa Stress for CBEMs at Fully Cured Condition Mixed & Compacted @ 32^{0}C & 200 gyrations

performance as they are more temperature susceptible. This consequently gave a wider room for movement within the mix. The aggregate-to-aggregate contact in the VACBEM played the pivotal role in the resistance to deformation. In fact, the VACBEM seem to have behaved more or less like unbound granular material at this level compared to the RAP CBEMs. This is in consonance with the observation made for the good performance of the VACBEM for high stress levels in fatigue response. Oruc et al (2006) are among a host of other researchers who have similarly found this trend for cold mixtures. From Figures 6.44 and 45, all the materials are pretty good since strain is less than 1% (Thom, 2008).

6.10.2 The effect of curing on deformation properties of CBEMs

Further detailed analysis was conducted to properly rank performance at early life and fully cured conditions of the CBEMs. In general, researchers have identified 3 distinct stages for full deformation tests conducted to failure (Gibb, 1996). In Stage 1 (primary creep phase) for example, the densification of the specimen essentially takes place. During this stage, vertical strain accumulation increases rapidly though with a progressive decrease in the accompanying permanent deformation. This stage is subsequently followed by Stage 2 (secondary creep phase) which witnesses a constant strain rate. This stage mimics primary deformation of asphalt mixture. The 3rd stage which is referred to as the tertiary creep phase witnesses a rapid increase in vertical strain until the specimen fails (Taherkhani, 2006; and Nguyen, 2009). As was earlier stated, the number of load cycles applied in this test was limited, and as such in most cases the plots only gave a representation of permanent deformation of the CBEMs in the first stage and some part of the second stage of the creep tests, while the unexpected behaviour of the 5dmmCBEM in Figure 6.46 spanned these three stages.

Therefore a rutting indicator analysis was conducted thus. The earlier referred figures indicate that stage 1, the densification stage terminates at the about 1000 load cycles, most especially for the tests conducted at 100kPa stress level. It is obvious that at this stage, vertical strain accumulation was high at the onset and that it stabilised thereafter. At the second stage which is assumed to have started at 1001 cycle, the vertical deformation displayed a linearly varying relationship with loading cycles. Since stage 1 is a densification stage, it is predominated by air void content and thus judging the rutting potential of the CBEMs on the entire 1800 cycles only could be misleading. The analysis was thus based on the secondary stage in which air voids have little or no role to play. More importantly, the CBEMs have assumed a more linear relationship for vertical strain with corresponding load cycles at that stage.

Table 6.25 details the calculated strain rates and the average air void contents of the CBEMs tested at each level of observation. The strain rate was calculated thus;

$$\text{Strain Rate} = \frac{\textbf{Strain at 1800 Pulse} - \textbf{Strain at 1000 Pulse}}{800} \qquad 6.17$$

Figures 6.48 to 6.51 were plotted using the information in Table 6.25. The Table also contains information on the standard deviation of strains at 1800 pulse. Except for the test conducted at 300kPa on fully cured specimens where the 5dmmCBEM displayed the lowest strain rate, the VACBEM demonstrated the lowest strain rate value in all other cases. The 300kPa test both in the early life and fully cured conditions showed a more definite and distinct order compared to the 100kPa test except for the behaviour of the 5dmmCBEM. The strain rate increases from VACBEM up the range to the 20dmmCBEM. This observation is approximately reflected in Figure 6.49. In line with logic, the strain rate and likewise the strain values for the CBEMs at fully cured conditions were lower than those for the early life mainly because at early life, the materials were still evolving. Overall, the VACBEM performed better than the RAP CBEMs since it has lower active binder content in the mix and hence aggregate-to-aggregate contact in the VACBEM played the pivotal role in the resistance to deformation. The results also seem not to have been affected by the air void contents of the CBEMs as the VACEBEM which performed best in permanent deformation resistance consistently displayed the highest air void contents of all the CBEMs studied.

6.11 The Vacuum Repeated Load Axial Test (VRLAT)-Confined Mode of Testing

This test is a development on the RLAT and is also referred to as confined creep test (see Plate D in Figure 6.9). Nunn et al (2000) stated that, although RLAT discriminates between asphalt mixtures of the same composition utilising different binders, it cannot discriminate between mixtures with different aggregate gradations. Read and Whiteoak (2003) in describing this test stated that, the specimen is evacuated through one of the loading platens (via a porous stone or slots machined into the platen). The loading platen is ported through its body to a vacuum pump. The latex sealing membrane, of the type used in triaxial testing of soils, is fitted over the specimen and both the upper and lower platens. This is then secured using rubber ‘O’ rings that are located in a groove machined around the circumference of each platen in the plane of the surface of the platen.

Table 6.25: Rutting Indicator for CBEMs at Early Life and Fully Cured Conditions Tested at 40⁰C RLAT

Test Stress Level &Curing Condition	CBEM Type	Air Void (%)	Rutting Indicator			
			Mean Strain @ 1000 Pulse	Mean Strain @ 1800 Pulse	SD for Strain @ 1800 Pulse	Strain Rate
100kPa, Early Life	VACBEM	16.2	4.43E-01	4.62E-01	0.071	2.40E-05
	5dmmCBEM	16.0	5.81E-01	6.30E-01	0.033	6.14E-05
	10dmmCBEM	12.5	5.53E-01	6.08E-01	0.121	6.92E-05
	15dmmCBEM	12.6	5.73E-01	6.36E-01	0.043	7.97E-05
	20dmmCBEM	10.6	5.10E-01	5.56E-01	0.081	5.75E-05
	VACBEM-V*	17.0	3.46E-01	3.50E-01	0.037	5.17E-06
100kPa, Fully Cured	VACBEM	17.2	5.56E-01	5.80E-01	0.083	3.05E-05
	5dmmCBEM	15.7	5.49E-01	5.96E-01	0.035	5.90E-05
	10dmmCBEM	14.1	5.18E-01	5.67E-01	0.054	6.08E-05
	20dmmCBEM	11.6	6.33E-01	6.81E-01	0.093	5.99E-05
300kPa, Early Life	VACBEM	16.9	1.02E+00	1.17E+00	0.162	1.82E-04
	5dmmCBEM	15.9	2.00E+00	4.78E+00	1.067	3.48E-03
	10dmmCBEM	15.5	1.53E+00	2.27E+00	0.524	9.28E-04
	15dmmCBEM	12.2	1.75E+00	2.52E+00	0.305	9.55E-04
	20dmmCBEM	10.9	1.78E+00	2.63E+00	0.450	1.07E-03
300kPa, Fully Cured	VACBEM	15.7	8.98E-01	1.30E+00	0.854	4.98E-04
	5dmmCBEM	15.9	1.11E+00	1.29E+00	-**	2.31E-04
	10dmmCBEM	15.3	1.46E+00	1.80E+00	0.094	4.34E-04
	20dmmCBEM	11.6	1.58E+00	2.14E+00	0.915	6.92E-04

** VACBEM tested for the VRLAT at 100kPa* *** Only 1 Specimen Tested*
SD = Standard Deviation

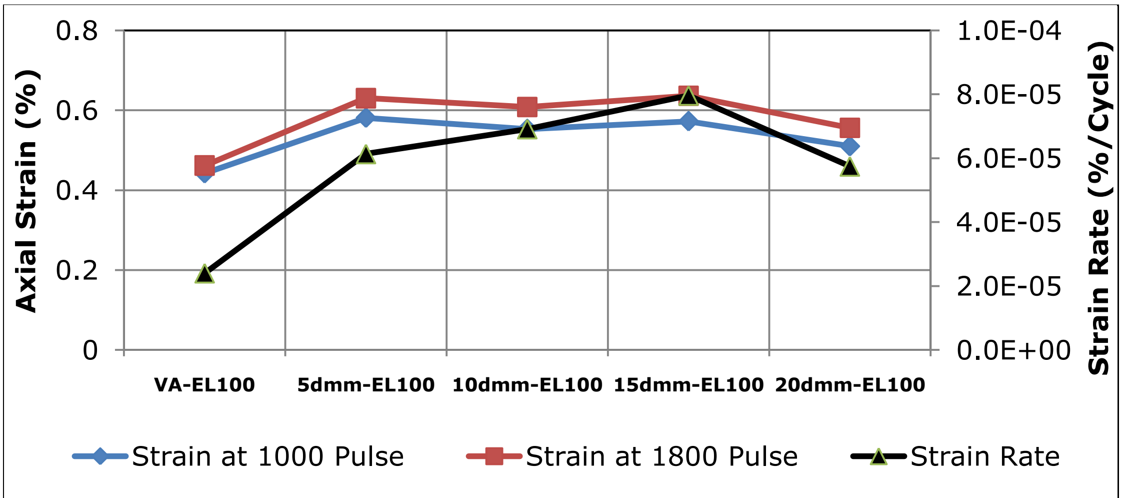


Figure 6.48: Rutting Indicator for CBEMs Conditioned to Early Life and Tested at 100kPa and 40⁰C for RLAT

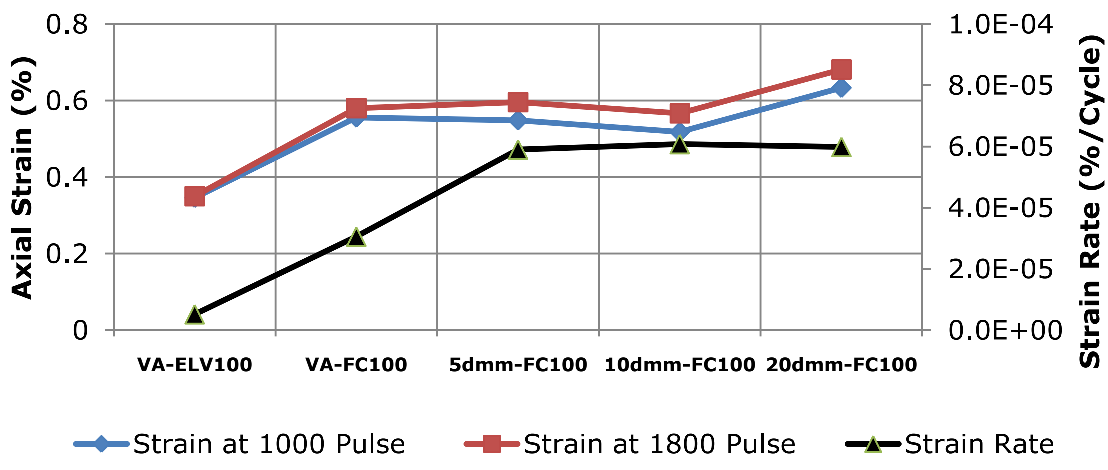


Figure 6.49: Rutting Indicator for CBEMs at Fully Cured Condition and Tested at 100kPa and 40^{0}C for RLAT

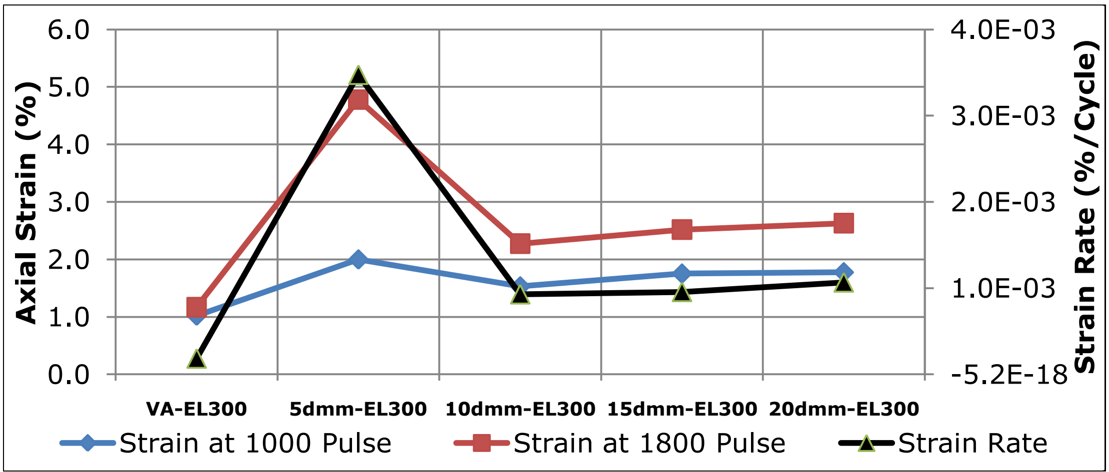


Figure 6.50: Rutting Indicator for CBEMs at Early Life Condition and Tested at 300kPa and 40^{0}C for RLAT

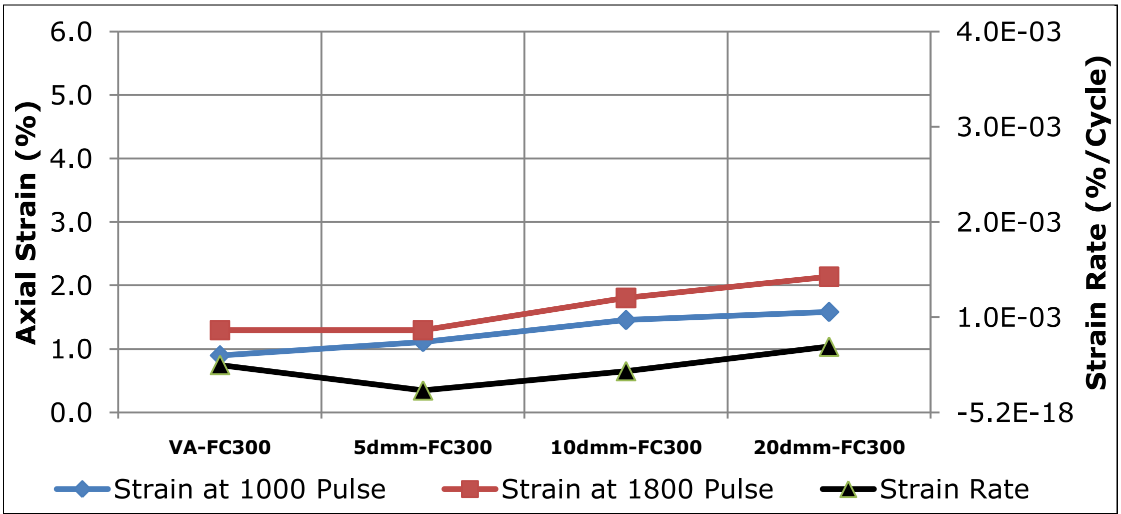


Figure 6.51: Rutting Indicator for CBEMs at Fully Cured Condition and Tested at 300kPa and 40^{0}C for RLAT

VRLAT test conditions are the same as those of RLAT except for the confinement pressure which is 50kPa. Figure 6.52 shows a full picture of the VRLAT mode in the NAT. This test was conducted in order to gain clearer knowledge of the permanent deformation properties of the CBEMs since this test simulates better the behaviour of CBEMs in service.

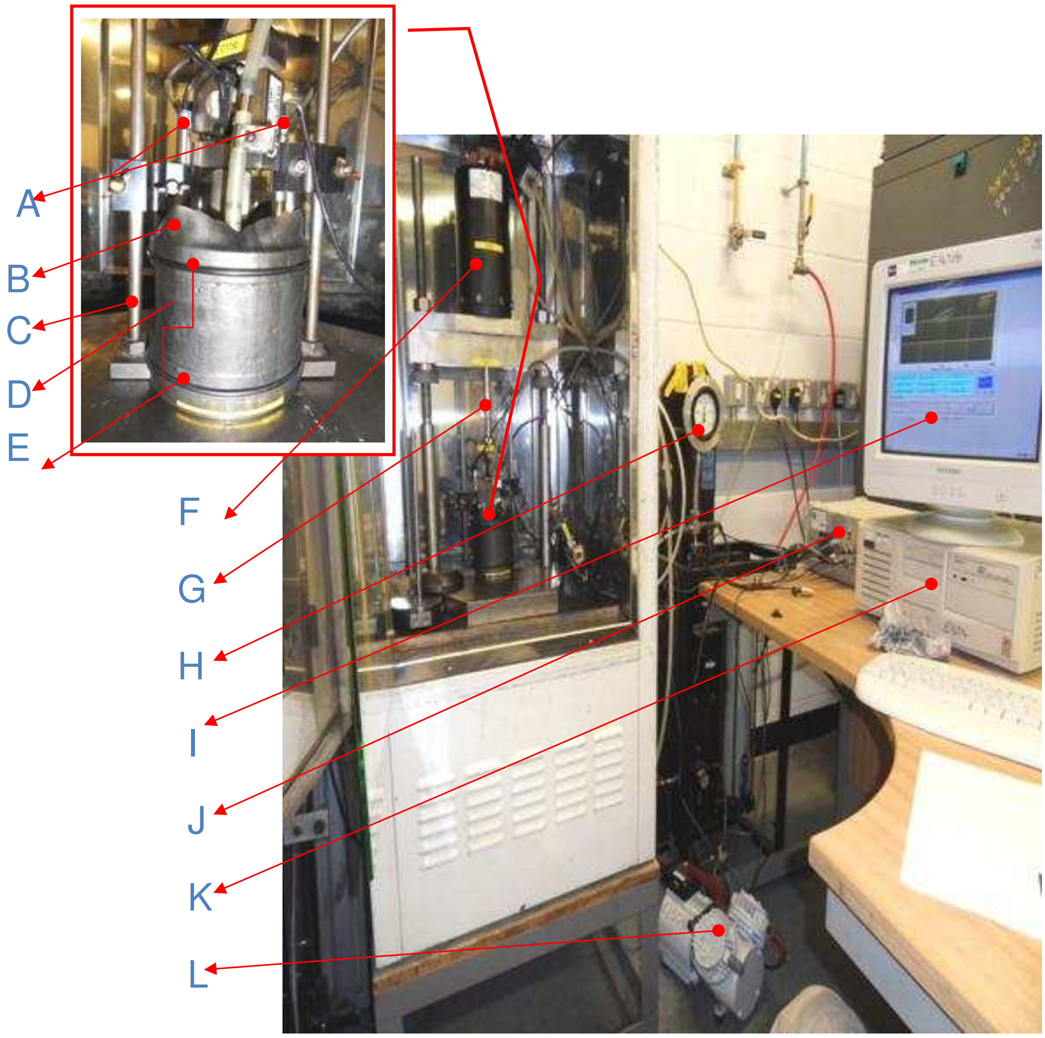
The test was conducted on similarly treated specimens to those in the preceding section but only at 300kPa. It was felt that conducting the test at 100kPa i.e. the normal test stress level, might not yield any useful result for the chosen test period of 1800cycles. The strain recorded for the VACBEM tested at this level was very low as indicated in Figure 6.44. For each level of observation, 3 specimens were tested and, the quoted strains are averages of the 3 specimens.

Figures 6.53 and 6.54 show the results of the investigations. The figures clearly confirm the views earlier expressed about the VACBEM. VACBEM displayed the lowest axial strains irrespective of its state or condition. Contrary to logic, the 10dmmCBEM followed in rank although the trend being followed by the 5dmmCBEM particularly in Figure 6.54 suggests that beyond 1800cycles, it might start performing better than the 10dmmCBEM. The 15dmmCBEM which was included in Figure 6.53 unexpectedly had the largest axial strain. The behaviour is in conformity with what was earlier observed for the CBEM under the RLAT and no logical explanations can be given here except for variations in the process of material preparation.

Meanwhile, just as was done for the RLAT results, a rutting indicator analysis was carried out. Table 6.26 details among other things the strain rate and air void contents of the CBEMs and also the standard deviation of strains at 1800 pulse. Figures 6.55 and 6.56 are relevant. The trend reflected here is in agreement with the results obtained for the RLAT most especially the strain rates for the RLAT conducted at 100kPa other than the large values displayed by the 5dmmCBEM. The strain rates for the fully cured condition were consistently lower than those recorded for early life. This similarly was observed for the axial strains. As was earlier stated, the VACBEM in this VRLAT displayed the best performance in resistance to permanent deformation. However, the performances of the RAP CBEMs were still within standards since all the axial strains recorded were below 2% (Thom, 2008).

6.12 Resilient Modulus Test

This test is also known as Repeated Load Triaxial Test. The test is known to



A: LVDTs B: Membrane for Confinement C: Loading Frame D: Specimen E: 'O' Rings F: Actuator G: Load Cell H: Confinement Pressure Gauge I: Monitor J: Data Acquisition Box K: Computer L: Vacuum Pump

Figure 6.52: A Cold Recycled Cylindrical Core being tested in the NAT using the VRLAT Mode

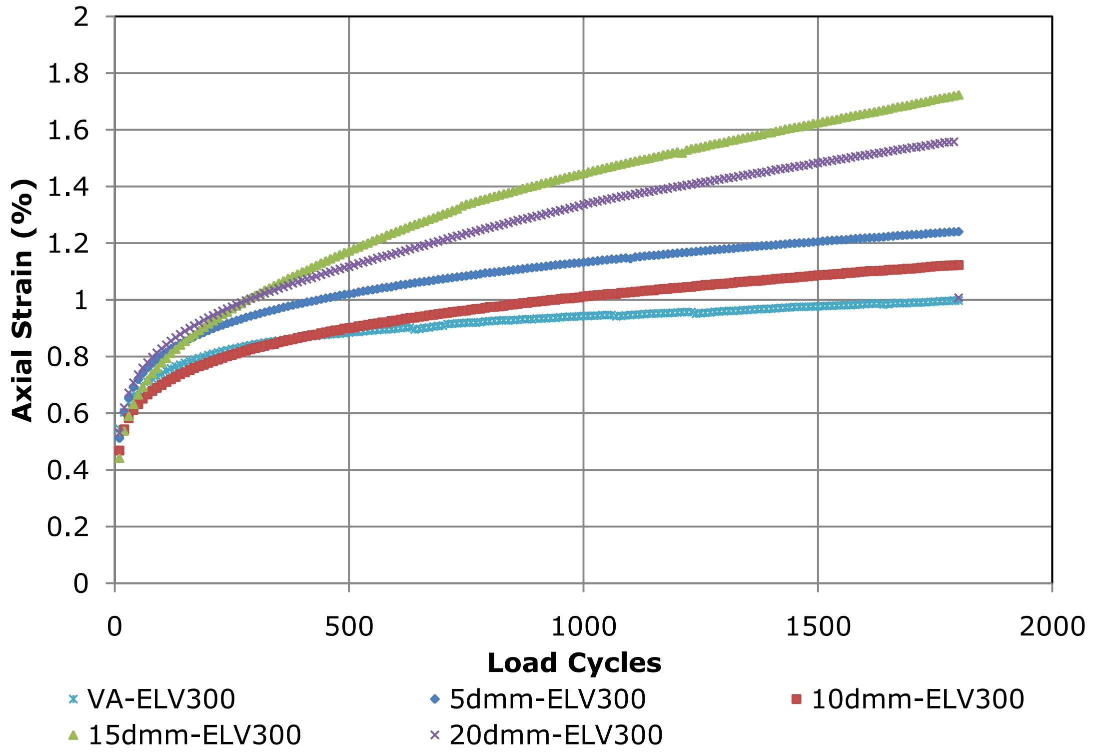


Figure 6.53: VRLAT @40^{0}C, 300kPa Stress and 50kPa Confinement Pressure for CBEMs at Early Life Mixed and Compacted @ 32^{0}C @ 200 gyrations

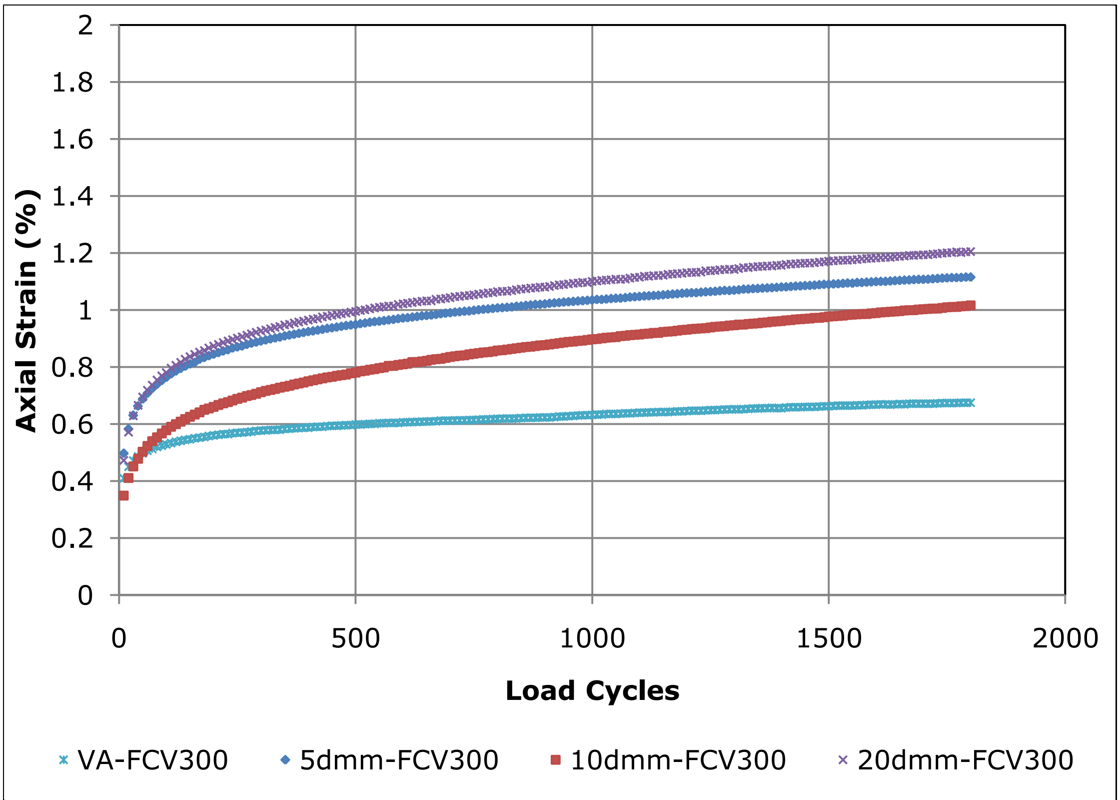


Figure 6.54: VRLAT @40^{0}C, 300kPa and 50kPa Confinement Pressure for CBEMs at Fully Cured Condition Mixed and Compacted @ 32^{0}C @ 200 gyrations

Table 6.26: Rutting Indicator for CBEMs at Early Life and Fully Cured Conditions Tested at 300kPa, 50kPa Confinement and 40°C for VRLAT

CBEM Condition	Air Voids (%)	Rutting Indicator			
		Mean Strain @ 1000 Pulse (%)	Mean Strain @ 1800 Pulse (%)	SD for Strain @ 1800 Pulse	Strain Rate
VA-ELV300	17.0	9.43E-01	9.98E-01	0.125	6.88E-05
5dmm-ELV300	15.5	1.13E+00	1.24E+00	0.235	1.35E-04
10dmm-ELV300	12.8	1.01E+00	1.12E+00	0.419	1.39E-04
15dmm-ELV300	12.4	1.45E+00	1.72E+00	0.154	3.47E-04
20dmm-ELV300	11.0	1.34E+00	1.61E+00	0.578	3.39E-04
VA-FCV300	16.9	6.32E-01	6.75E-01	0.058	5.42E-05
5dmm-FCV300	15.3	1.04E+00	1.12E+00	0.104	1.00E-04
10dmm-FCV300	13.6	8.97E-01	1.02E+00	0.065	1.49E-04
20dmm-FCV300	11.1	1.10E+00	1.21E+00	0.503	1.32E-04

Note: SD = Standard Deviation

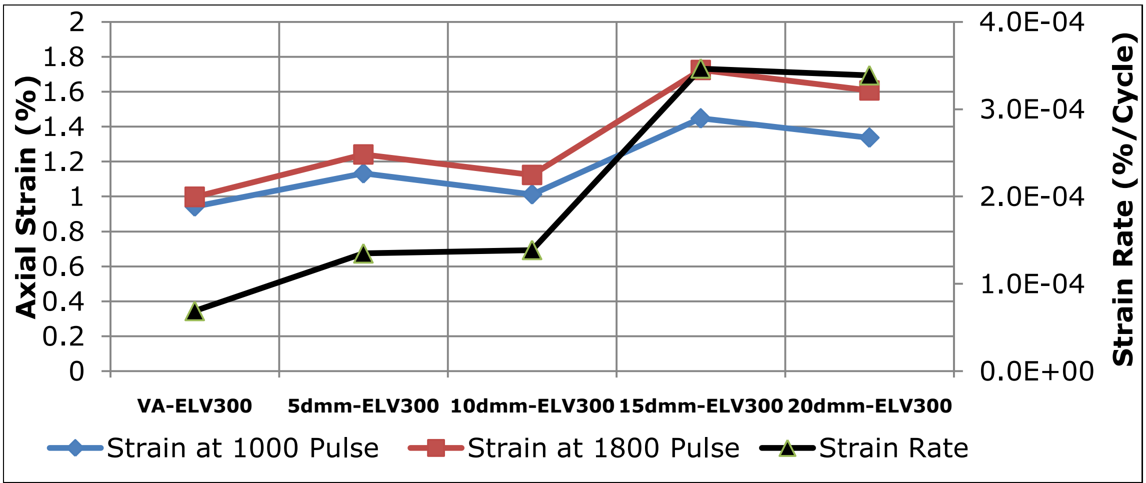


Figure 6.55: Rutting Indicator for CBEMs at Early Life Conditions Tested at 300kPa, 50kPa Confinement and 40°C for VRLAT

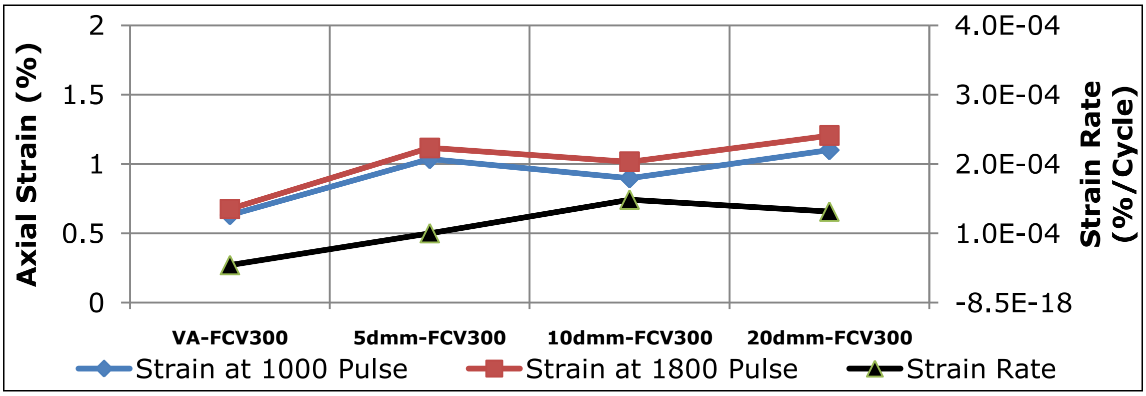


Figure 6.56: Rutting Indicator for CBEMs at Early Life and Fully Cured Conditions Tested at 300kPa, 50kPa Confinement and 40°C for VRLAT

simulate vehicle loading in a road pavement. Also, specimens assume a much more realistic stress state compared to unconfined tests.

The concept of resilient modulus (M_R) was initially introduced by Seed et al (1962) for characterizing the elastic modulus response of subgrade soils in flexible pavements. Due to its reliability in both measurement and application, resilient modulus response of unbound granular materials was used in the American Association of State Highway and Transportation Official (AASHTO) design guide for pavement structures (AASHTO, 1986). For this reason and many others, Read and Whiteoak (2003) stated that the test is extensively used in research to study permanent deformation as well as the elastic properties of asphalt. Just like the VRLAT, it has the advantage that both vertical and horizontal stresses can be applied at the levels predicted in the pavement. However, Anochie-Boateng et al (2009) in a recent review carried out to evaluate test methods for estimating resilient modulus, observed that during the past decades, research groups and agencies in the different countries have proposed test methods and procedures for repeated load testing to help in establishing appropriate resilient modulus test procedures for pavement design. In their review for example, out of the seven different procedures mentioned, three were from South Africa alone. Also the trend in testing for resilient modulus in the industry has since been extended to fully bound and intermediate mixtures.

It implies thus that presently there is no universally accepted procedure for conducting the test. Anochie-Boateng et al (2009) opined that there is a need for improved or new procedures to better estimate resilient properties of pavement materials. Among other things, they suggested the use of simple but correct equipment along with reliable software for successful testing of resilient modulus. Such would aid the migration of the test from research labs into the industry.

This test, although time consuming compared to the ITSM test was thus embarked upon as a step towards filling the identified gap above and at the same time gaining a better understating of the behaviour of the CBEMs studied in this work. Although this test is also good for characterising the deformation properties of road pavement materials, the focus here was essentially on the resilient modulus. Though both vertical and horizontal strains induced by stresses are normally monitored in such tests, it was only the vertical deformation that was monitored in this investigation. This was due to limitations of the on-specimen instrumentation considering the fact that the VRLAT mode that was used in this investigation uses specimens that are vacuumed by using a latex membrane.

All the specimens used in this work were manufactured in the gyratory compactor and were tested in a manner representative of both early life and fully cured conditions using the Nottingham Asphalt Tester (NAT) in the Vacuum Repeated Load Axial Test (VRLAT) mode. The digitally controlled servo-pneumatic system of the NAT combined with a newly developed Universal Software allowed for a multi-stage testing protocol. A haversine wave form with pulse width of 0.124s and rest period of 1.116s was employed throughout the tests. This was specifically chosen in order to relate the results to those obtained during the ITSM test. As earlier mentioned, ITSM test is conducted at a rise time of 124±4ms.

It is worth noting that in order to gain knowledge of the ITSM and resilient modulus behaviours of the specimens at the same time, the earlier described sample size of 100±2mm diameter and 50±2 thickness was maintained. Specimen conditioning involving 1000 cycles of 100kPa loading at a holding load of 20kPa was adopted having been found suitable after a series of trials. This was subsequently followed by multi-stage tests of 100 load applications each at the 8 target deviatoric stress levels of 50, 100, 200, 300, 400, 500, 600 and 700kPa. A constant confining pressure of 50kPa was applied using an external vacuum pump. This was adopted since there was no significant difference between 50kPa which was better achieved using the vacuum pump and 80kPa. The tests were conducted at 20, 30 and 40^{0}C.

Meanwhile for each cycle of loading, the Universal Software was pre-programmed to capture 500 data points for the actuator position (mm), load (kN), LVDTs (2 of) position, vertical stress (kPa), vertical strain (μstrain), vertical deformation (mm) and output (V). Although this will obviously give rich information for each test, the quantity of data involved warranted limiting the data set captured to the last ten cycles. For each cycle of loading, the corresponding, lowest point ante-peak, the peak point and the lowest point post peak for the vertical strain were extracted. The peak load for each cycle was similarly extracted. The resilient strain (recoverable) is the difference between the peak point and the lowest point post-peak, while the permanent deformation is the difference between the lowest point ante-peak and the lowest point post-peak. These values for each cycle as analysed were averaged and subsequently used for the determination of resilient modulus.

Figure 6.52 shown in the preceding section details a complete set up for this resilient modulus test. 3 specimens were tested at each level of observation and the average of the results was reported. Figure 6.57 details the stress state in road pavements and the definition of both permanent and resilient deformation.

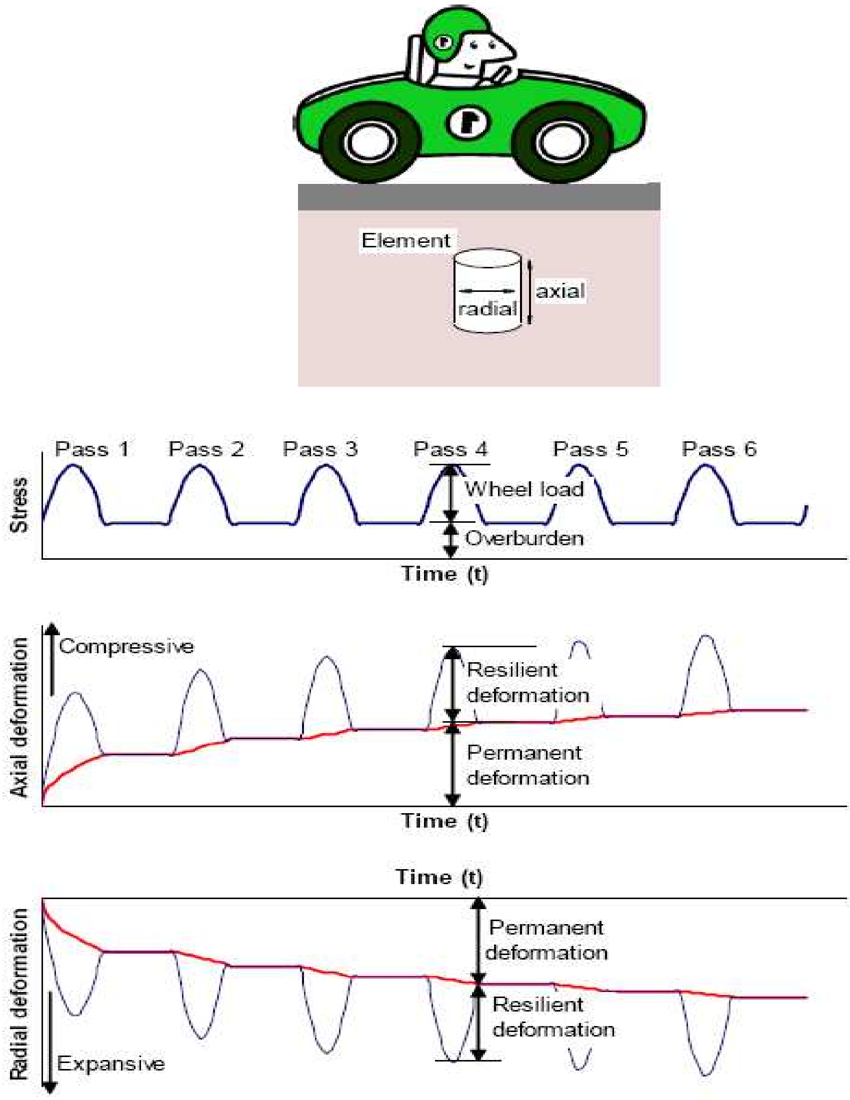


Figure 6.57: Loading in Pavements under Traffic (Arnold, 2004)

Figures 6.58a and 6.58b detail the K- Θ model used to fit unbound granular materials and the stress state of specimens under the resilient modulus test. Figure 6.59a and 6.59b detail the waveform for ten stress (or load) and vertical strain cycles respectively as observed in this work. Figure 6.60 shows a typical test screen at the completion of the multi stage test. The blue bracket intentionally inserted into the figure shows the conditioning period, while the red bracket shows the multi-stage test period.

As indicated in Figures 6.57 and 6.58, the resilient modulus is calculated thus:

$$M_R = \frac{\sigma_d}{\varepsilon_r} \qquad 6.18$$

Where:

M_R = Resilient Modulus
σ_d = deviatoric stress
ε_r = Resilient Strain

The resilient modulus was then plotted against the bulk stress to check the goodness of fit of the data obtained to the K- Θ model. The relationship between the resilient modulus and the bulk stress is as expressed in Figure 6.58a and similarly thus:

$$M_R = k_1 \theta^{k_2} \qquad 6.19$$

Where:

k_1 and k_2 are related to material properties, θ is the bulk stress and calculated from:

$$\theta = \sigma_1 + \sigma_2 + \sigma_3 \qquad 6.20$$

$\sigma_2 = \sigma_3$ = confinement pressure = 50kPa in this test

$$\sigma_1 = \sigma_d + \sigma_3 \qquad 6.21$$

σ_d is the deviatoric stress (cyclic stress applied during the test).

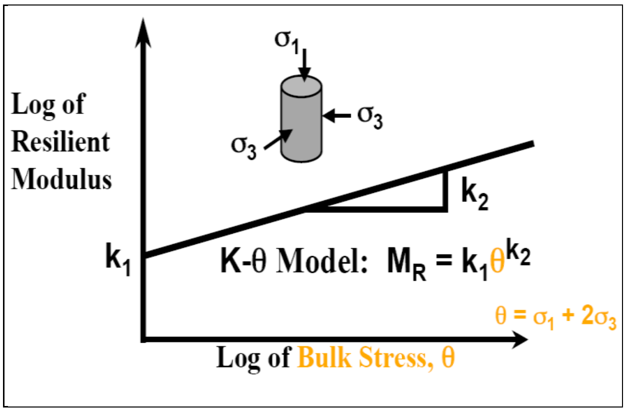


Figure 6.58a: Bulk Stress (K- Θ) M_R Model (Mishra and Tutumluer, 2007)

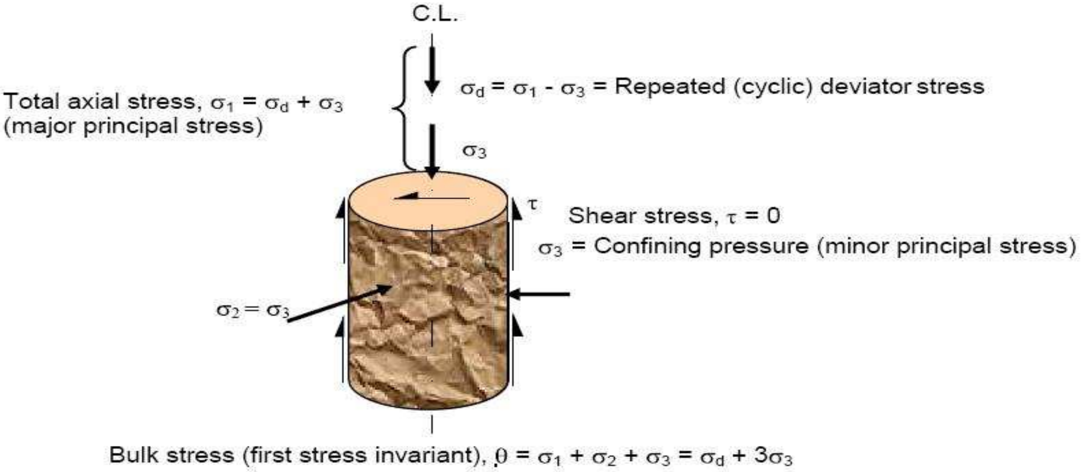


Figure 6.58b: Stress State for Resilient Modulus Testing (Anochie-Boateng et al, 2009)

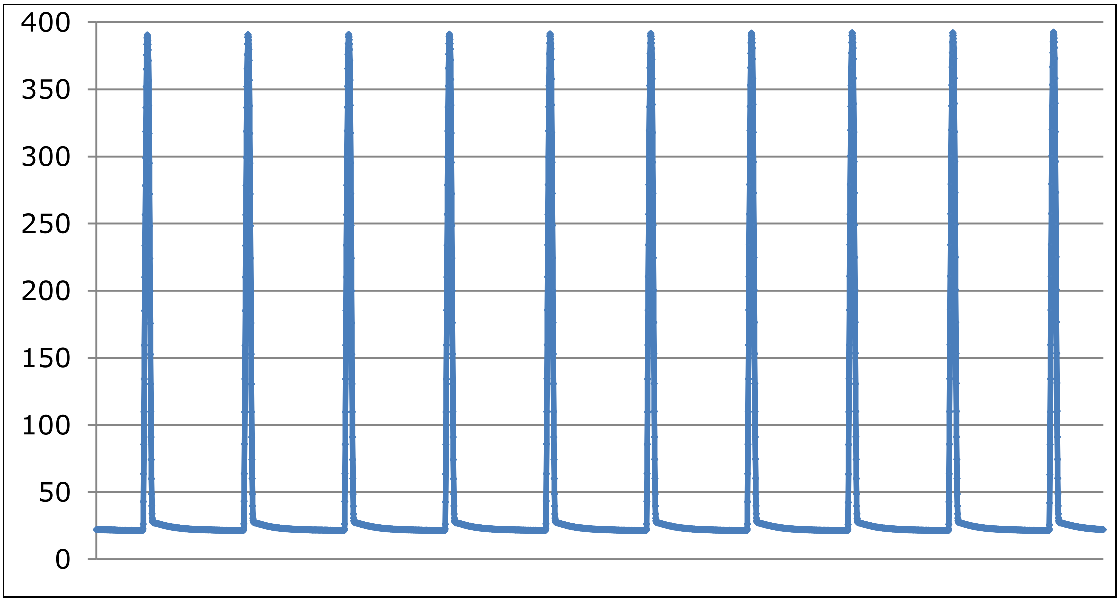


Figure 6.59a: Vertical Stress (kPa) for Ten Complete Cycles during Resilient Modulus Test

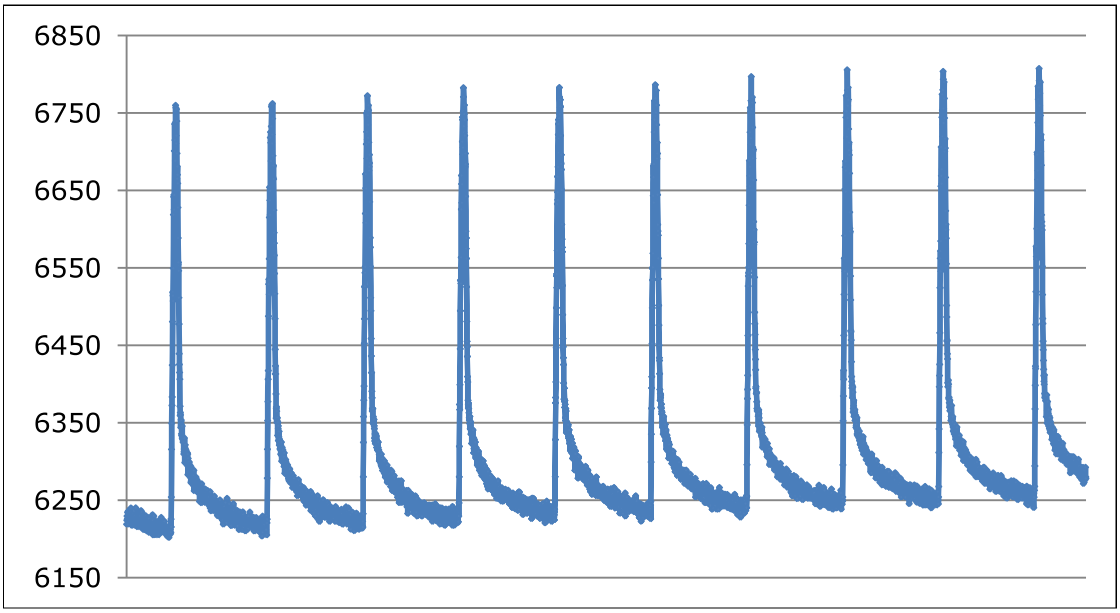


Figure 6.59b: Corresponding Haversine Waveform for Ten Complete Cycles of Vertical Strain (μstrain) in the Resilient Modulus Test

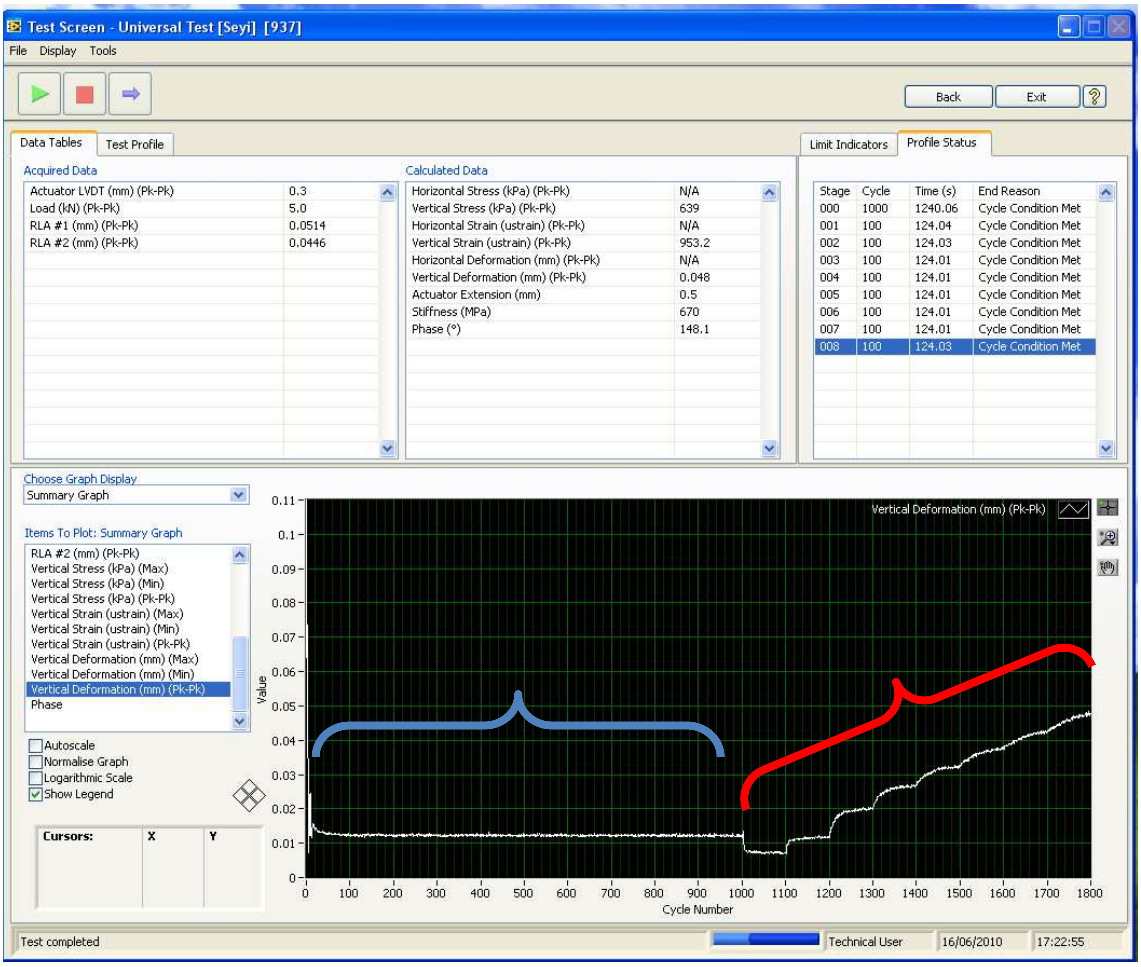


Figure 6.60: Typical Multi-Stage Test Screen Showing Vertical Deformation of a Specimen

6.12.1 Sensitivity of the resilient modulus test and the CBEMs

Figures 6.61 to 6.71 are relevant here. They all show the plots of resilient modulus (M_R) against bulk stress for the CBEMs. Table 6.27 details the mean M_R values and the standard deviation. The results clearly indicate that the resilient modulus of the mixtures investigated in this work are stress dependent as they all fitted the k-Θ model in equation 6.18. The goodness of fit as indicated by the respective R-squared of the plots was in all the cases greater than 0.97. The test as it was done here was also able to discriminate the effect of testing temperature on the resilient modulus responses of the CBEMs. Figures 6.61 to 6.65 show the individual performances of the CBEMs. As expected and in consonance with the ITSM test findings (though with an exception to the 10dmmCBEM), the fully cured (F.C.) specimens and the early life alike, tested at 20ºC, showed the best performance. These were followed next in rank by the same materials tested at 30ºC. Those specimens tested at 40ºC showed the least good performance.

Since the test as it was done here was able to discriminate the properties of the mixtures with regard to temperature susceptibility, it was felt that it might also be able to discriminate the CBEM type. Figures 6.66 to 6.71 showed that the VACBEM indicated the poor performance irrespective of the condition of curing and the test temperature. As expected also, the best performances for the CBEMs were recorded at a test temperature of 20ºC on the fully cured specimens. This was closely followed by those tested at similar temperature but conditioned to early life. At early life, while the RAP CBEMs were still able to contain the effect of testing at 30ºC on the resilient modulus, the VACBEM was unable and thus performed poorly at this level compared to the other CBEMs although as expected the worst performance of the VACBEM was recorded at 40ºC. The most interesting aspect here was that the test has the potential to clearly discriminate the CBEMs at a test temperature of 40ºC as indicated in Figure 6.71. The figure clearly indicated the trend of each of the CBEMs compared to the ITSM results at such temperature (Figure 6.19) which was less clear. The resilient modulus result is more reliable in this regard as it simulates stress condition in the pavement.

6.12.2 The effect of air voids, and stiffness modulus on the resilient modulus of CBEMs

Using the equations of fits for the CBEMs, and substituting 250kPa as the bulk stress, a regression analysis was conducted to ascertain the effect of air void content on the resilient modulus properties of the CBEMs just as was earlier done

Table 6.27: Statistics for the Resilient Modulus (M_R) of CBEMs

CBEM & Test Condition	VACBEM		5dmmCBEM		10dmmCBEM		15dmmCBEM		20dmmCBEM	
	Mean M_R (MPa)	SD	Mean M_R (MPa)	SD	Mean M_R (MPa)	SD	Mean M_R (MPa)	SD	Mean M_R (MPa)	SD
EL@ 20ºC	947	74	1139	49	1194	134	1115	50	1168	70
EL@ 30ºC	741	38	931	50	1075	10	981	37	924	83
EL@ 40ºC	632	19	776	38	765	33	738	62	759	48
FC@ 20ºC	1020	128	1142	20	1085	80	-	-	1225	86
FC@ 30ºC	904	43	970	77	1099	15	-	-	1065	27
FC@ 40ºC	720	57	825	57	841	76	-	-	904	53

\- *Not tested* *SD= Standard Deviation*

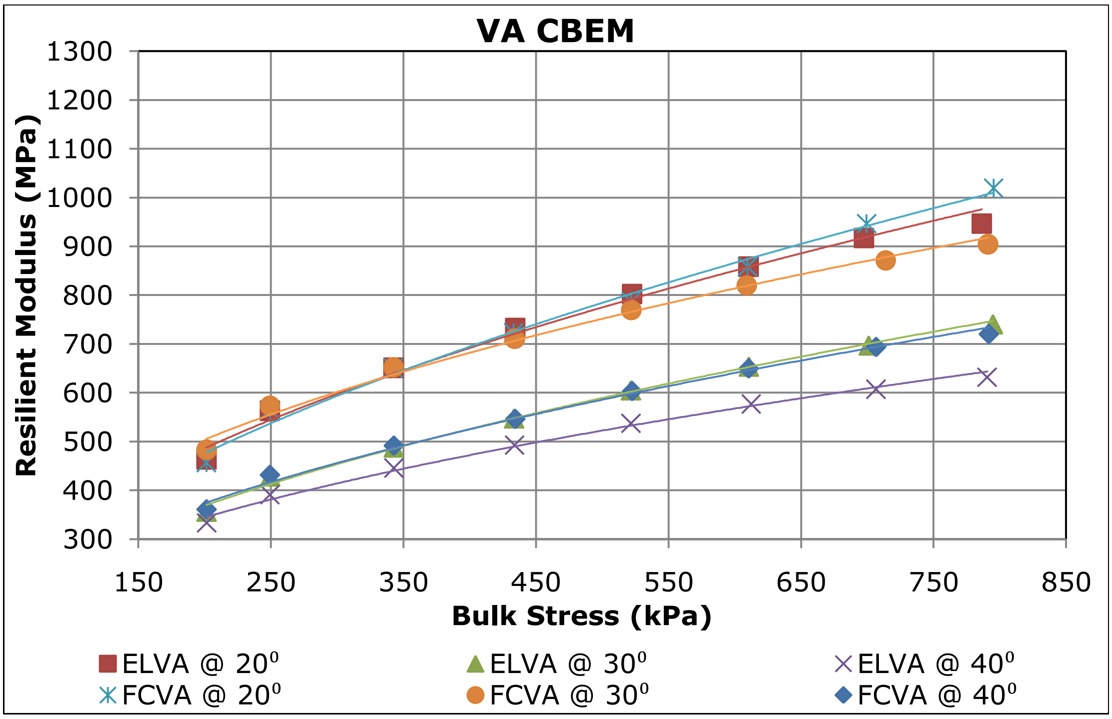


Figure 6.61: Resilient Modulus Response of VA CBEM

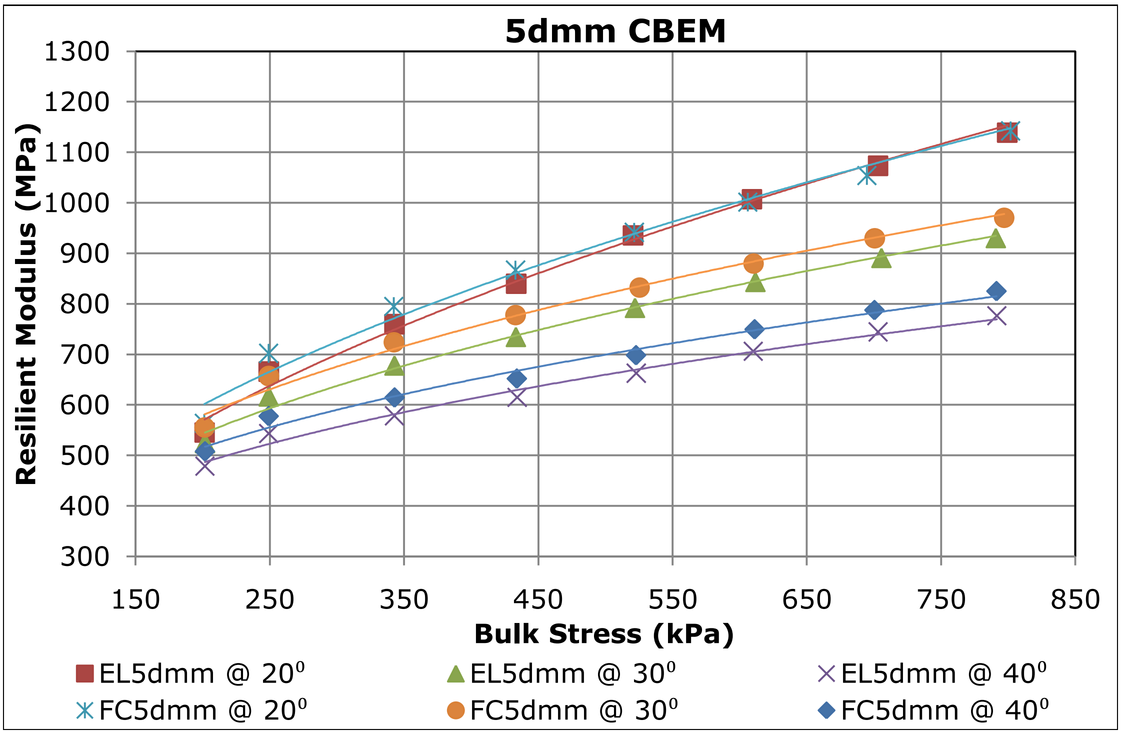


Figure 6.62: Resilient Modulus Response of 5dmm CBEM

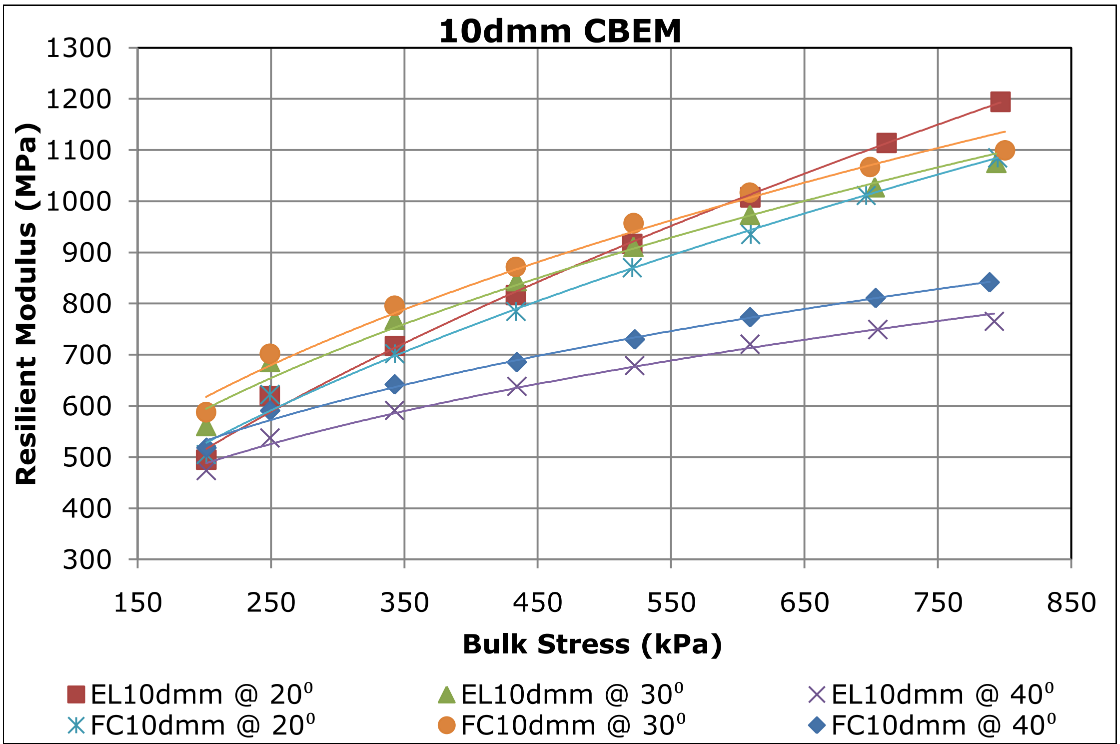


Figure 6.63: Resilient Modulus Response of 10dmm CBEM

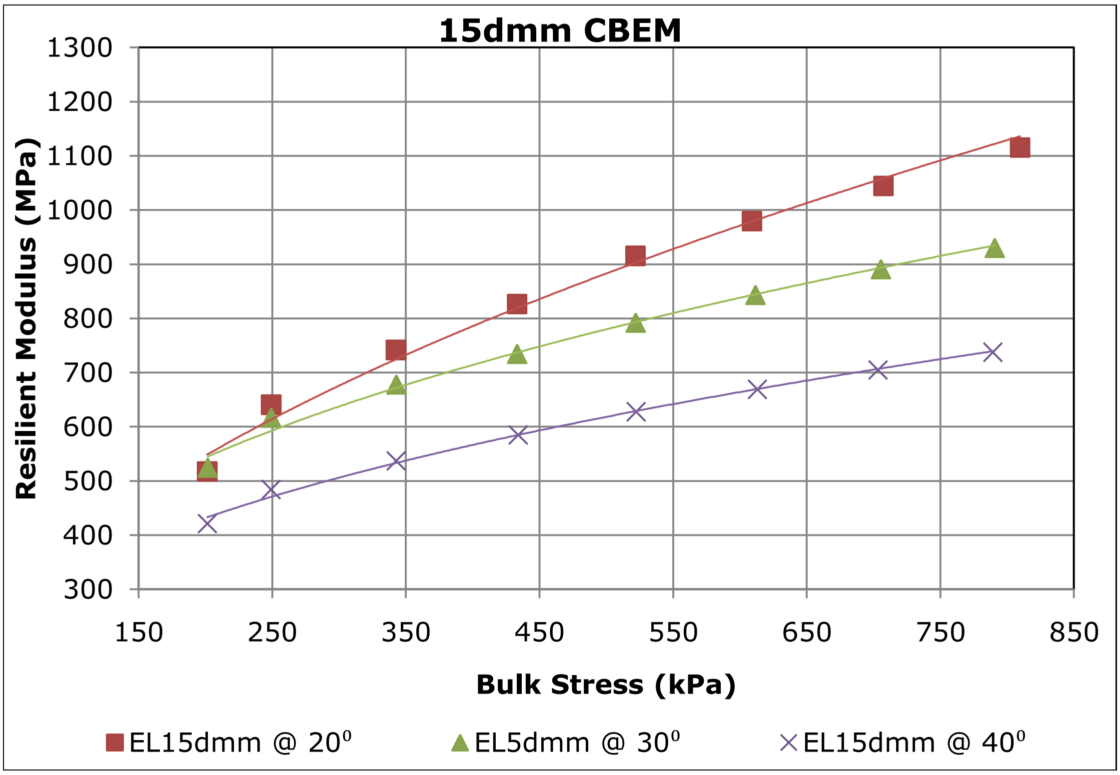


Figure 6.64: Resilient Modulus Response of 15dmm CBEM

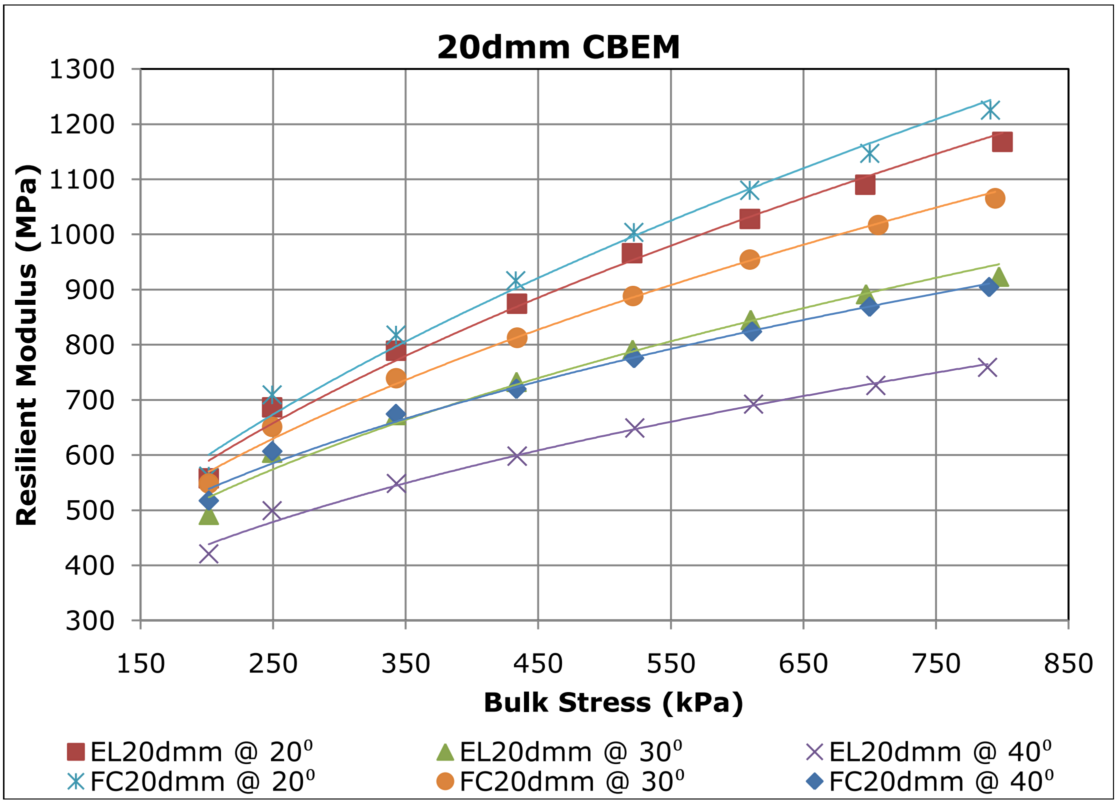


Figure 6.65: Resilient Modulus Response of 20dmm CBEM

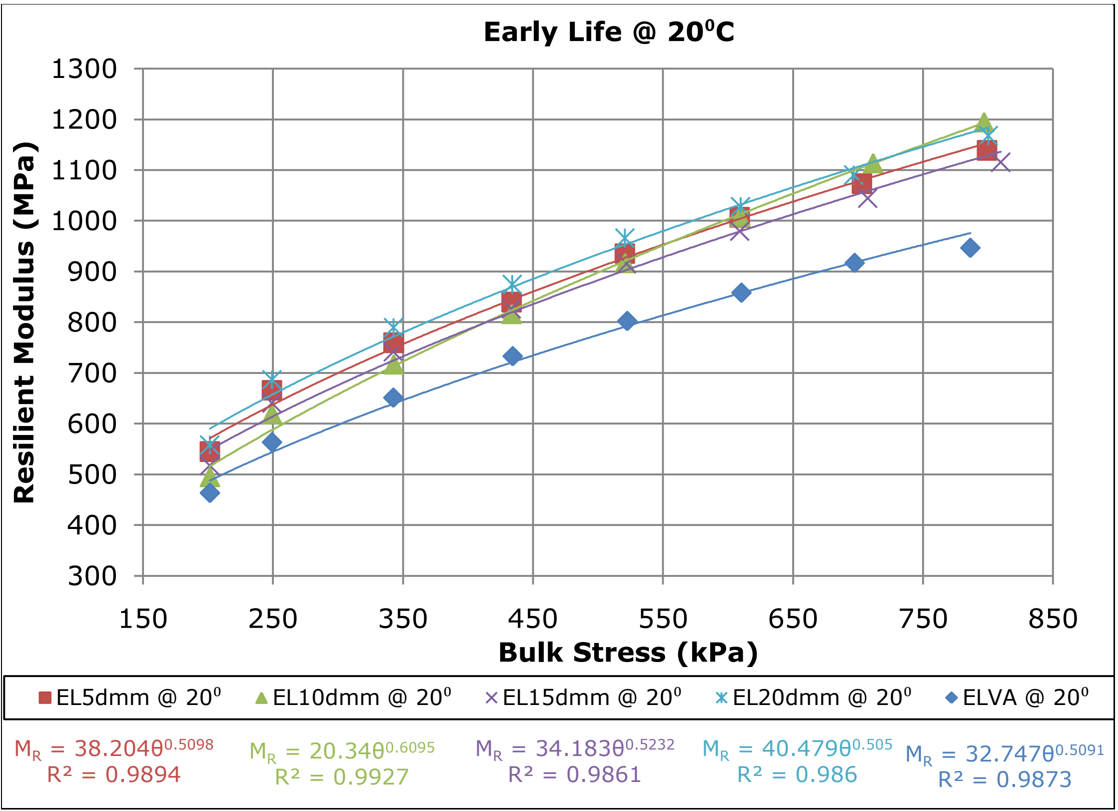


Figure 6.66: Resilient Modulus Response of CBEMs Tested at 20°C for Simulated Early Life Condition

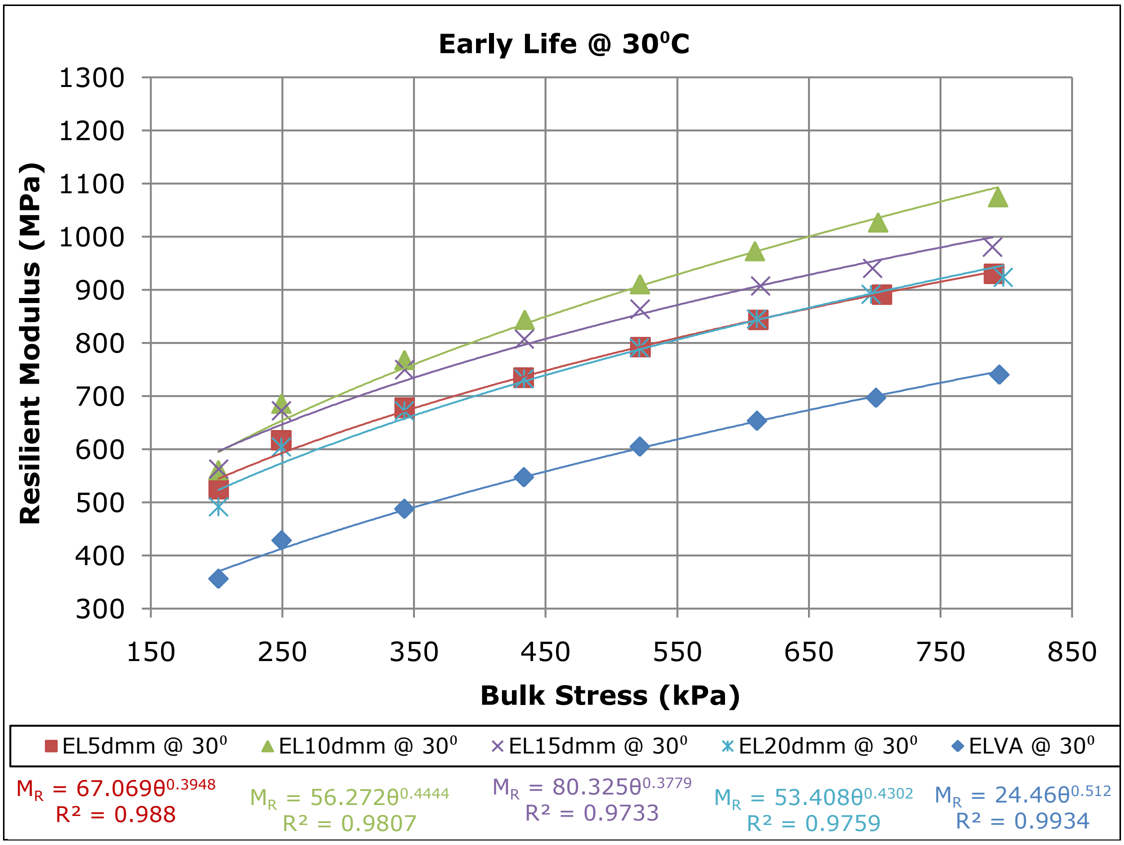


Figure 6.67: Resilient Modulus Response of CBEMs Tested at 30°C for Simulated Early Life Condition

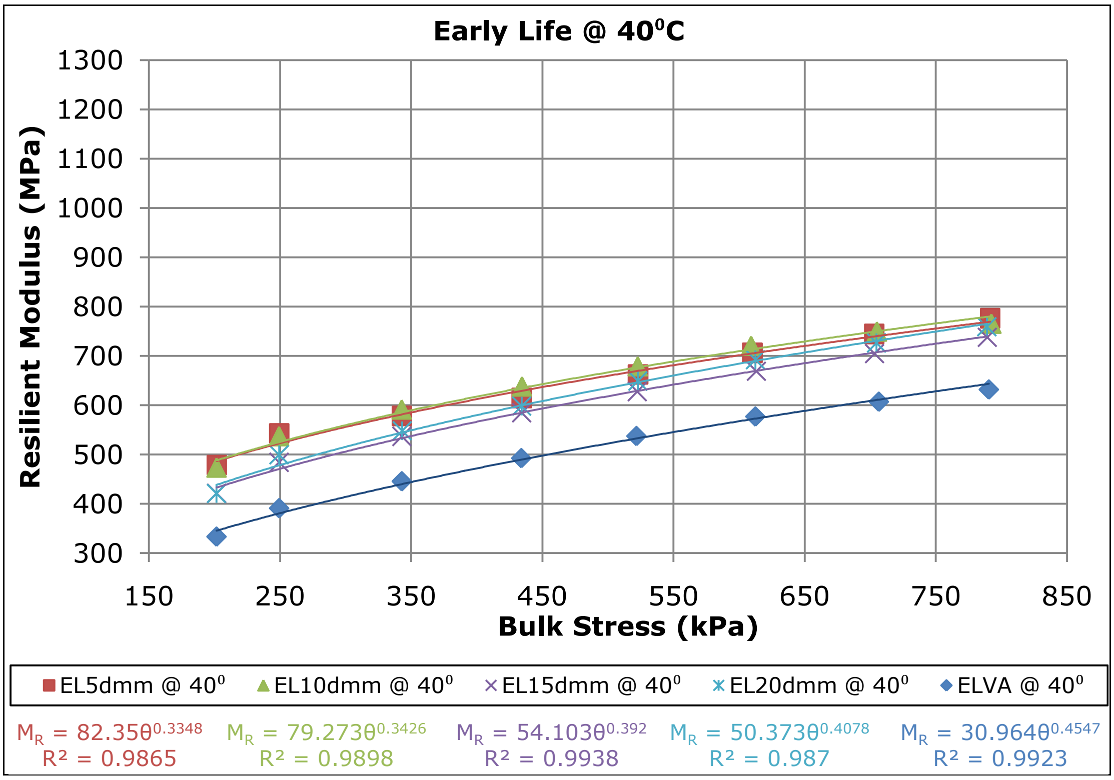


Figure 6.68: Resilient Modulus Response of CBEMs Tested at 40⁰C for Simulated Early Life Condition

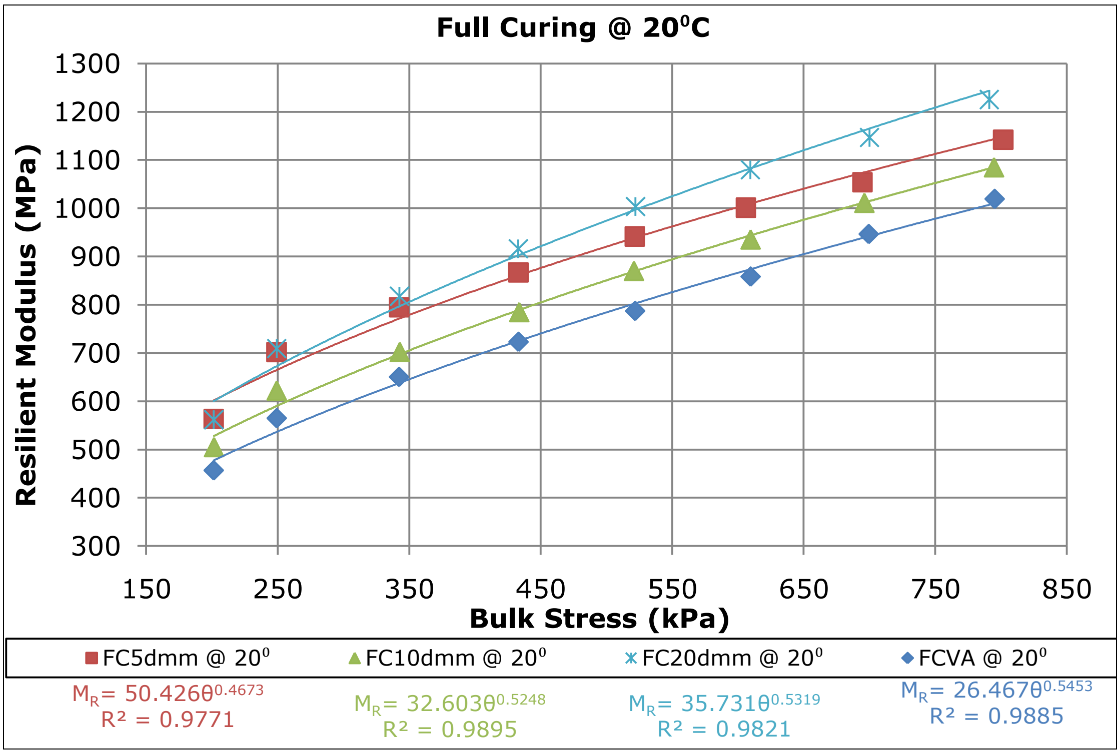


Figure 6.69: Resilient Modulus Response of CBEMs Tested at 20⁰C for Simulated Fully Cured Condition

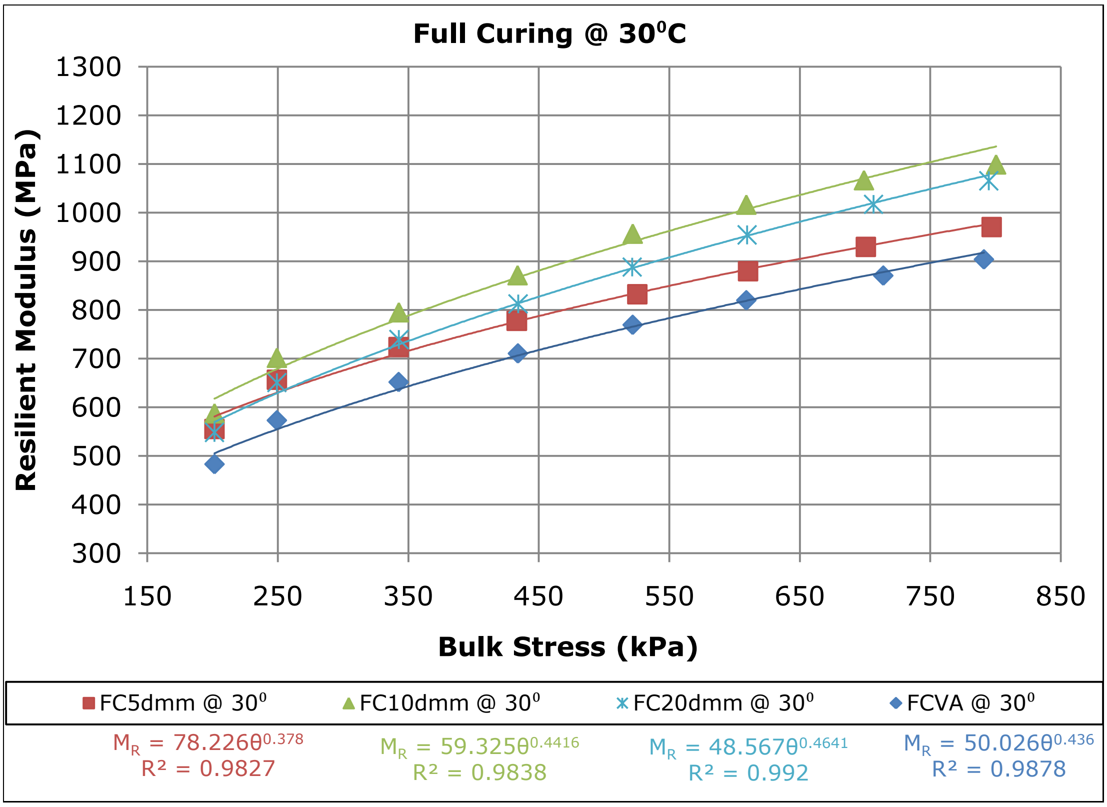


Figure 6.70: Resilient Modulus Response of CBEMs Tested at 30⁰C for Simulated Fully Cured Condition

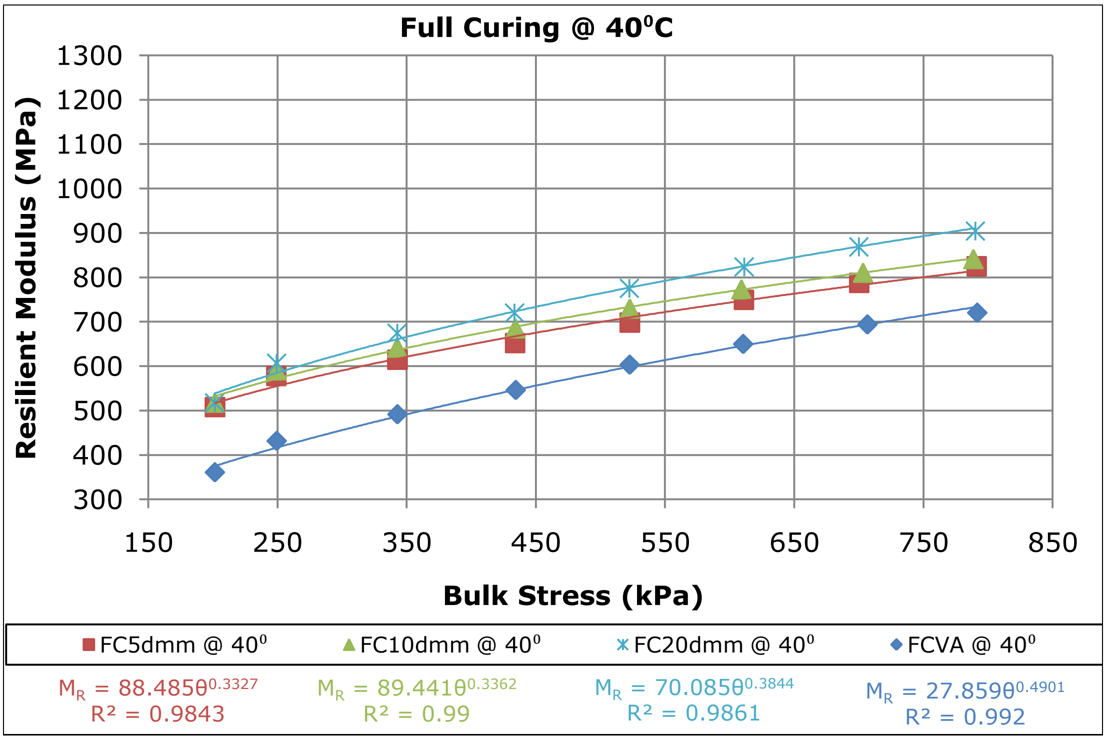


Figure 6.71: Resilient Modulus Response of CBEMs Tested at 40⁰C for Simulated Fully Cured Condition

for ITSM.

Table 6.28 details the result of the One-Sample Kolmogorov-Smirnov (Normality) Test for M_R and Corresponding ITSM and Air Voids at Simulated Early Life and Fully Cured Conditions of the CBEMs. The result indicated normality for all the cases considered. Table 6.29 details the result of the multiple regression analysis carried out on the CBEMs. It is worthy of note that in order to have a relatively robust sample number, all specimens with similar treatments and tested at similar temperature were regressed together. The VACBEM was treated as the reference in the regression model while all the other CBEMs in the order, 5dmmCBEM, 10dmmCBEM, 15dmmCBEM (involved in three cases) and 20dmmCBEM were regarded as first, second, third and fourth independent variables respectively in the model.

A close inspection of Table 6.29 indicated that for the regression of M_R vs. Air voids, the only significance was recorded for specimens fully cured and tested at 20ºC. Similarly, for regression of M_R vs. ITSM, significance levels were recorded at fully cured, and tested at both 20ºC and 40ºC. The result in Table 6.29 did not indicate significance for all other cases. However, irrespective of these differences which are not significant, Figure 6.72 shows the potential losses in resilient modulus associated with 1% increase in air voids, which is logical. The trend in Figure 6.72 demonstrates that as the CBEMs cure further, an increase in air void makes the loss in modulus more significant. For example, in the case of the only observation in which significance was observed here, with a unit increase in air void content, a loss of 35MPa in resilient modulus is recorded. This trend was similarly observed for ITSM vs. air voids in section 6.8.4 (Figures 6.35 and 6.36), although stronger in the ITSM case than for resilient modulus.

Figure 6.73 did not indicate any trend in particular. However temperature significantly affects the M_R vs. ITSM relationship.

6.13 Summary of Results

This chapter focused on a detailed characterisation of five CBEMs as follows:

1. VACBEM- with 100% virgin binder and aggregates
2. 5dmmCBEM
3. 10dmmCBEM
4. 15dmmCBEM

Table 6.28: One-Sample Kolmogorov-Smirnov (Normality) Test for MR and Corresponding ITSM and Air Voids at Simulated Early Life and Fully Cured Conditions of CBEMs

Test	Parameters		EL @ 20^{0}C	FC @ 20^{0}C	EL @ 30^{0}C	FC @ 30^{0}C	EL @ 40^{0}C	FC @ 40^{0}C
M_R	Sample Size		15	12	15	12	15	12
	Normal Parameters[a]	Mean	609	617	576	624	476	533
		SD	89.421	78.036	98.596	59.470	67.608	83.898
	Kolmogorov-Smirnov Z		0.534	0.645	0.855	0.616	0.673	0.558
	Asymp. Sig. (2-tailed)		0.938	0.799	0.458	0.842	0.755	0.915
ITSM @ 20^{0}C	**Sample Size**		15	12	15	12	15	12
	Normal Parameters[a]	Mean	1852	3039	1909	3093	1916	3124
		SD	248	637	245	641	241	637
	Kolmogorov-Smirnov Z		0.539	0.436	0.389	0.576	0.398	0.77
	Asymp. Sig. (2-tailed)		0.933	0.991	0.998	0.894	0.997	0.590
Air voids	**Sample Size**		15	12	15	12	15	12
	Normal Parameters[a]	Mean	13.882	14.589	13.305	14.595	13.904	14.620
		SD	2.270	2.118	2.023	2.161	2.618	2.240
	Kolmogorov-Smirnov Z		0.493	0.443	0.585	0.515	0.602	0.892
	Asymp. Sig. (2-tailed)		0.968	0.989	0.884	0.954	0.862	0.404

a. Test distribution is Normal.

Table 6.29: Results of Regression Analysis of CBEMs for M_R vs. Air voids and M_R vs. ITSM

Regression	CBEM & Test Condition	Ref. Parameter	Coef.	Std Error	t	P>|t|	[95% Conf. Interval]	
M_R vs. Air voids	EL@20^{0}C	Air void	-0.933	29.239	-0.03	0.975	-67.08	65.21
	EL@30^{0}C		23.669	14.831	1.60	0.145	-9.88	57.22
	EL@40^{0}C		-27.541	18.046	-1.53	0.161	-68.36	13.28
	FC@20^{0}C		-35.260	4.814	-7.32	0.000	-46.64	-23.88
	FC@30^{0}C		-13.295	21.017	-0.63	0.547	-62.99	36.40
	FC@40^{0}C		-62.533	83.869	-0.75	0.480	-260.85	135.78
Mr vs. ITSM	EL@20^{0}C	ITSM	0.102	0.163	0.63	0.547	-0.27	0.47
	EL@30^{0}C		0.007	0.077	0.10	0.926	-0.17	0.18
	EL@40^{0}C		-0.108	0.082	-1.31	0.222	-0.29	0.08
	FC@20^{0}C		0.102	0.030	3.39	0.012	0.03	0.17
	FC@30^{0}C		-0.001	0.036	-0.01	0.989	-0.09	0.09
	FC@40^{0}C		-0.213	0.081	-2.64	0.033	-0.40	-0.02

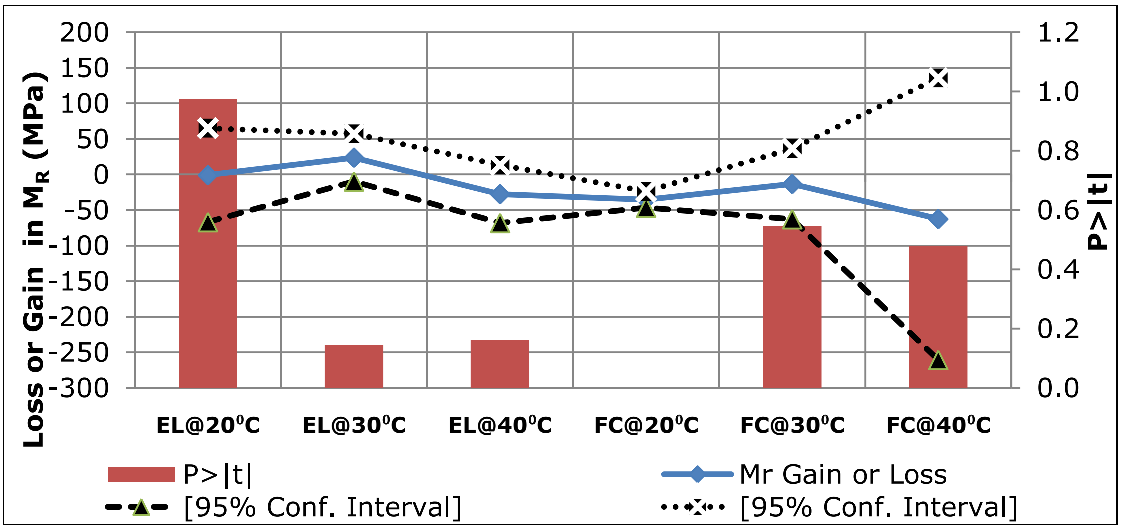


Figure 6.72: Loss or Gain in M_R Tested at 20⁰C, 30⁰C & 40⁰C at Early Life (EL) and Fully Cured (FC) Conditions due to a 1% Rise in Air Void Content

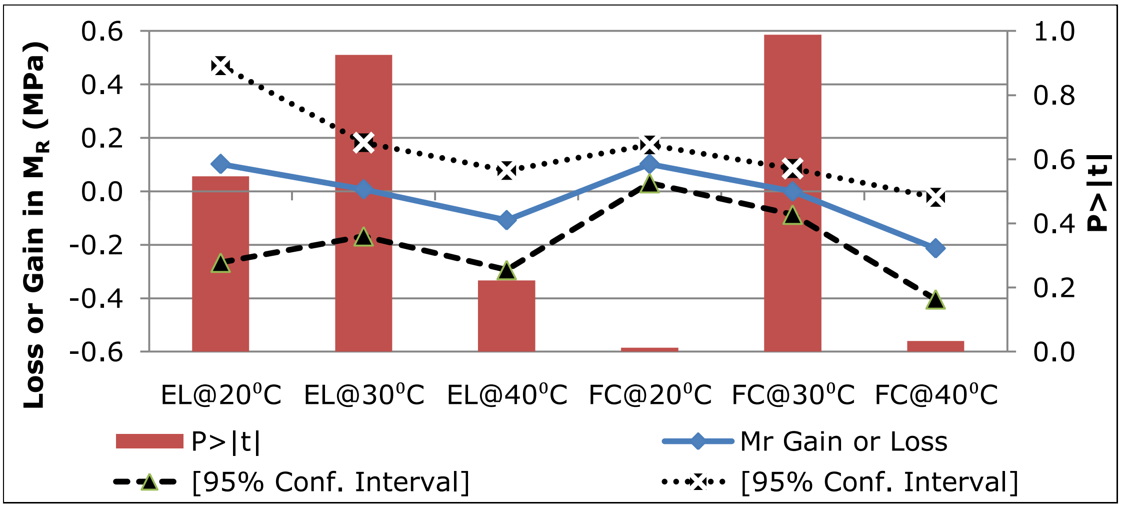


Figure 6.73: Loss or Gain in M_R Tested at 20⁰C, 30⁰C & 40⁰C at Early Life (EL) and Fully Cured (FC) Conditions due to a Unit Rise in ITSM

5. 20dmmCBEM

The CBEMS listed in 2 to 5 were constituted in the ratio 65:30:5 for RAP, fine aggregate (5mm) and filler respectively. The VACBEM was similarly constituted except that the RAP was replaced with virgin aggregate. The numbers in the respective CBEM names i.e. 5dmm indicate the penetration of the residual bitumen in the RAP. This was intentionally done in order to ascertain the effect of varying degrees of ageing in RAPs on the mechanical properties of CBEMs. Cationic bitumen emulsion was used for all the mixtures. Two mixing and compaction temperatures of 20ºC and 32ºC were used. Similarly, two compactive efforts of 100 gyrations and 200 gyrations were applied. The materials were all conditioned to early life (40ºC over 12hrs), intermediate (40ºC over 72hrs) and fully cured (60ºC over 96hrs) conditions using the forced draft oven. The VACBEM was used as the control.

The main tests conducted to assess the performance of the CBEMs were:

- Compactability tests using the gyratory compactor
- ITSM test to check stiffness modulus responses of the CBEMs
- Temperature Susceptibility tests using the ITSM test
- Water Susceptibility tests also using the ITSM test
- Fatigue tests using the ITFT
- Permanent deformation tests using
    - RLAT
    - VRLAT
- Resilient Modulus test using the VRLAT

The following general conclusions can be drawn from the studies:

Compactability

- CBEMs as observed in this study generally have high air void contents compared to HMA. They in some instances displayed values as high as 23% with the minimum experienced in this study being around 8%. However, the air void content depends on the condition of curing (and mix design) which essentially has to do with moisture content. Early life conditioned specimens displayed the lower values due to the presence of moisture, while the fully cured specimens had the high values. The moisture content at early life condition for all the CBEMs ranged from 1.0% to 1.4% while at fully cured condition it ranged from 0.2% to 0.8%.
- Air void content affects the rate of moisture loss in the CBEMs. The higher the initial air void content, the higher the rate of moisture loss from the CBEM. In this regard, the VACBEM showed the highest loss of moisture content.

- The better the compactability (or lower the air void content) of CBEM the higher the tendency to retain more moisture content in the compacted mixture. This however seemed not to have affected the stiffness properties of the CBEMs.
- Mixing and compaction temperature is very important for achieving low air void contents in CBEMs. Mixing and compacting CBEMs at 32ºC gave better results than at 20ºC.
- Mixing and compaction temperature showed a significant effect on compactability of CBEMs compared to compactive effort.
- At early life, the RAP CBEMs compacted better compared to the control mix i.e. the VACBEM. The 20dmmCBEM had the best compactability.
- Also, the study showed that applying high compactive effort on CBEMs at an operational temperature of about 20ºC may be appropriate, this however may amount to a waste of energy when the ambient temperature is around 32ºC.
- The results obtained for the fully cured condition similarly rated the RAP CBEMs high compared to the VACBEM. The observed air void contents also showed that the consistency and reproducibility of percentage performance responses of CBEMs mixed and compacted at 32ºC are uniform and significantly better compared to those mixed and compacted at 20ºC. The results also confirmed that there is little difference in applying a compactive effort of 100 gyrations and 200 gyrations whereas the difference is significant at mixing and compaction temperatures of 32ºC and 20ºC. CBEMs prepared at 32ºC showed superior quality.

Stiffness Modulus Responses

- The stiffness modulus responses referred to as ITSM for the CBEMs generally showed that the curing protocols used in this study sufficiently simulated the desired conditions in a logical order as stiffness improved up the range starting with early life, to intermediate life, and fully cured condition.
- At early life, the ITSM test clearly ranked the CBEMs in a logical order and the results were in agreement with findings on compactability of the CBEMs. 5dmmCBEM had the least percentage improvement over VACBEM of 8.8% while 20dmmCBEM had the highest of 32%. As the penetration of the residual binder in the severely aged RAP increases, there is a corresponding increase in ITSM. At intermediate life, all CBEMs mixed and compacted at 32ºC demonstrated ITSM values clearly above 2000MPa

irrespective of the compactive effort applied. The VACBEM performed better in all cases than the RAP CBEMs with the exception of the 20dmmCBEM.

- At fully cured condition as simulated in this study, the VACBEM was superior in ITSM response compared to the 5dmmCBEM. Also, there is an indication of better performance over the 10dmmCBEM and 15dmmCBEM by the VACBEM generally at the mixing and compaction temperature of 20ºC irrespective of the compactive effort applied. Although the 20dmmCBEM significantly improved here compared to the intermediate life ITSM responses, the result indicates the best response as expected at mixing and compaction temperature of 32ºC but at the lower compactive effort of 100 gyrations.
- Some of the results suggest that the 10dmm, 15dmm and 20dmm RAPs were not completely acting as black rock i.e. bitumen in RAP was influencing the behaviour of the RAP CBEM, most especially when constituted at 32ºC.
- Although the VACBEM performed better than some of the RAP CBEMs at intermediate and fully cured conditions, the very high air void contents consistently observed for VACBEM compared to the other RAP CBEMs obviously predisposes it to problems associated with durability in bituminous mixtures.

Temperature Susceptibility measured by ITSM

- ITSM responses tested at 20ºC for fully cured CBEMs mixed and compacted at 32ºC were generally above 2500MPa (with the exception of the 5dmmCBEM) irrespective of the compactive effort applied.
- The responses dropped to around 1500MPa for ITSM tests conducted at 30ºC and further down to around 650MPa for test conducted at 40ºC on the same specimens.
- Irrespective of the CBEM type, the condition of curing and testing temperature for ITSM, CBEMs prepared at 32ºC consistently performed better than those prepared at 20ºC even at an ITSM test temperature of 40ºC.
- The CBEM materials prepared at 32^{0}C, 100 gyrations have least loss in ITSM due to high temperature which implies just enough compactive effort along with appropriate temperature for preparation of CBEMs.
- VACBEM as observed here shows the highest temperature susceptibility.

Water Susceptibility

- The RAP CBEMs performed better than the VACBEM in water susceptibility.
- In Protocol 2 where recovery was monitored after the CBEMs were subjected to severe conditions, the VACBEM performed poorly compared to the other CBEMs.
- The poor performance of the VACBEM could have been partly caused by its high air void content, while the lower air voids of the other CBEMs could have been partly responsible for their better performance. More importantly, it is believed that the residual bitumen in the severely aged RAPs that constitute the RAP CBEMs is possibly being rejuvenated thus causing the RAP CBEMs to have larger volume of active binder. Furthermore, the performance could similarly be linked to the possible changes in the volumetrics of the RAP CBEMs as a result of the softening of the residual bitumen in the RAP and not necessarily rejuvenation.
- A trend which indicates that as the penetration of residual bitumen in RAP CBEMs increases, performance similarly increases was observed.

ITSM and Air voids

- There is a negative relationship between ITSM and air voids i.e. as air voids increase, stiffness modulus reduces for CBEMs. The results here indicate generally that as CBEMs cure, the air voids – stiffness relationship gets stronger.
- This is particularly observed for VACBEM as a unit rise in air void content could result in loss of ITSM as high as 240MPa for fully cured conditions.

Fatigue Response

- Fatigue lives of the CBEMs generally were lower than those for HMA.
- For the tests conducted at 30^{0}C, the result for the 5dmmCBEM in all cases was the best followed by the 20dmmCBEM while the VACBEM had the poorest performance.
- For tests conducted at 20^{0}C, the results for 1.0E + 03 cycles to failure indicated that the VACBEM had the least strain while the RAP CBEMs returned values generally greater than 200με. This trend was similarly observed for tests at 30^{0}C.
- The results imply that the VACBEM has the best resistance at high stresses compared to all the other mixtures. The 5dmmCBEM on the other hand returned the least strain for CBEMs prepared at 32^{0}C and tested at 30^{0}C.

- Overall, the RAP CBEMs prepared at a mixing and compaction temperature of 32^{0}C performed better in fatigue than their counterparts prepared at 20^{0}C.
- Also, considering the behaviour of both the VACBEM and 5dmmCBEM, it seems that the fatigue responses of CBEMs were not affected by air void contents although it was earlier established that air voids have a strong correlation with stiffness of CBEMs, depending on the level of curing attained. Generally, these two CBEMs have relatively high air void contents compared to the other CBEMs.

Deformation Properties

- In line with logic, the strain rate and the strain values (for the RLAT and the VRLAT) for the CBEMs at fully cured conditions were lower than those for the early life.
- VACBEM performed better than the RAP CBEMs. The results also seem not to have been affected by the air void contents of the CBEMs as the VACEBEM which performed best in permanent deformation resistance consistently displayed the highest air void contents of all the CBEMs studied.
- The performance of the RAP CBEMs seems to indicate more active binder contributing to the matrix (with the 20dmmCBEM having the greatest proportion) than the VACBEM. In the case of the RAP CBEMs the binding medium played a significant role which is reflected in their performance as they are more susceptible to rutting.
- The VACBEM seems to have behaved more or less like unbound granular material. Oruc et al (2006) among a host of other researchers similarly found this trend for cold mixtures.

Resilient Modulus

- The results clearly indicate that the resilient moduli of the mixtures investigated in this work are stress dependent as they all fitted the k-Θ model.
- The test as it was done here was able to discriminate the properties of the mixtures with regard to temperature susceptibility.
- As expected also, the best performances for the CBEMs were recorded at a test temperature of 20^{o}C on the fully cured specimens.
- The most interesting aspect here was that the test has the potential to clearly discriminate the CBEMs at a test temperature of 40^{o}C better than the ITSM test.

- Though this needs further investigation, the results here suggest that there seems to be a relationship between M_R and Air voids, such that with a unit increase in air void content there is a potential loss in resilient modulus just as for ITSM.
- The results also suggest that, as the CBEMs cure further, an increase in air void makes the loss more significant. For example, in the case of the only observation in which significance was observed, with 1% increase in air void content, a loss of 35MPa in resilient modulus is recorded.
- The results suggest similarly that the relationship between M_R and ITSM, is weak. The relationship seemed to have been influenced by temperature.
- The results show that the Indirect Tensile Stiffness Modulus (ITSM) is useful for quickly ranking CBEMs (most especially for early life condition) and whereas the resilient modulus may be more appropriate for pavement design involving such materials since they are non-linear in their behaviour as observed in this study.

6.14 Conclusions

The RAP CBEMs with residual binder of 10dmm or more showed improved compactability compared to the VACBEM indicating an active role of the residual binder, even in these heavily aged mixes. This was particularly the case at an elevated mixing and compaction temperature of 32ºC, typical of tropical ambient temperatures. At this temperature, the air void content of the RAP CBEMs was lower than that of the VACBEM, even at low compactive effort. While the VACBEM was in some cases as stiff as the RAP CBEMs, the higher air void content and lower active binder content led to higher levels of temperature and moisture susceptibility for stiffnesses compared to all the RAP CBEMs, down to residual penetration of 5dmm. This indicates an impact due to the presence of even severely aged residual binder in addition to improved compactability. Under these conditions, aged RAP is possibly not acting entirely as ‘black rock’.

The RAP CBEMs similarly were slightly better in fatigue and resilient modulus under the above mentioned ambient tropical conditions. Although, the VACBEM was better in permanent deformation resistance compared to the RAP CBEMs, the responses of the RAP CBEMs were still within standards. Furthermore, the results of the permanent deformation test also suggest that the RAP CBEMs had more active binder in the mix compared to the VACBEM which corroborates the earlier claim that the severely aged RAP is possibly not entirely acting as ‘black rock’.

7 Structural Design and Modelling of Flexible Pavement Containing CBEMs

7.1 Overview

The structural design of a flexible road pavement is required in order to estimate its design life. This in essence will ensure that the road pavement serves its purposes structurally and functionally in an economically viable manner within such estimated/predicted design life.

This design task for road pavements is complex (Ebels, 2008; and Twagira, 2010) and it is not as straightforward as obtained in other engineering structures (Thom, 2008). In fact Thom (2008) opined that there is never a unique solution under such circumstance. More importantly, long term characteristics of a pavement have to be borne in mind as much as the day one performance in the process of design. He further suggested that such design should aim to:

- Protect the subgrade
- Guard against deformation in the pavement layers
- Guard against break up of pavement layers
- Protect from environmental attack
- Provide suitable surface
- Ensure maintainability

As mentioned in the preceding chapter, researchers have realised that once fatigue and rutting (cumulative deformation) which are the two classical failure modes of pavements have been taken care of in design, then those earlier listed design aims by Thom (2008) are conveniently met (Read and Whiteoak, 2003). Thus, the structural design detailed in this chapter is mainly focused on fatigue and rutting.

Although researchers have commonly agreed that such design could be done empirically or by using mechanistic approach (Jitareekul, 2009; Ebels, 2008; Twagira, 2010), Huang (2004) in a more elaborate manner stated that there are five methods for executing such design. These are:

- Empirical method with or without a soil strength test
- Limiting shear failure method

- Limiting deflection method
- Regression method based on pavement performance or road test
- Mechanistic – empirical method

The mechanistic-empirical method of the listed five is particularly appealing and has been used for the works described in this chapter mainly because of the procedure involved. For example, Huang (2004) reported that the mechanistic-empirical method is based on the mechanics of materials that relates an input, such as a wheel load, to an output or pavement response such as strain or stress. The response values are subsequently used to predict distress from laboratory-test and field performance data. This aspect essentially makes this method suitable since many laboratory tests have been conducted in this present work and the results of such are useful in this regard. Huang (2004) believes that dependence on observed values is important because theory alone has not proven sufficient to design pavements realistically. The advantages of the mechanistic methods therefore are the improvement in the reliability of a design, the ability to predict the types of distress, and the feasibility to extrapolate from limited field and laboratory data.

Similarly, following the mechanistic-empirical method described above for the design of pavements containing the five (5) CBEMs (as a road base layer material) examined in the preceding chapter allows for a comparative analysis to ascertain the benefit of using RAP CBEMs or otherwise compared to the VACBEM. Although the study here is focused mainly on the road base layer which contains the CBEMs, Ebels (2008) suggested that the responses of interest for a full pavement analysis are:

- Horizontal tensile strain at the bottom of the HMA (surface layer)
- Principal stresses in the CBEM layer (road base)
- Horizontal strain at the bottom of the CBEM layer
- Vertical strain on top of the subgrade.

This chapter essentially focuses on examining the effect of CBEM type on the relevant responses (stresses and strains) using 16 pavement structures. Table 7.1 details the pavement structures. The structures have been chosen such that they represent the common practice in the industry. Flexible pavement failure criteria for fatigue and rutting are first discussed before briefly looking at the relevant modelling tools and requirements. A detailed description of non-linear elastic analysis is similarly given in this chapter. Since one of the objectives of this chapter is to check the performance of the CBEMs, the structural analysis started

Table 7.1: Pavement Layer Thicknesses used for the 16 cases Modelled

Pavement Structure Case	HMA (mm)	Road Base – CBEM (mm)	Sub Base (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		

**: Implies number of sub-layer divisions for non-linear analysis* by examining the stress and strain distribution in pavements containing the CBEMs using non-linear modelling tools because the resilient modulus test results in Chapter 6 suggested this form of behaviour. This is later extended to linear modelling tools for a comparative analysis in order to ascertain the suitable method of analysis for pavements containing CBEMs. Similarly, the maximum horizontal/vertical strains within the CBEM layers and those generated at the bottom of the layers were compared and the more suitable approach for structural analysis was adopted. The effect of the CBEMs on the fatigue life of the surfacing layer and as well the rutting potential of the subgrade is similarly examined and the results are used to produce hypothetical design charts. Although CBEMs both in the early life (EL) and fully cured (FC) condition were studied, only the FC ones have been reported here, since fatigue tests were only conducted on the FC CBEMs.

7.2 Flexible Pavement Failure Criterion

As earlier mentioned, researchers have established that the basic failure mechanisms of flexible pavements are fatigue and rutting related. Therefore, a failure criterion for a flexible pavement must be set in concert with the results obtained for the fatigue and rutting responses of the CBEM materials in the preceding chapter. The fatigue line equations from ITFT and the parameters obtained from the resilient modulus test are relevant here.

In the case of fatigue failure in asphalt pavements, Thom (2008) observed that a relationship is generally found between tensile strains in asphalts (at the bottom of the asphalt, immediately under the load or; near the surface, just outside the loaded area or; at the surface in the tyre tread contact zone) under traffic load and fatigue tests conducted in the laboratory. In this regard, 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 which cater for field (in-service) effects must be introduced, i.e. establish a relationship between laboratory and field performance, for a realistic criterion to be set. Oliveira (2006) opined that this is very difficult to do since such depends on the type of test, mode of loading, testing temperature and type of mixture. He observed that laboratory fatigue tests for example use sinusoidal loading and fixed strain or stress during one test, while in practice, the mode of loading is randomly distributed, including rest periods and lateral distribution of loads. He stated further that temperature variations in the asphalt layer and healing effects, due to intermittent loading, also influence the field performance of asphalts.

Oliveira (2006) reported that researchers in the light of the above have suggested various shift factors but with high diversity and no generally accepted one. Although his work focused on grouted macadam, he eventually found a shift factor of 41.4 suitable for fatigue failure criterion for grouted macadam. In a related work to the one described in this thesis, Jitareekul (2009) used the shift factors of 440 (1.1 for lateral wheel load distribution, 20 for rest periods and 20 for crack propagation) and 77 suggested by Brunton (1983) for adjusting laboratory determined fatigue lives (the fatigue lives are multiplied by the shift factors) to suit failure and so-called critical points in the field respectively.

Meanwhile, for rutting, Yusof (2005) and Rahman (2004) among a host of other researchers suggested that by limiting the maximum vertical strain at the top of the sub-grade layer, a failure criterion can be established. In this regard, Brown and Brunton (1986) suggested that:

$$N_{cr} = fr\left(\frac{7.6\ x\ 10^{8}}{\varepsilon_z^{3.7}}\right) \qquad 7.1$$

$$N_F = fr\left(\frac{3\ x\ 10^{9}}{\varepsilon_z^{3.57}}\right) \qquad 7.2$$

Where:

- N_{cr} = life to critical condition (10mm rut)
- N_F = life to failure (20mm rut)
- Fr = a rut factor to account for the hot mixture type- 1.56 for dense bitumen mixtures
- ε_z = Strain on top of subgrade obtained from structural analysis

However, Thom (1996) cautioned that of these criteria, subgrade strain is less fundamentally based than asphalt tensile strain for fatigue cracking.

These shift factors for fatigue and rutting were derived for hot mixtures, however they have been used in this work since there are no such values for cold mixtures presently.

7.3 Modelling Tools and Requirements

For the structural design approach chosen in this work i.e. the mechanistic-empirical approach, Brown and Brunton (1986) summarised the design process as follows:

- Specify the loading (vehicular loading),
- Estimate the components (layers) and sizes (thicknesses),
- Consider the materials available,

- Conduct structural analysis using theoretical principles,
- Compare critical stresses, strains or deflections with allowable values (based on the result of laboratory tests) to ascertain whether the design is satisfactory,
- Make adjustments to materials or geometry until satisfactory design is achieved,
- Consider the economic feasibility of the result.

Figure 7.1 shows the hypothetical flexible road pavement structure that was used for the exercise described in this chapter though the main focus was on the road base (CBEM) layer. This structure is commonly used in Nigeria and for ease of analysis, a single wheel loading system of 40kN tyre load (80kN being the single standard axle load), 700kPa tyre pressure and tyre contact area radius of 143mm was used for simple symmetry of loading involved (Ebels, 2008). The single wheel loading makes the critical horizontal position to lie directly under the wheel load. More importantly, Twagira (2010) observed that the use of a single wheel also leads to a slightly more conservative estimate, because the stresses under single wheel loading are generally slightly higher than under dual wheel loading.

The results obtained under resilient modulus testing in the preceding chapter indicated that the CBEMs are stress dependent and thus are non-linear in elastic response. 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. Although some researchers (Jitareekul 2009; and Twagira, 2010) have found linear analytical tools useful for quick analytical design of stresses and strains in such pavements, Ebels (2008) however in a comparative analysis using both linear (BISAR 3.0) and non-linear (KENLAYER) analytical tools on similar materials (cold mixtures) found that it is better to use the appropriate approach for each material. He reported that using non-linear elastic approach and dividing the cold mix layer into sub-layers, each with a unique stiffness, results in higher stress ratios which are better estimates than those obtained by linear elastic calculations. This observation clearly corroborates the suggestions of Huang (2004) on same issue. Therefore the non-linear elastic approach was adopted in this exercise.

7.4 Non-Linear Elastic Analysis

The KENLAYER mode in the KENPAVE computer program developed by Huang (2004) for flexible pavement design which is widely available was found useful in this regard. Huang (2004) reported that the backbone of KENLAYER is the

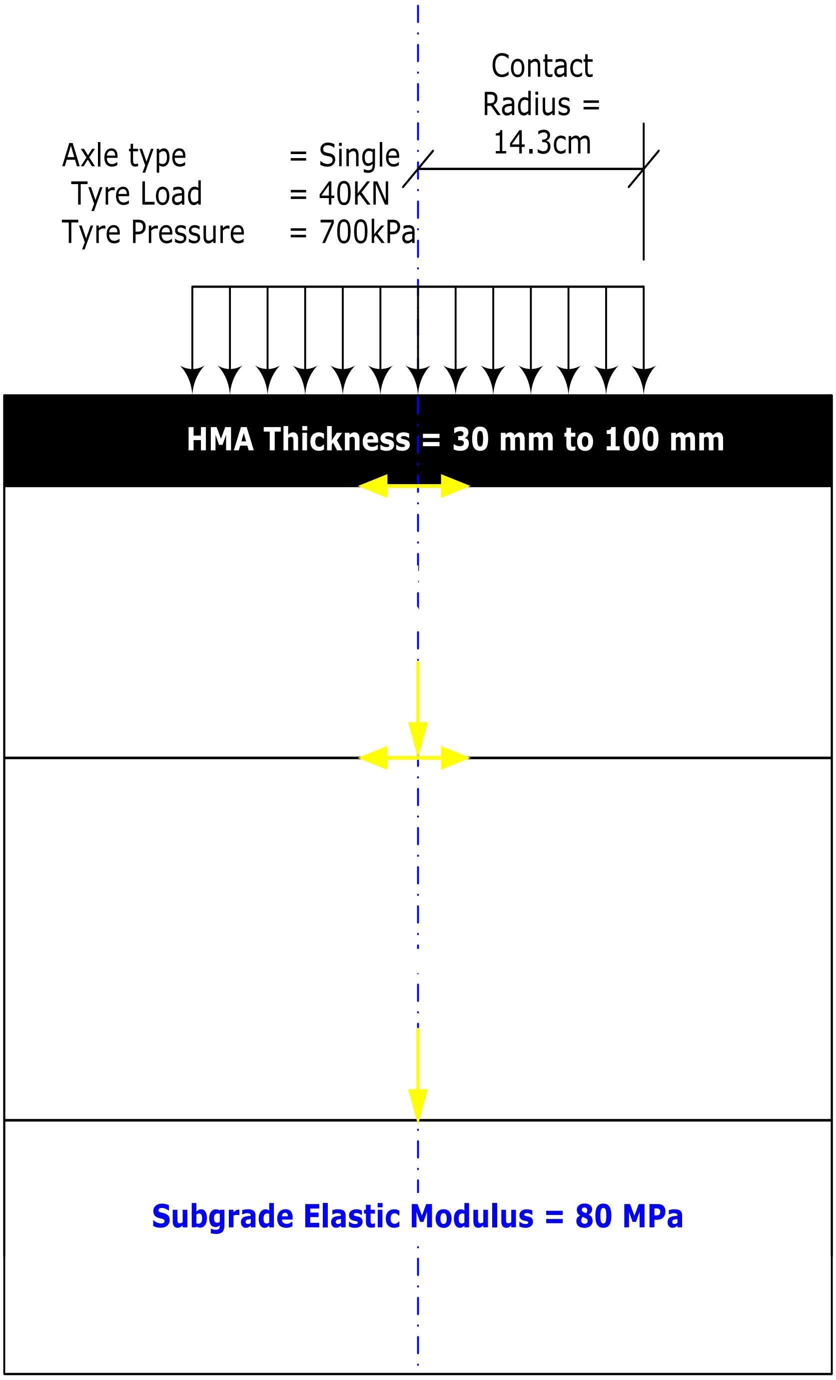


Figure 7.1: Hypothetical Road Pavement Structure Containing CBEMs used for the Modelling

solution 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. Therefore, KENLAYER is deemed capable of analysing layered systems under single, dual, dual-tandem or dual-tridem wheels with each layer behaving differently, linear elastic, non-linear elastic or viscoelastic.

For non-linear elastic analysis in KENLAYER, the K-θ model described in the preceding chapter applies. Huang (2004) stated that 3 methods have been incorporated into KENLAYER for non-linear analysis. While Method 1, the most accurate but time consuming (Huang, 2004) and in which the stress dependent layer is subdivided, was used in this work, in light of the suggestions of Ebels (2008) for similar materials, the other 2 methods and detailed description of the capabilities of the program are documented by Huang (2004).

The K_1, and K_2 values obtained from the resilient modulus test reported in Chapter 6 which are required for this exercise are detailed in Table 7.2. The other relevant required properties of the pavement materials are similarly detailed in Table 7.3. The 'k' i.e. K_1, K_2, K_3, K_4 values are material constants (Huang, 2004). Ekwulo and Eme (2009) noted that K_1 and K_2 are dependent on moisture content while K_3 and K_4 are related to the soil types, either coarse grained or fine grained. While all layers are assumed to be fully bonded, the following medium condition parameters (see Figure 7.2) suggested by Huang (2004) were also used in the analysis for the subgrade and subbase layers:

- Coefficient of Earth Pressure = K_0 = 0.8
- Subbase K_1 = 30.912
- Subgrade K_1 = 52.992 (medium condition)
- K_2= 42.8
- K_3 = 1110
- K_4 = 178
- Emin – Emax = 30000 – 100000 (minimum and maximum stiffness modulus for subgrade)

These values were used because materials for the layers in question were not studied in this work. Meanwhile, for the surfacing layer on top of the CBEM, HMA Stiffness Modulus of 2500MPa was used. This was adopted and made constant for all cases considered since from initial trials conducted though not reported in this thesis for brevity, changes in stiffness modulus of such layers do not have significant effect on the strain and stress distribution in the underlying layers although the results indicated that as stiffness modulus of the surfacing layer increases, fatigue life increases.

Table 7.2: K_1, K_2 and R^2 Values from Resilient Modulus Test of CBEMs

CBEM Type	FC @ 20°C			FC 30°C		
	K_1 (MPa)	K_2	R^2	K_1 (MPa)	K_2	R^2
VA	26.467	0.5453	0.9885	50.026	0.436	0.9878
5dmm	50.426	0.4673	0.9771	78.226	0.378	0.9827
10dmm	32.603	0.5248	0.9895	59.325	0.4416	0.9838
20dmm	35.731	0.5319	0.9821	48.567	0.4641	0.992

FC: Fully Cured

Table 7.3: Pavement Layer Material Properties

Layer Material	Density (kg/m^3)	Poisson's Ratio	Elastic Behaviour
HMA	2400	0.35	Linear
VA	2180	0.35	Non-Linear
5dmm	2120	0.35	Non-Linear
10dmm	2158	0.35	Non-Linear
20dmm	2224	0.35	Non-Linear
Sub base	2000	0.35	Non-Linear
Subgrade	1800	0.35	Non-Linear

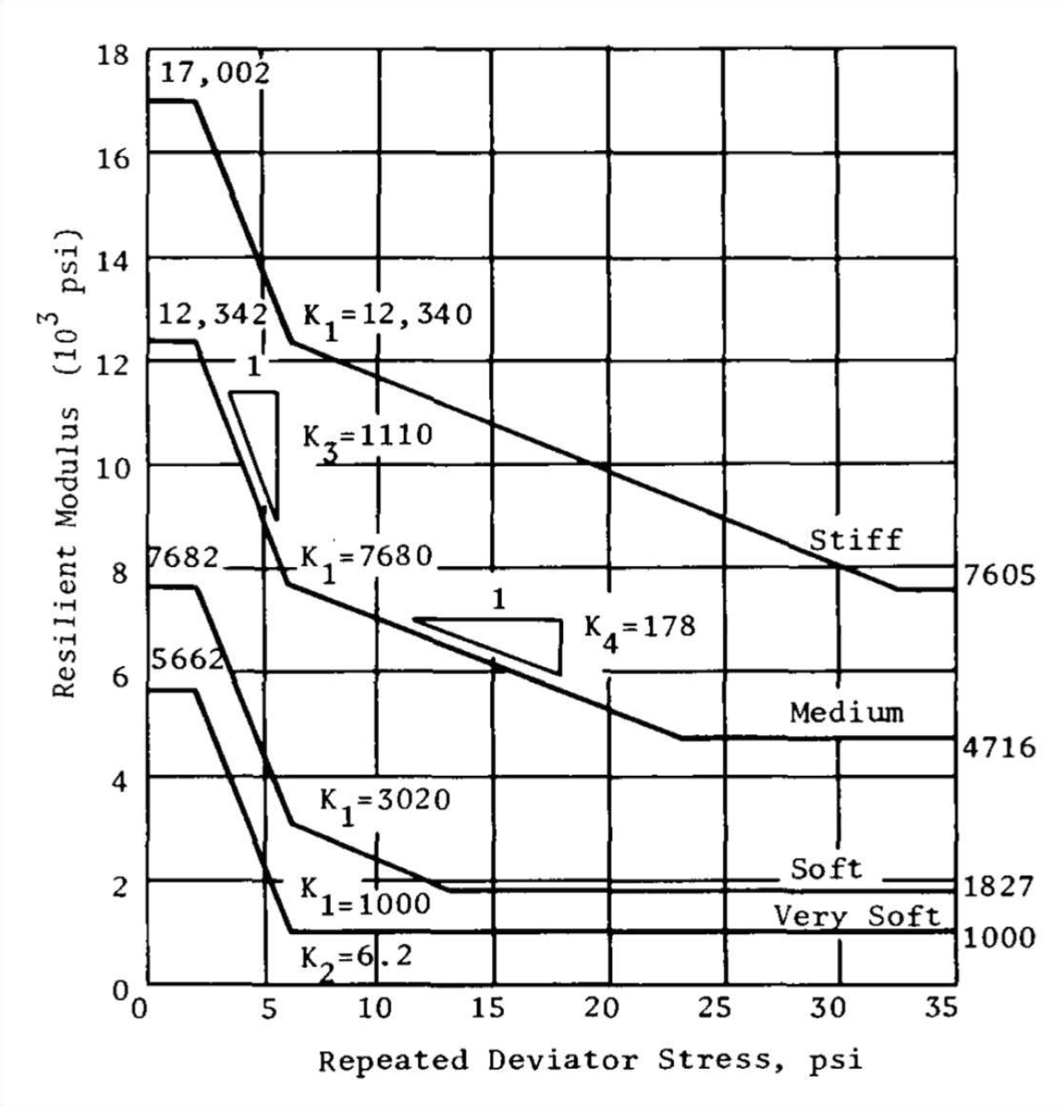


Figure 7.2: Resilient Modulus-Deviator stress Relationship for Four Types of Subgrade (1 psi = 6.9 kPa) (Thompson and Elliot in Huang, 1994, pg 109)

Seeding resilient modulus (compressive modulus) values for CBEMs (non-linear and stress dependent layer) ranged between 1000MPa and 600MPa depending on the number of sub-layers (each sub-layer of the CBEM is 50mm thick). This seeding value is needed because an iterative procedure is used, in which the moduli of non-linear layers are adjusted as the stresses vary, while the moduli of the linear layers remain the same. During each iteration, a constant set of moduli is computed based on the stresses obtained from the previous iteration. After the stresses due to single or multiple wheels are determined, the elastic moduli of non-linear layers are recalculated and a new set of stresses is determined. This is repeated until the moduli converge to a specified tolerance (Huang, 2004). The Sub-base Modulus was taken as 300MPa (stabilised laterite of 30% CBR) and 80MPa (lateritic soil of 8% CBR) was adopted for the Subgrade Modulus since these were observed for such layers for a typical Nigerian road by Ekwulo and Eme (2009). The other required information such as loads and layer thicknesses are as stated in Figure 7.1 and Table 7.1.

It is worth noting that all required entries must be made correctly as the KENLAYER program is highly sensitive to inputs. The outputs from the analysis are the vertical, horizontal and shear strains and stresses. Also included is resilient modulus of each layer and vertical displacements.

It is also worth noting that although this multi-layer analysis is easy to execute (Oliveira, 2006), researchers have found out that performance prediction from such an approach is not as accurate as those obtained from finite element analysis (Oliveira, 2006; Huang, 2004; Ebels, 2008; and Twagira, 2010). However, Oliveira (2006) and Twagira (2010) both observed that finite element analysis is difficult to implement unless users have a thorough knowledge of basic principles.

7.5 Stress and Strain Distribution in Pavements Containing CBEMs

Using pavement structure case number 9 from Table 7.1 i.e. surfacing layer thickness of 30mm, CBEM (road base) layer thickness of 250mm (divided into 5 equal sub-layers) and a sub-base layer of 200mm, the stress and strain distributions through pavements containing VACBEM, 5dmmCBEM, 10dmmCBEM and 20dmmCBEM as road base layers which were analysed using the KENLAYER analytical tool were studied. All the CBEMs used were in a fully cured condition but tested at 20ºC and 30ºC.

Figures 7.3 to 7.6 are relevant here. Figure 7.3 shows the vertical stress distribution in the pavements. The results of the analyses indicate that vertical stress distribution in the CBEMs is not significantly different for all the pavement types considered. The figure shows also that vertical stress reduces with depth. These were also observed for the horizontal stress distribution as indicated in Figure 7.4 though with more pronounced differences indicated in the area covered by the red dash line circle. It is to be noted that the differences here occurred in the CBEM layer only since all other layers have similar properties for all the pavements considered. The figure also shows that the horizontal stress gradually changed with depth from a relatively high negative value at the top of the surfacing layer to a positive value on top of the subgrade. Figure 7.5 shows the vertical strain distribution in the pavements while Figure 7.6 shows similar distribution for the horizontal strain in the pavements. The red dash lines in the figures show the bottom level of the CBEM layers in the hypothetical pavement structure. The figures indicate significant differences in strains for the CBEM layers and that the maximum horizontal and vertical strains are not at the bottom of the CBEM layers, but very close to the middle of the CBEM layers. These observations are consistent with those observed by Ebels (2008) for similar materials. He observed that cold mix materials with different properties showed significant differences in strains while stress values had little differences using the KENLAYER analytical tool. The figures showed that the RAP CBEMs generally have lower strain values compared to the VACBEM.

Meanwhile, as observed from the relevant figures for strains, VACBEM indicated the highest values (worst performance) for the two temperatures while the 20dmmCBEM tested at 20ºC indicated the lowest strain values. Materials tested at 30ºC generally showed higher strain values compared to those tested at 20ºC.

7.6 The Effect of Analytical Method on Stress and Strain Distributions

The effect of the applied analytical tool on the stress and strain distribution in the pavement was briefly examined. This was done by comparing the stress and strain distributions generated from the KENLAYER analytical tool with those generated using the Bitumen Stress Analysis in Roads (BISAR 3.0) software which was developed by Shell (1998).

Since in the preceding section, the analyses showed that the vertical stress distributions for all the pavements considered were close, the corresponding modulus values (obtained iteratively) to the stress values observed at the bottom

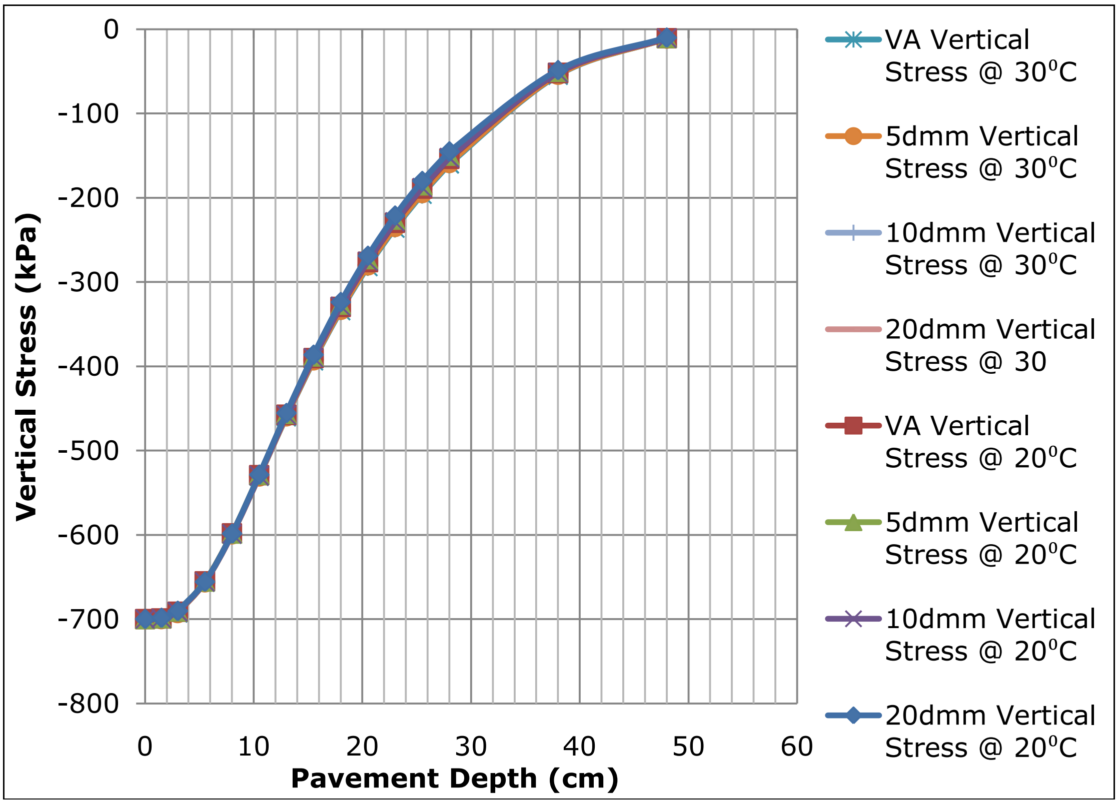


Figure 7.3: Vertical Stress Distribution through Pavement Structure Case Number 9 using KENLAYER

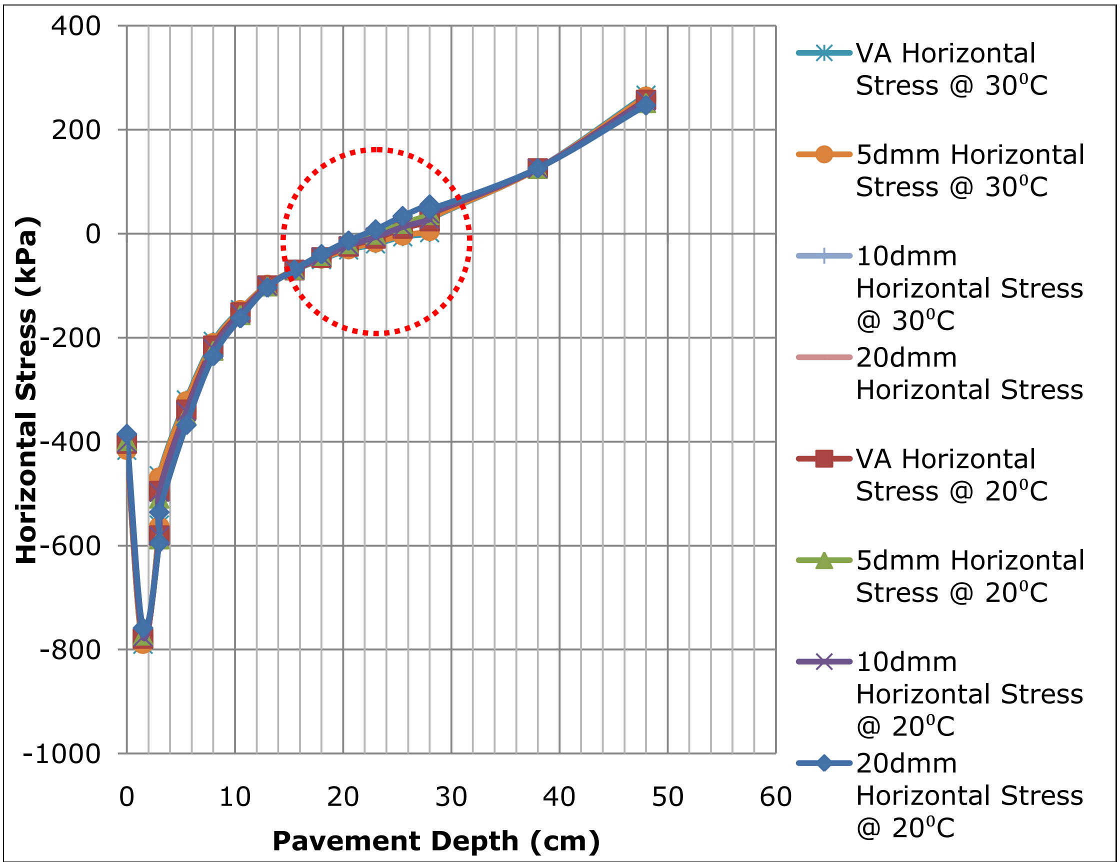


Figure 7.4: Horizontal Stress Distribution through Pavement Structure Case Number 9 using KENLAYER

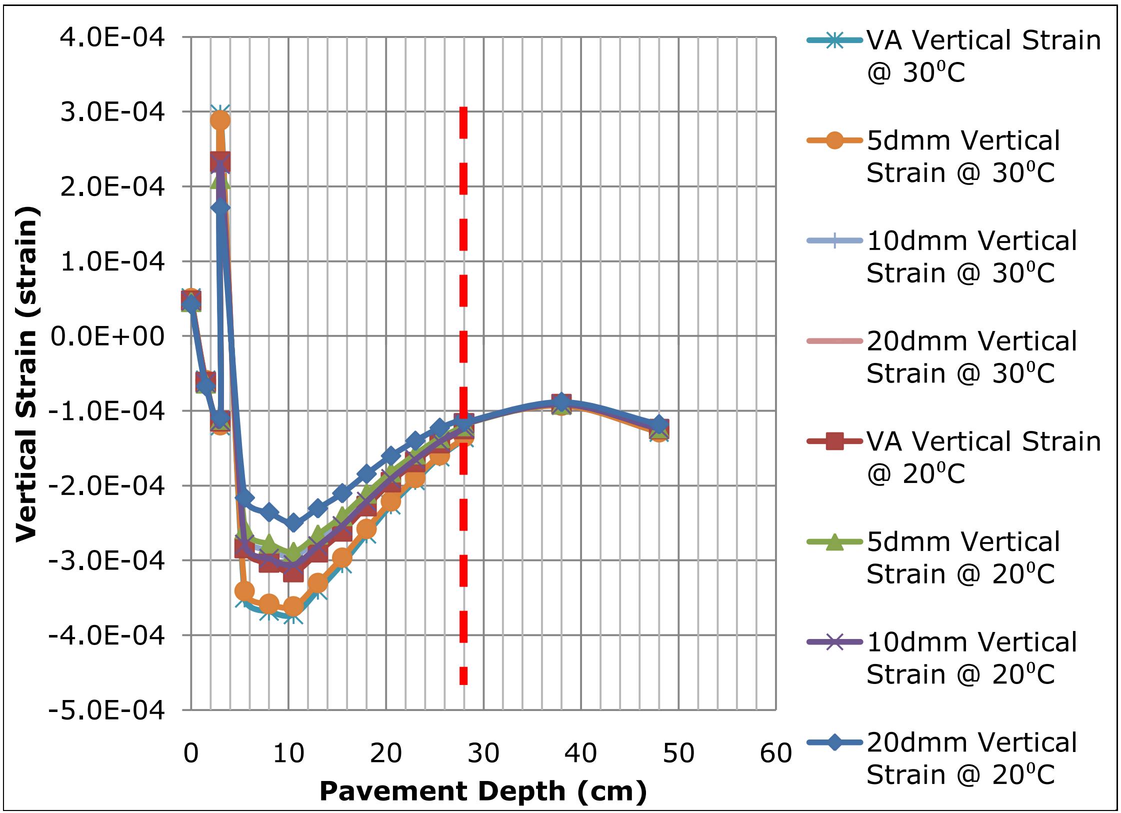


Figure 7.5: Vertical Strain Distribution through Pavement Structure Case Number 9 using KENLAYER

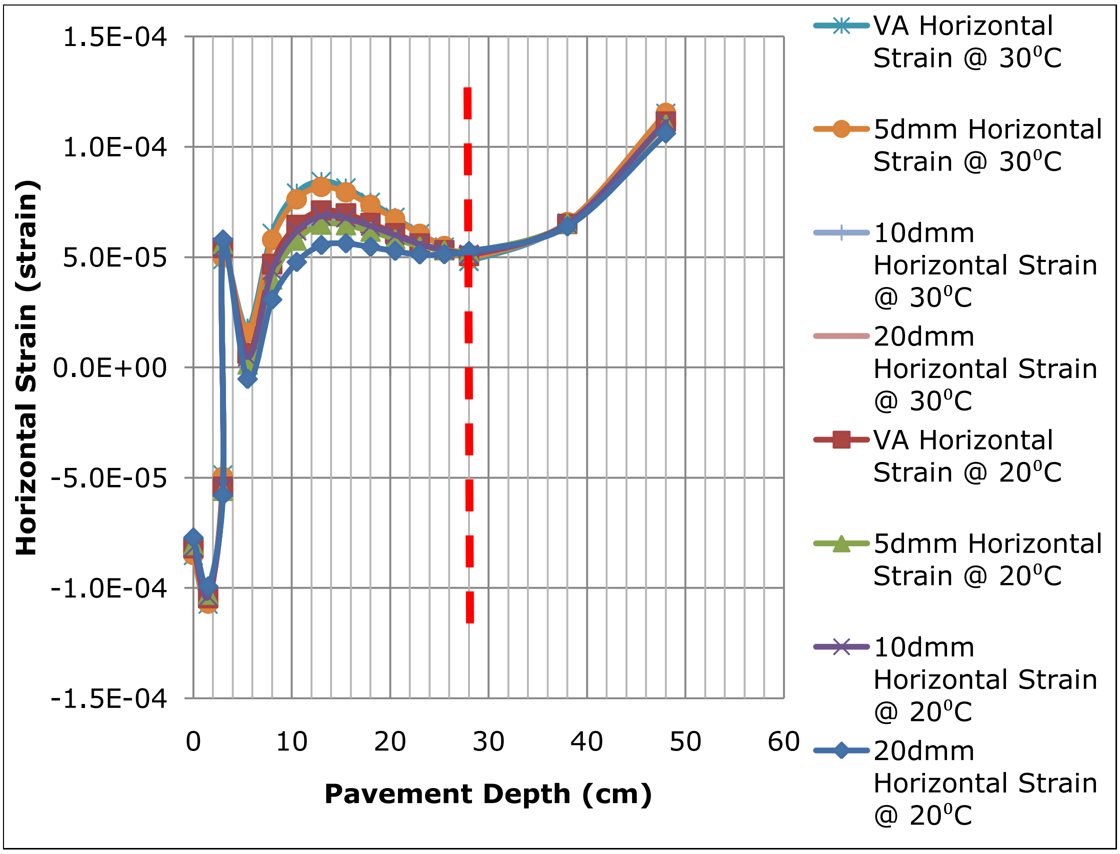


Figure 7.6: Horizontal Strain Distribution through Pavement Structure Case Number 9 using KENLAYER

of each of the CBEM (road base) layers were used in BISAR 3.0 (linear elastic analysis). For this case, the pavement structure number 6 i.e. surfacing layer thickness of 50mm, road base layer (CBEM) of 200mm (sub divided into 4 equal layers) and for the sub-base layer, a thickness of 200mm was used. For brevity, only the materials tested at 20ºC were considered.

Figure 7.7 to 7.10 are relevant here. Figure 7.7 shows the vertical stress distribution for pavements containing the CBEMs using both linear and non-linear analytical tools i.e. BISAR 3.0 and KENLAYER. The figure shows that there is a little difference in the trend of stress distributions recorded by BISAR 3.0 and KENLAYER although the two results indicated no significant differences between pavement types considered within each of the two groups i.e. BISAR 3.0 analysis results and KENLAYER analysis results respectively. Similarly, the results of the KENLAYER analysis in the referred figure when compared with those in Figure 7.3 indicated that change in thicknesses of the surfacing and road base (CBEM) layers did not introduce any significant difference in stress distributions in the grouped pavements.

A not so similar trend was observed in Figure 7.8 which details the results of the horizontal stress distributions for the BISAR 3.0 and KENLAYER analyses with a significant difference in the stress distribution in the surfacing layer (HMA). The BISAR 3.0 results indicated very high compressive stresses (about 2000kPa) on top of the HMA layer and similarly high tensile stresses (about 1000kPa) at the bottom of the same layer. The KENLAYER recorded for this same pavement little compressive stresses i.e. about 400kPa and 500kPa on top and at the bottom of the HMA respectively. These obviously are responsible for the trends observed for the strains in Figures 7.9 and 7.10 with BISAR 3.0 returning large strain values.

Although as expected, the stress distributions converged at the bottom of the CBEM layers since it was the vertical stresses at such points that were used to calculate the corresponding modulus values used in the BISAR 3.0 analysis, the observed trends clearly show that BISAR 3.0 must have overestimated the vertical and horizontal strains required for design. Such high strain values will affect the sizing of the pavement layers which might be practically uneconomical. The modulus values used in the BISAR 3.0 analyses for the CBEM layers were generally low (about 300MPa) and this strongly affected the response of the surfacing (returning very large strain values).

Since modulus values were calculated for each of the sub-layers of the CBEMs in KENLAYER, another set of analyses was conducted in BISAR 3.0 for the same

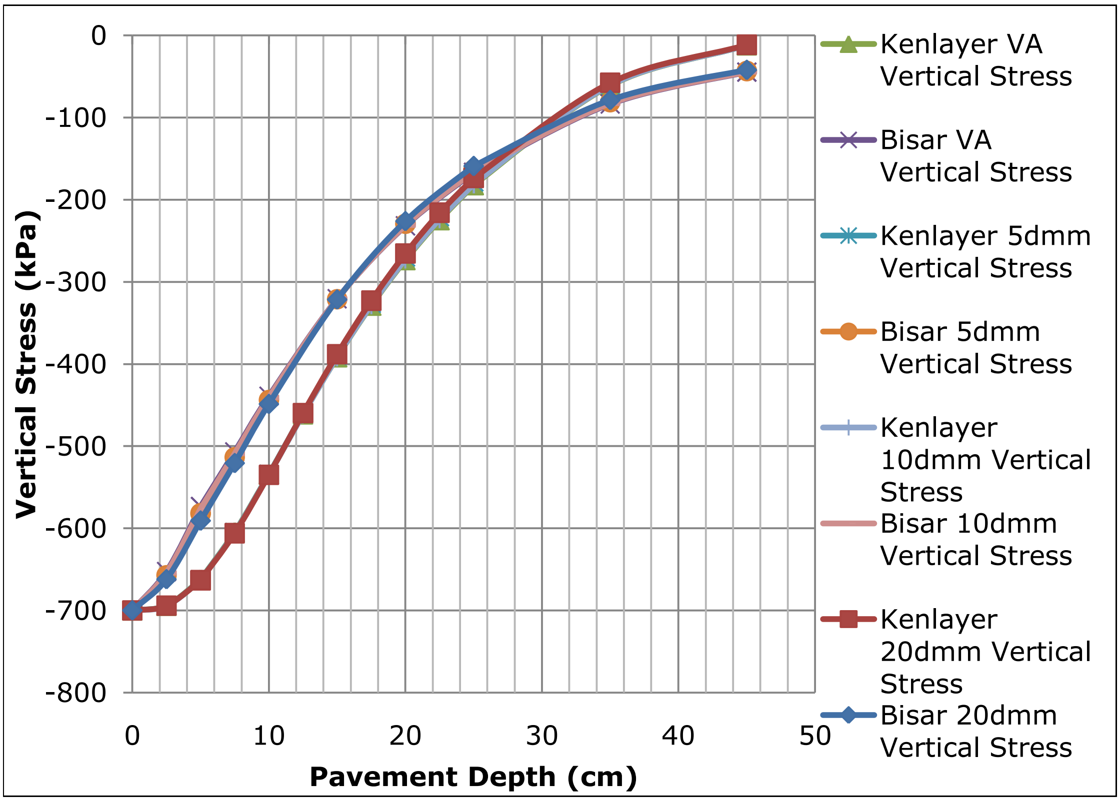


Figure 7.7: Vertical Stress Distribution through Pavement Structure Case Number 6 using BISAR 3.0 and KENLAYER

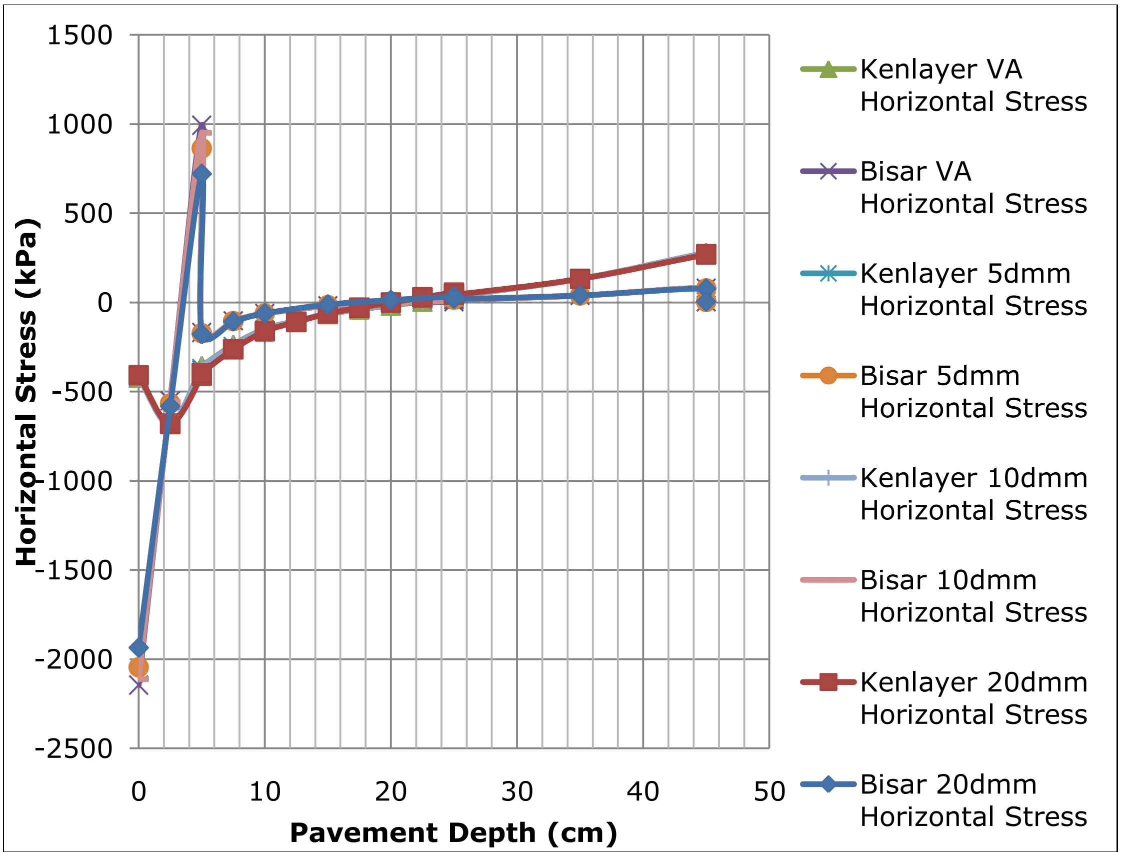


Figure 7.8: Horizontal Stress Distribution through Pavement Structure Case Number 6 using BISAR 3.0 and KENLAYER

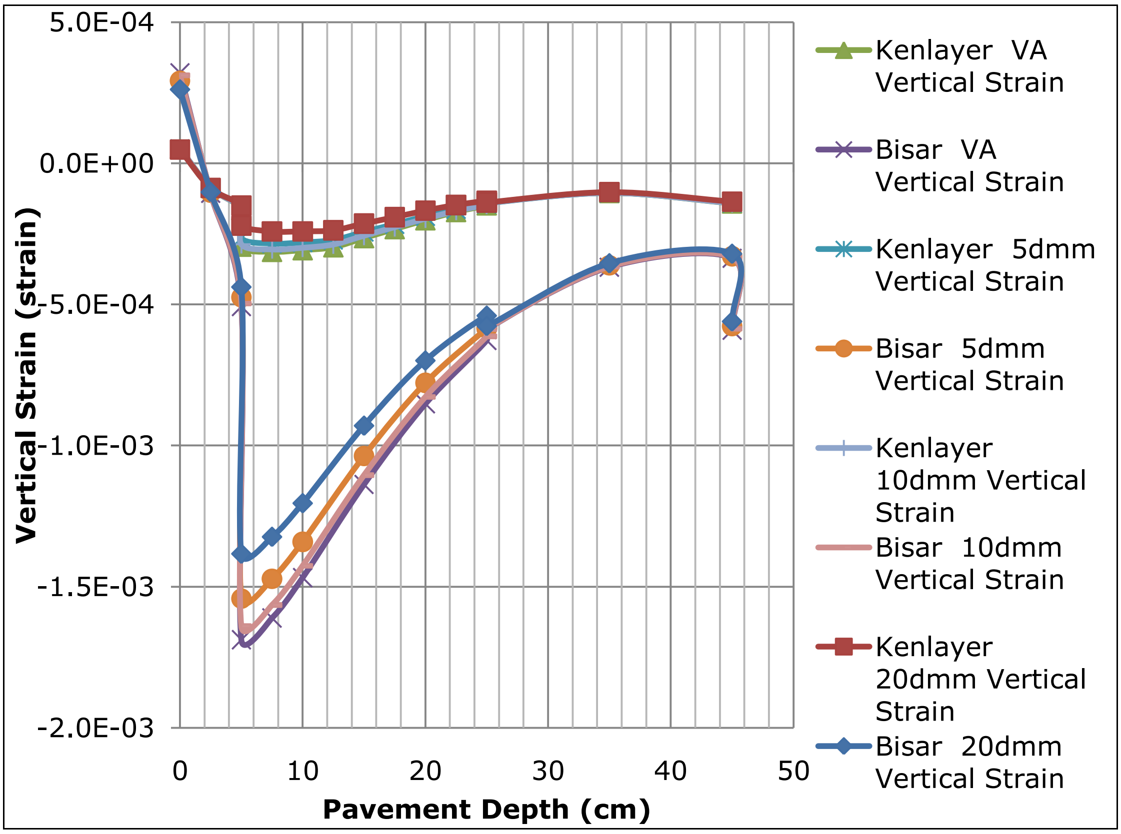


Figure 7.9: Vertical Strain Distribution through Pavement Structure Case Number 6 using BISAR 3.0 and KENLAYER

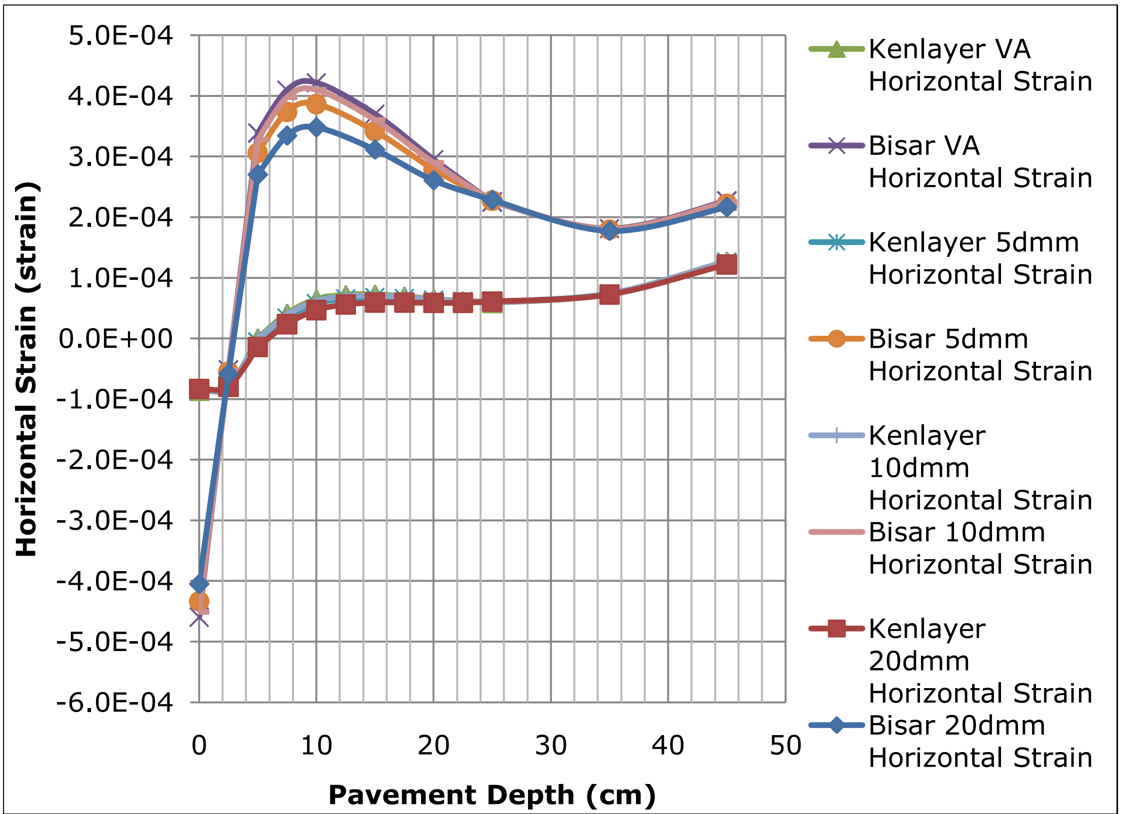


Figure 7.10: Horizontal Strain Distribution through Pavement Structure Case Number 6 using BISAR 3.0 and KENLAYER

pavement now using the averages of the calculated modulus values for each of the CBEM layers from KENLAYER. Table 7.4 details the modulus values. It was observed from the table that modulus reduces from top to bottom which reflects the stress dependency of the CBEMs. Also since the materials were stress dependent, the stress values at the bottom of the CBEMs which were used for the first set of analyses on the BISAR 3.0 were not large enough to represent the average characteristic properties of the CBEMs.

Figures 7.11 to 7.14 indicate that when the correct modulus values of the sub-layers of the CBEMs are known, BISAR 3.0 can also be used to determine the stress and strain distributions through the non-linear layers just as the KENLAYER does. This observation is consistent with the findings of Ebels (2008). The sub-layers are now treated as linear layers in this regard. However it is of no use using BISAR 3.0 to estimate the stress and strain distributions in such pavements containing non-linear layers since estimating the required modulus values needed for accurate estimates of strains and stresses in BISAR 3.0 is first done using KENLAYER. It is better to execute such designs involving non-linear layers with KENLAYER. BISAR 3.0 therefore might not be suitable for analysing pavements containing such non-linear materials.

7.7 Design of CBEM Road Base for Fatigue Life

The results of the analyses done in the preceding sections using KENLAYER showed that the maximum horizontal strains in the CBEM layers are somewhere close to the middle, and thus, such strains are deemed more appropriate theoretically for design purposes than those observed at the bottom of the CBEM (road base) layers recommended by Ebels (2008). The design presented here considered these set of strains i.e. maximum and at the bottom, to ascertain their suitability or not.

Where applicable, only laboratory test results (fatigue line equations reported in Chapter 6) conducted at 30ºC were used since the focus of the work is hot tropical climate. A conventional HMA road base layer of 28mm DBM (Read, 1996) though analysed using BISAR 3.0 was included for comparisons. A stiffness value of 3000MPa was assumed for the layer (at 30ºC). The design was conducted for both critical condition (point at which pavement crack initiation occurs) and failure. The shift factors of 77 and 440 were used for the respective points as suggested by Brunton (1983). It is worth noting however that such shift factors might not be appropriate for the CBEMs (cold mixtures) since they were developed for HMA. At present, no such universally accepted values are available for cold mixtures.

Table 7.4: Calculated Modulus Values for each CBEM Sub-Layer in KENLAYER (@ 20°C)

Sub-layer No.	Modulus Values from KENLAYER Analyses			
	VA CBEM	5dmmCBEM	10dmmCBEM	20dmmCBEM
1	1400	1519	1431	1746
2	1308	1421	1350	1614
3	1296	1399	1335	1561
4	1324	1415	1457	1596
Average Modulus	1332	1439	1368	1634

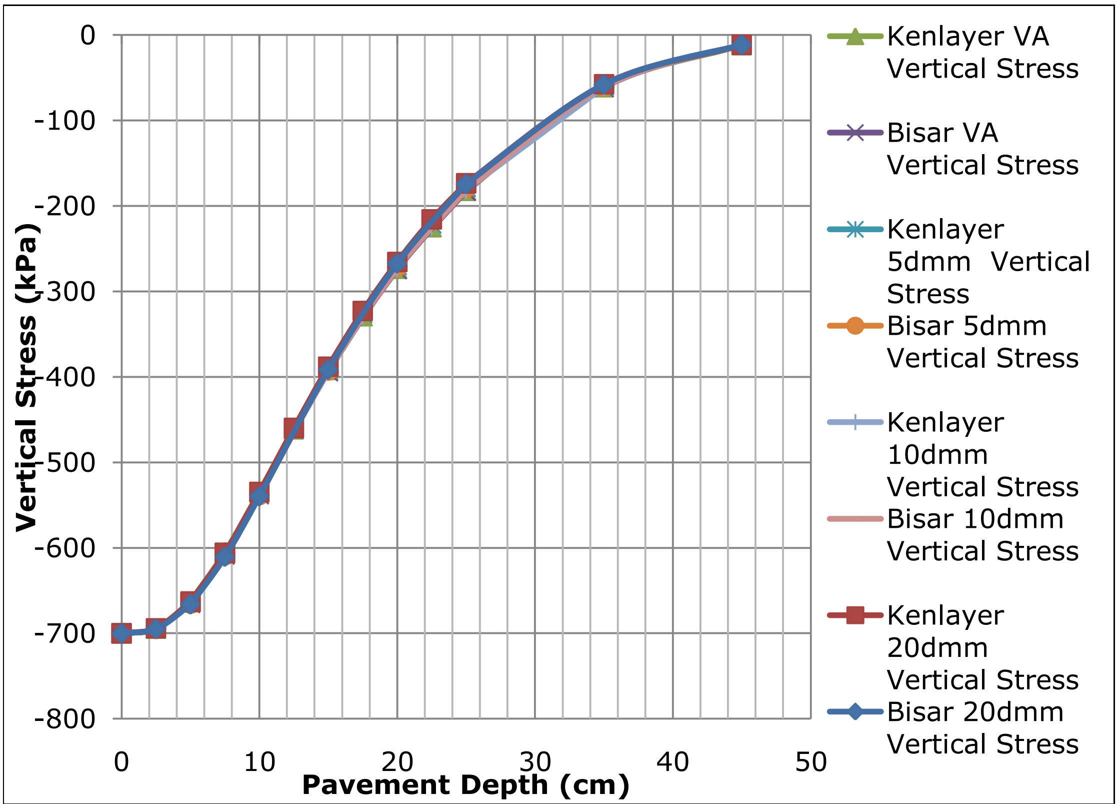


Figure 7.11: Vertical Stress Distribution through Pavement Structure Case Number 6 using BISAR 3.0 (with calculated modulus) & KENLAYER (@ 20°C)

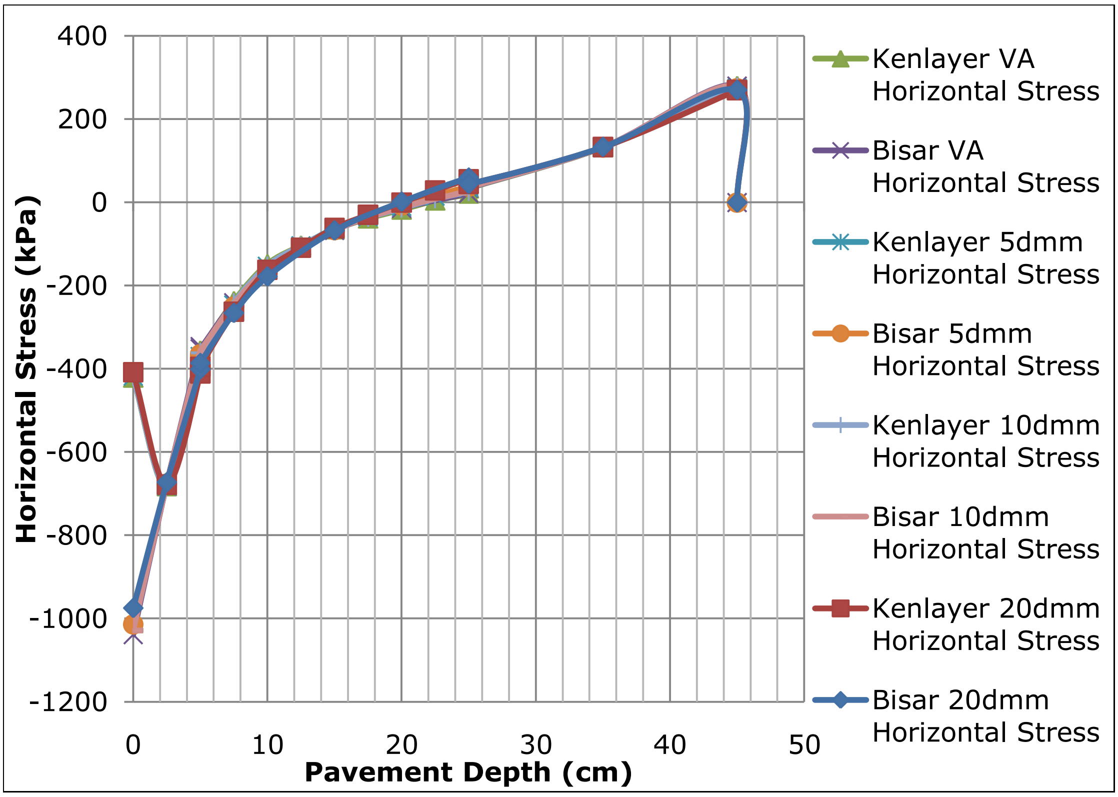


Figure 7.12: Horizontal Stress Distribution through Pavement Structure Case Number 6 using BISAR 3.0 (with calculated modulus) & KENLAYER (@ 20°C)

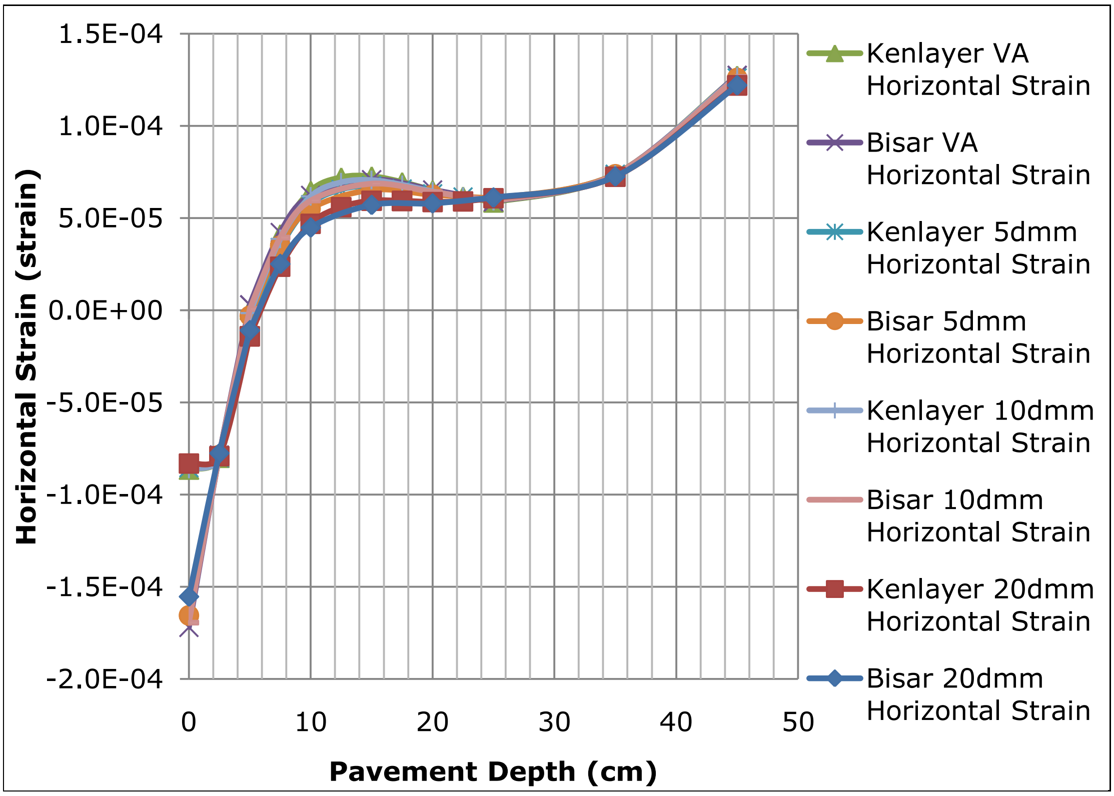


Figure 7.13: Horizontal Strain Distribution through Pavement Structure Case Number 6 using BISAR 3.0 (with calculated modulus) & KENLAYER (@ 20°C)

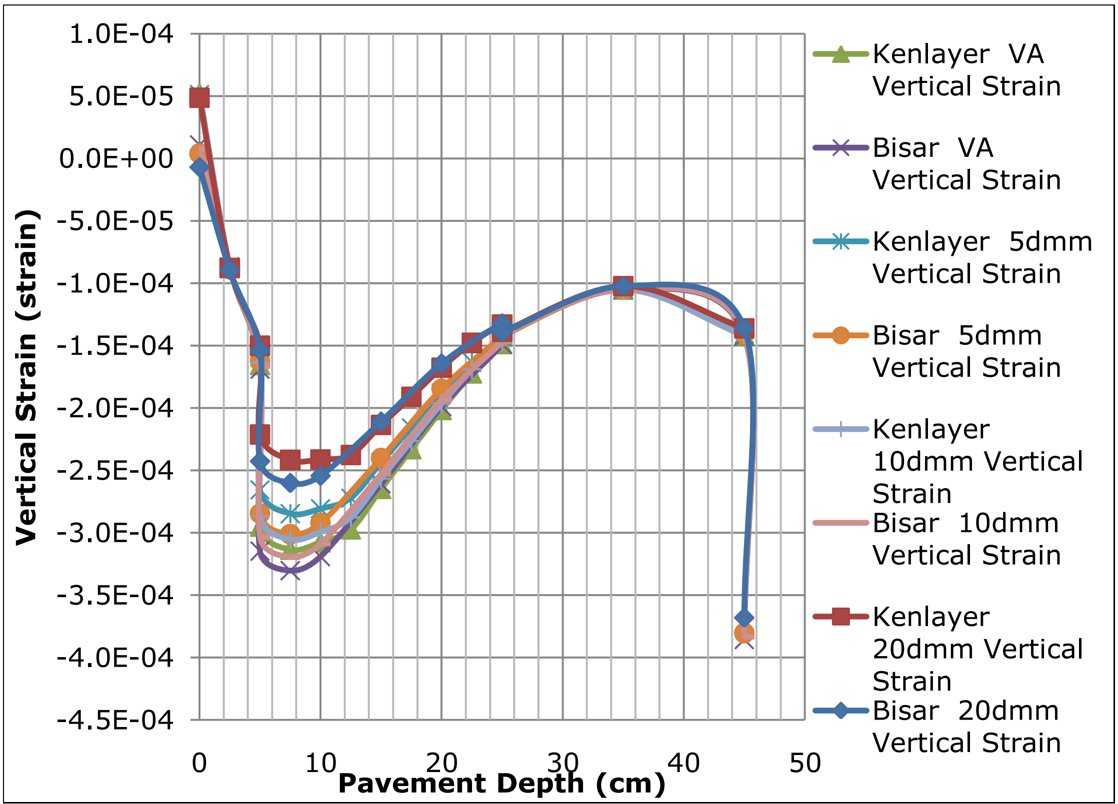


Figure 7.14: Vertical Strain Distribution through Pavement Structure Case Number 6 using BISAR 3.0 (with calculated modulus) and KENLAYER (@ 20°C)

While the 440 value for life to failure might be too high, 77 may be too low to describe total pavement life. Considering the fact that cold mixtures are not as good as HMA as observed in this thesis, it is believed that the appropriate value for life to failure should be somewhere in-between these two values. Thus, the results presented here are just estimates and can only be used with caution. Figures 7.15a to 7.17 detail design charts based on horizontal strains at the bottom of the CBEM layers and are the results for equivalent standard axle loads (ESALs) to critical condition. Figures 7.18 to 7.21 show design charts based on maximum horizontal strains in the CBEM layers and are similarly the results for ESAL to critical condition. The surfacing layer thicknesses used were 30mm, 50mm, 75mm and 100mm. The road base layer thicknesses used were 150mm, 200mm, 250mm and 300mm.

For cases where horizontal strains at the bottom of the CBEM layers were used, the results clearly indicate that as CBEM layer thickness increases, fatigue lives (to critical or failure point) increase. This was similarly observed for the HMA 28mm DBM layer although with better responses in life to thickness increases. The results also indicate that changes in the surfacing layer thickness do not cause significant improvement to the fatigue lives of the CBEM road base layers compared to the HMA. Of all the CBEM materials considered for the road base layer, the 5dmmCBEM had the best performance while the VACBEM had the worst performance. Reasons behind the performance of the 5dmmCBEM here have been fully discussed in Chapter 6. All the RAP CBEMs and the HMA 28mm DBM had fatigue lives clearly above 1 million ESALS applications for failure and 0.1 million for critical condition irrespective of the layer thickness.

Except for ranking the CBEMs in the same order as observed for horizontal strains at the bottom of the CBEM layers, the results for using maximum horizontal strains in the CBEM layers as indicated in Figures 7.18 to 7.21 show that changes in surfacing and CBEM layers' thicknesses were of no effect on the fatigue lives of the CBEM layers. This indeed must be responsible for the earlier mentioned recommendation of Ebels (2008) that strains at the bottom be used for such design purposes since increase in thicknesses of the CBEM layers as earlier observed in Figures 7.15a to 7.17 caused an increase in their fatigue lives. Although not reported in this thesis, similar observations were noted for the rutting lives of the CBEMs using maximum vertical strains in the CBEM layers. However, rather than using the horizontal strains at the bottom of the CBEM layer for fatigue design lives which are lower than the maximum which is not common in practice, it is not advisable to use the fatigue and rutting lives of the

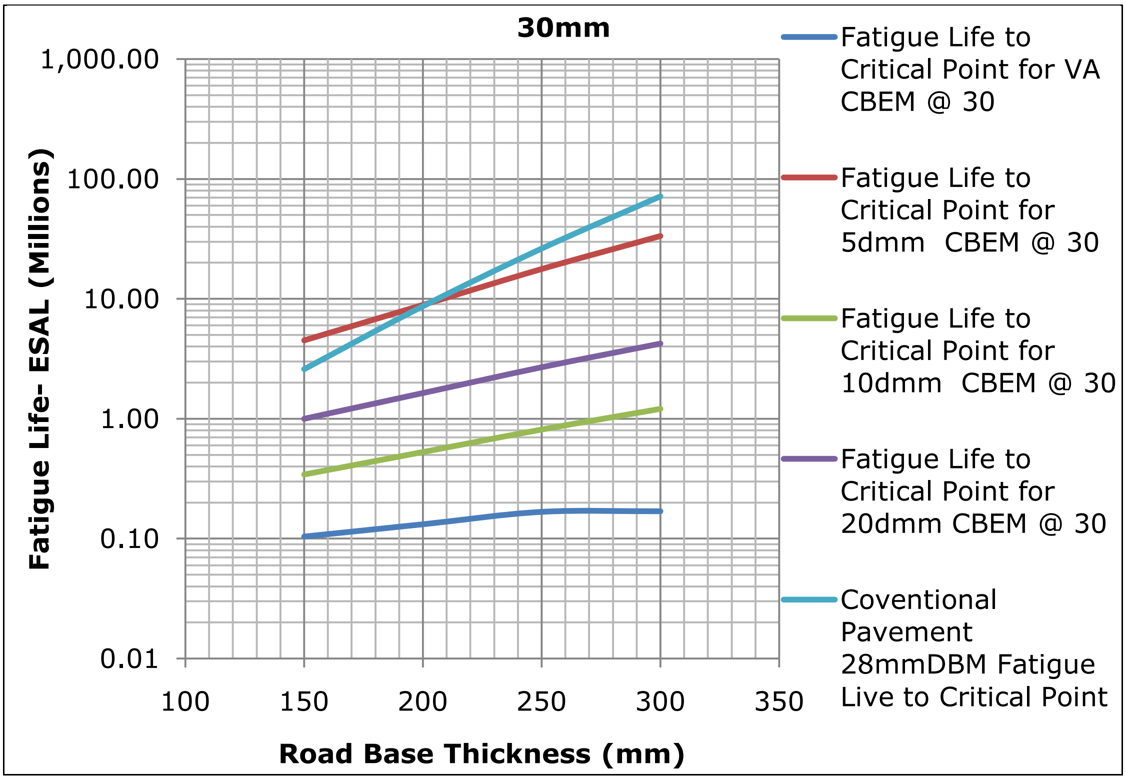


Figure 7.15a: Designed Road Base Layer Thicknesses Using Horizontal Strains at the Bottom of CBEM Layers to Determine Corresponding Fatigue Lives to Critical Condition for Pavements with Surfacing Thickness of 30mm HMA

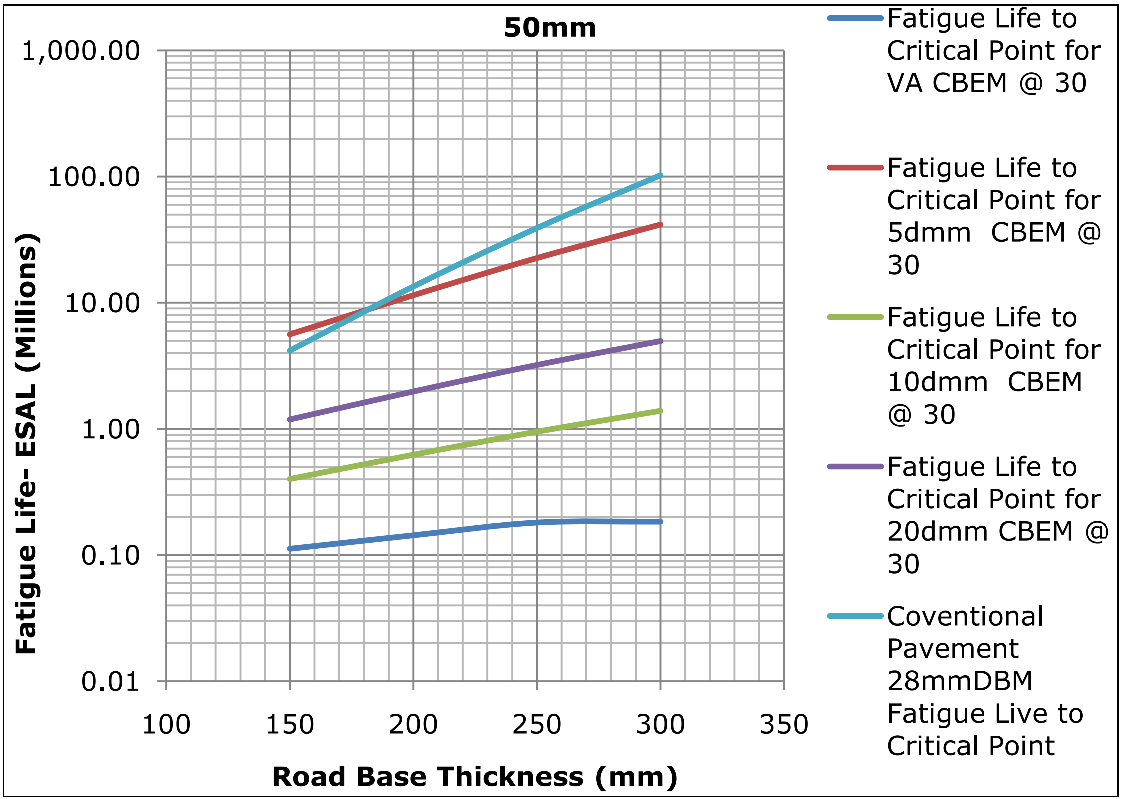


Figure 7.15b: Designed Road Base Layer Thicknesses Using Horizontal Strains at the Bottom of CBEM Layers to Determine Corresponding Fatigue Lives to Critical Condition for Pavements with Surfacing Thickness of 50mm HMA

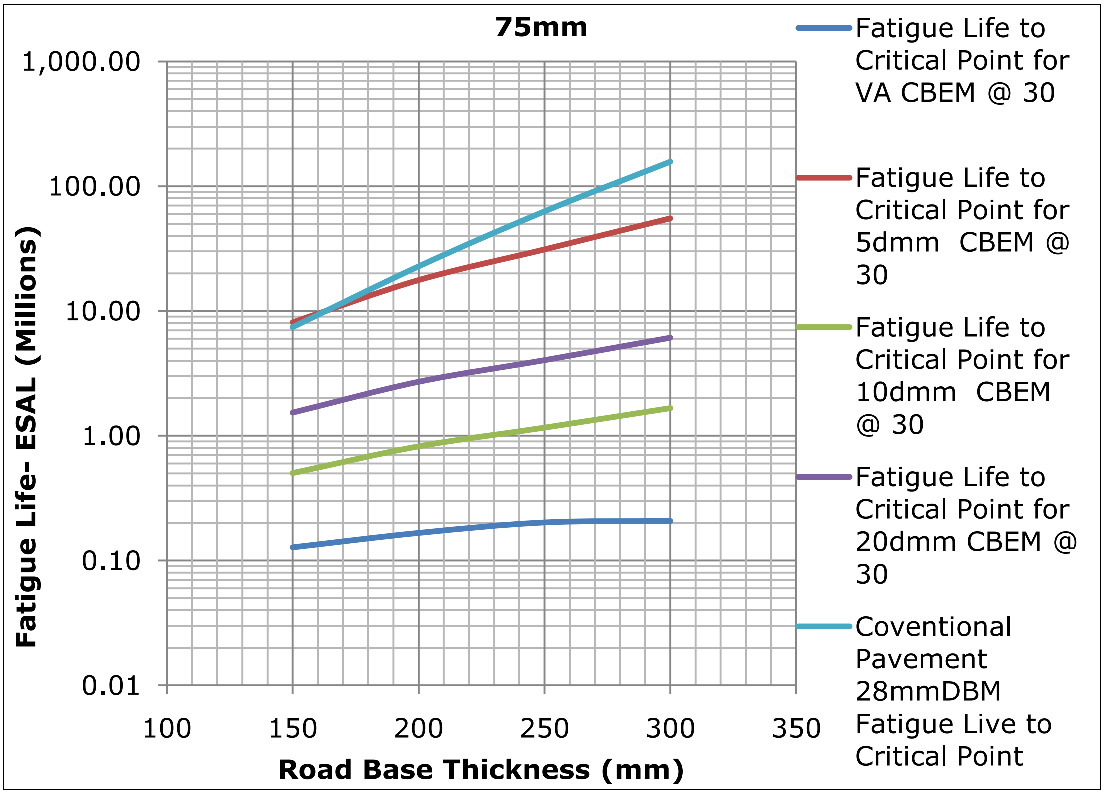


Figure 7.16: Designed Road Base Layer Thicknesses Using Horizontal Strains at the Bottom of CBEM Layers to Determine Corresponding Fatigue Lives to Critical Condition for Pavements with Surfacing Thickness of 75mm HMA

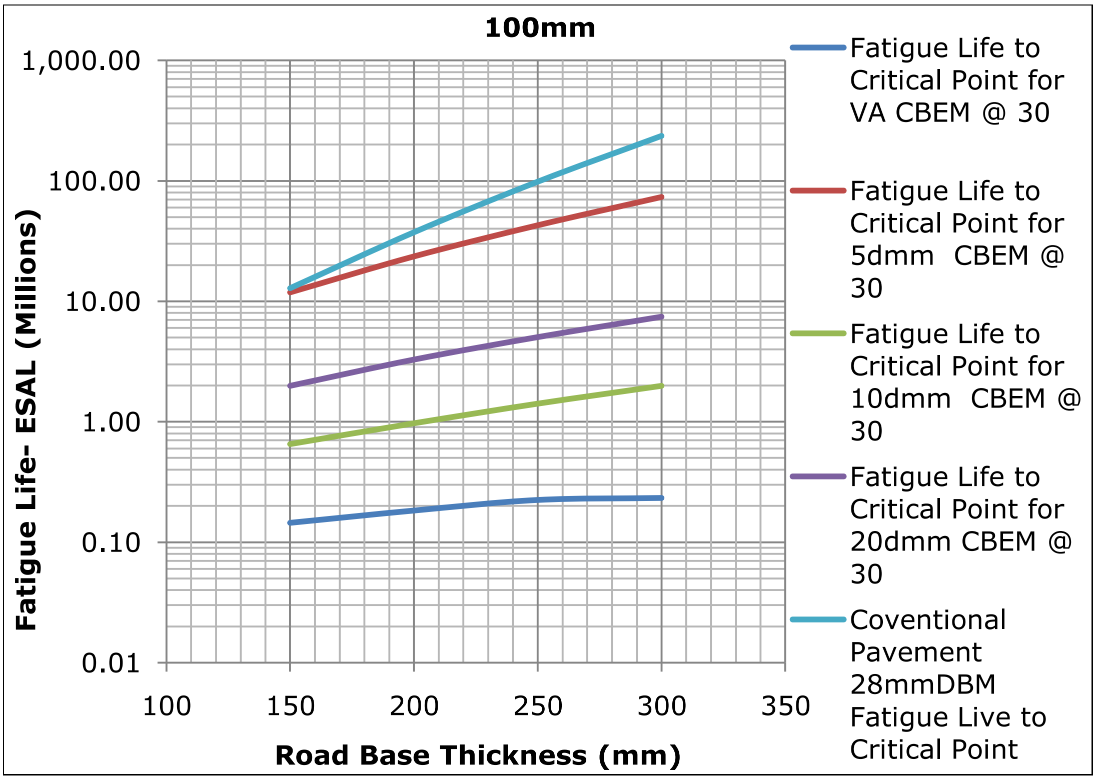


Figure 7.17: Designed Road Base Layer Thicknesses Using Horizontal Strains at the Bottom of CBEM Layers to Determine Corresponding Fatigue Lives to Critical Condition for Pavements with Surfacing Thickness of 100mm HMA

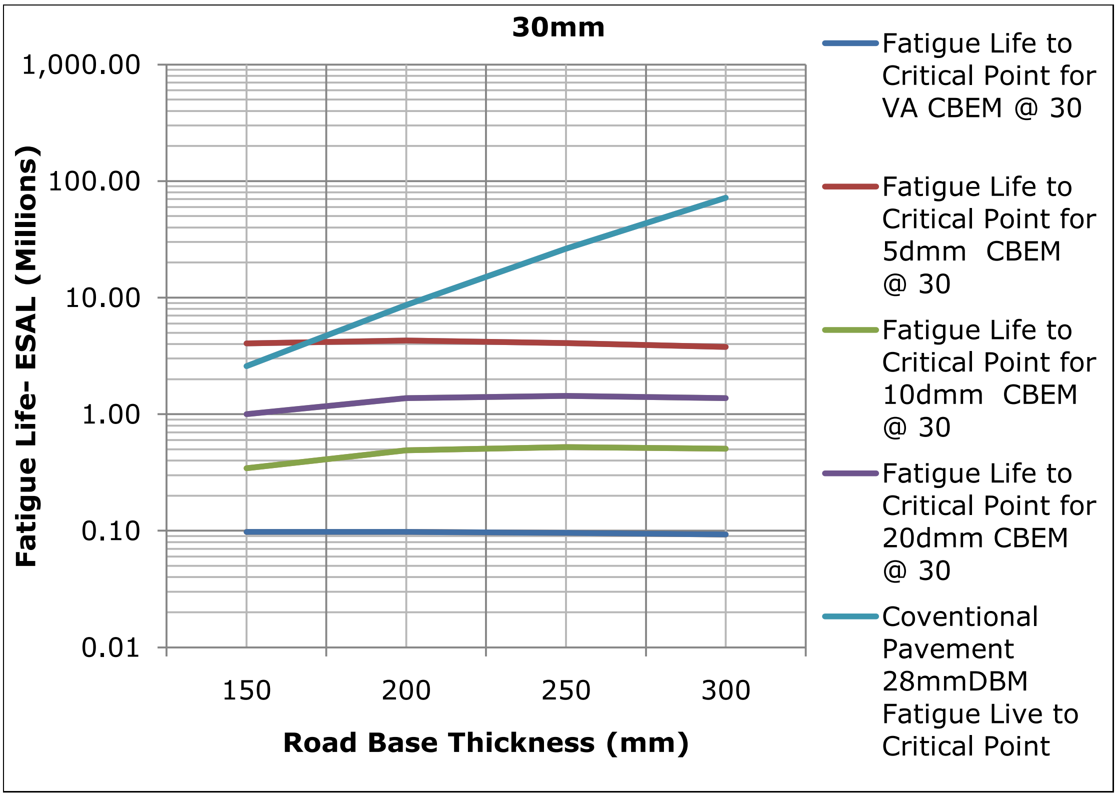


Figure 7.18: Designed Road Base Layer Thicknesses Using Maximum Horizontal Strains in the CBEM Layers to Determine Corresponding Fatigue Lives to Critical Condition for Pavements with Surfacing Thickness of 30mm HMA

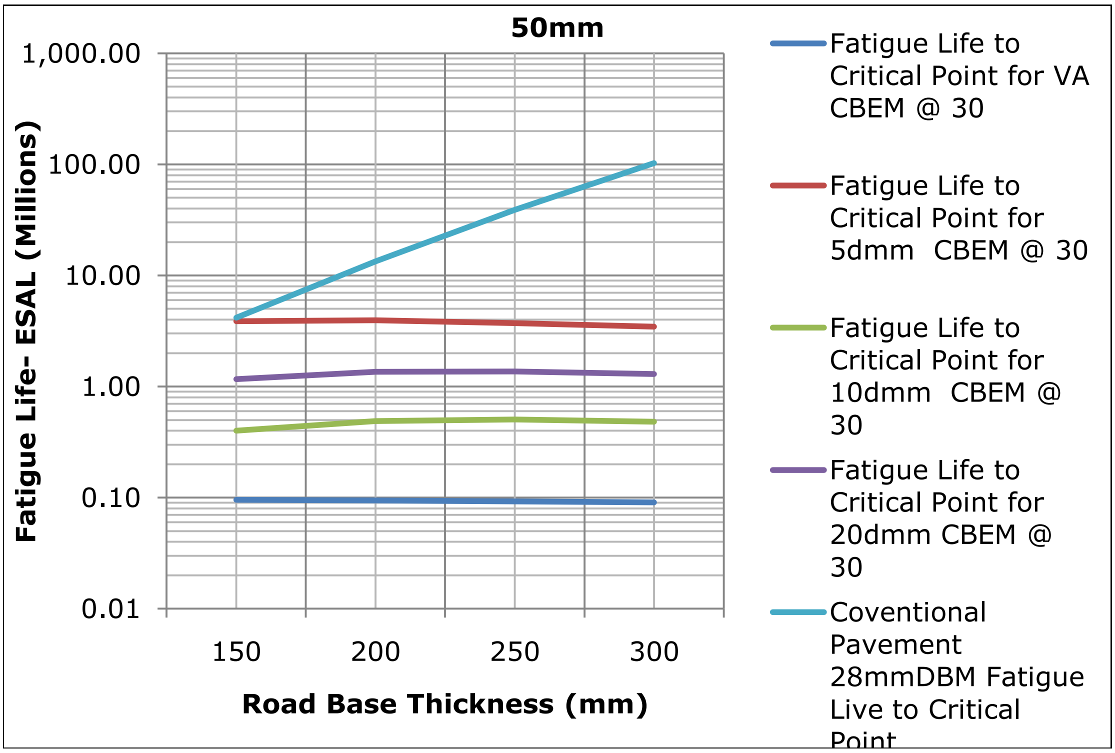


Figure 7.19: Designed Road Base Layer Thicknesses Using Maximum Horizontal Strains in the CBEM Layers to Determine Corresponding Fatigue Lives to Critical Condition for Pavements with Surfacing Thickness of 50mm HMA

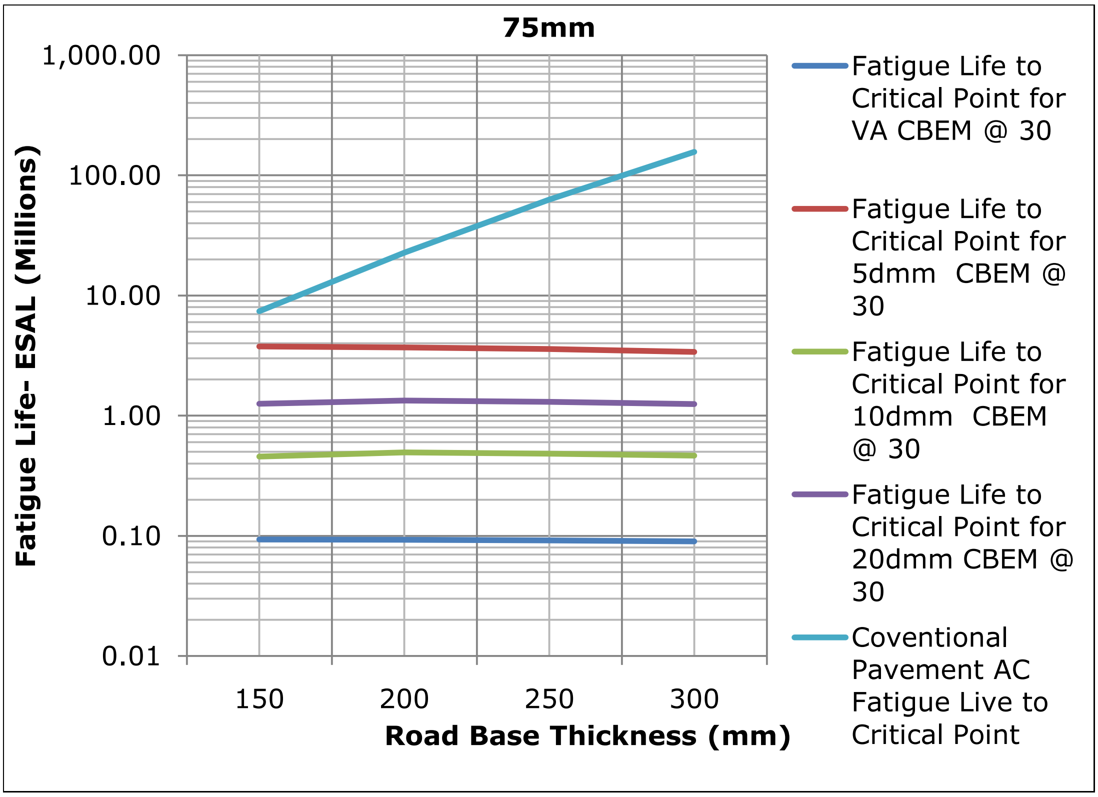


Figure 7.20: Designed Road Base Layer Thicknesses Using Maximum Horizontal Strains in the CBEM Layers to Determine Corresponding Fatigue Lives to Critical Condition for Pavements with Surfacing Thickness of 75mm HMA

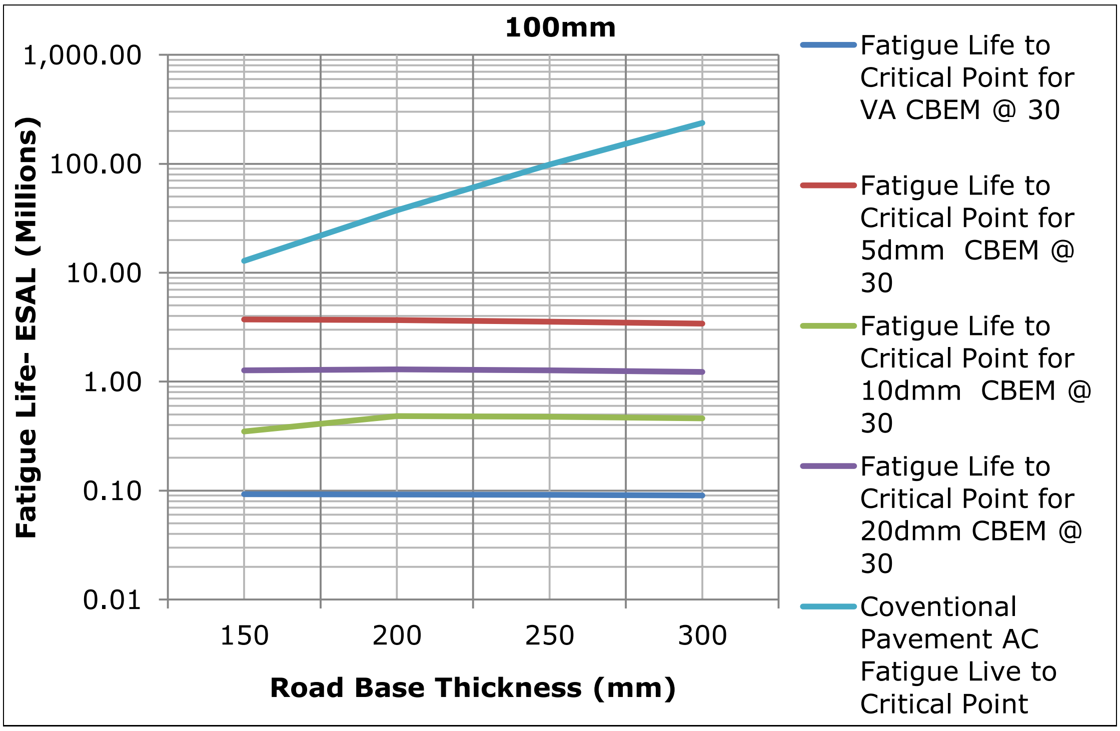


Figure 7.21: Designed Road Base Layer Thicknesses Using Maximum Horizontal Strains in the CBEM Layers to Determine Corresponding Fatigue Lives to Critical Condition for Pavements with Surfacing Thickness of 100mm HMA

CBEM layers as criteria for design. It is however possible that the representative strains required for considering CBEM layers in structural design of pavements can be ascertained through full scale pavement tests and this will be an interesting area for further research work.

7.8 The Effect of CBEM Road Base Layer Property on the Fatigue Life of Surfacing Layer

The fatigue lives of the overlying surfacing HMA layers were studied to ascertain the effect of the property of the road base (CBEM) on such. The HMA used here was the 30/14 HRA studied by Read (1996). In the initial analyses conducted using the horizontal strains at the bottom of the HMA layers from the KENLAYER results, all the strain values for HMA layers of 30mm and some of 50mm were negative (compressive) values while the others were positive (tensile). Although the observed compressive strain value(s) is an indication of good performance for the HMA surfacing layer considering the criterion used i.e. horizontal strain at the bottom of the HMA surfacing layer, in real life, it is impossible for a pavement to last forever, which the results imply. The results also suggest that failure in the pavement due to the compressive strains may be predominantly governed by top down cracking.

Therefore another analysis was conducted using OLCRACK developed by Thom (2000; and 2008). This analytical tool takes into account both the top-down and bottom-up crack propagations in the surfacing HMA layer for the estimation of standard axle loads to failure. The method of analysis in OLCRACK assumes that the layer is linear in elastic response. Since the modulus of the road base is required for such, the calculated average values for such layers done using KENLAYER were again found useful here. These were 1167MPa, 1193MPa, 1428MPa and 1390MPa for the VACBEM, 5dmmCBEM, 10dmmCBEM and 20dmmCBEM respectively at 30^{o}C. The stiffness modulus of the HMA road base used was 3000MPa.

Figures 7.22 to 7.24 detail the results of the analysis conducted using OLCRACK. For brevity, the results for the 30mm HMA surfacing layer were not included since they showed a negative trend (between fatigue life and pavement thickness) for pavements containing CBEMs. The result suggest that the 30mm HMA surfacing is either too thin for overlay in pavements containing CBEM layers as road base or that there is a need to improve on the properties of the underlying layers i.e. sub-base and subgrade in order to obtain realistic results. Asphalt Academy (2009)

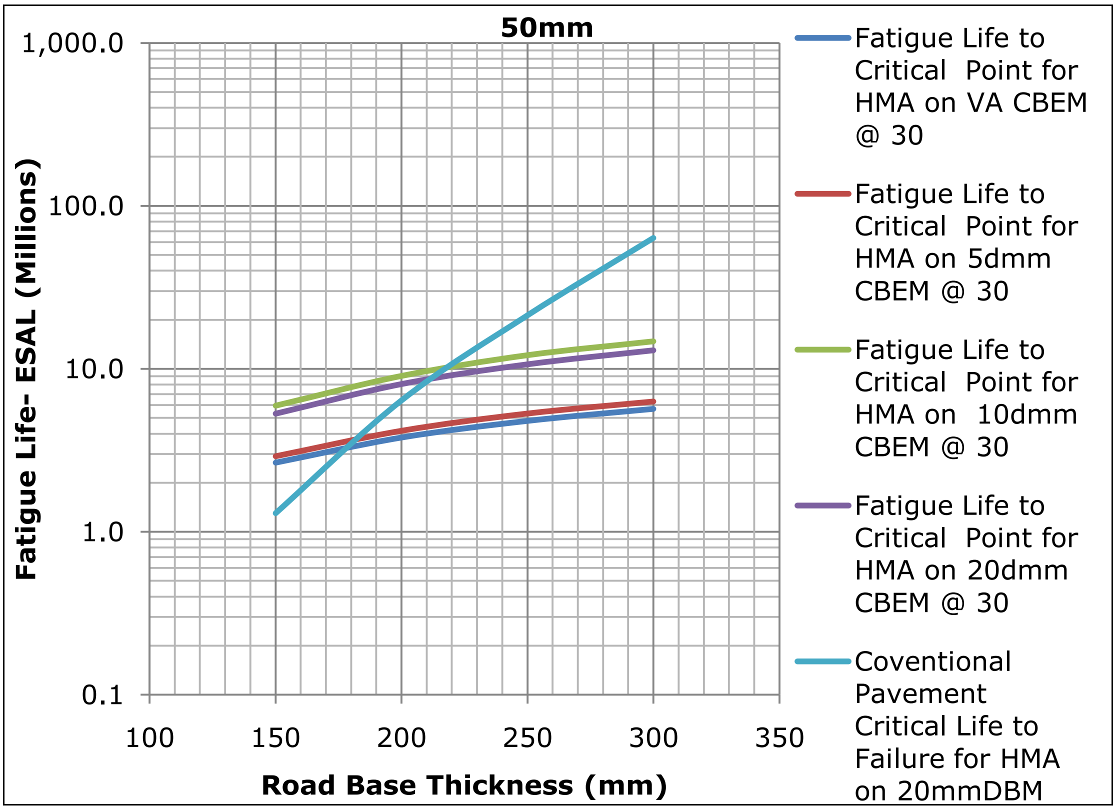


Figure 7.22: Effect of Road Base Layer Thickness and Material Type on Fatigue Lives to Critical Condition for Surfacing Thickness of 50mm HMA

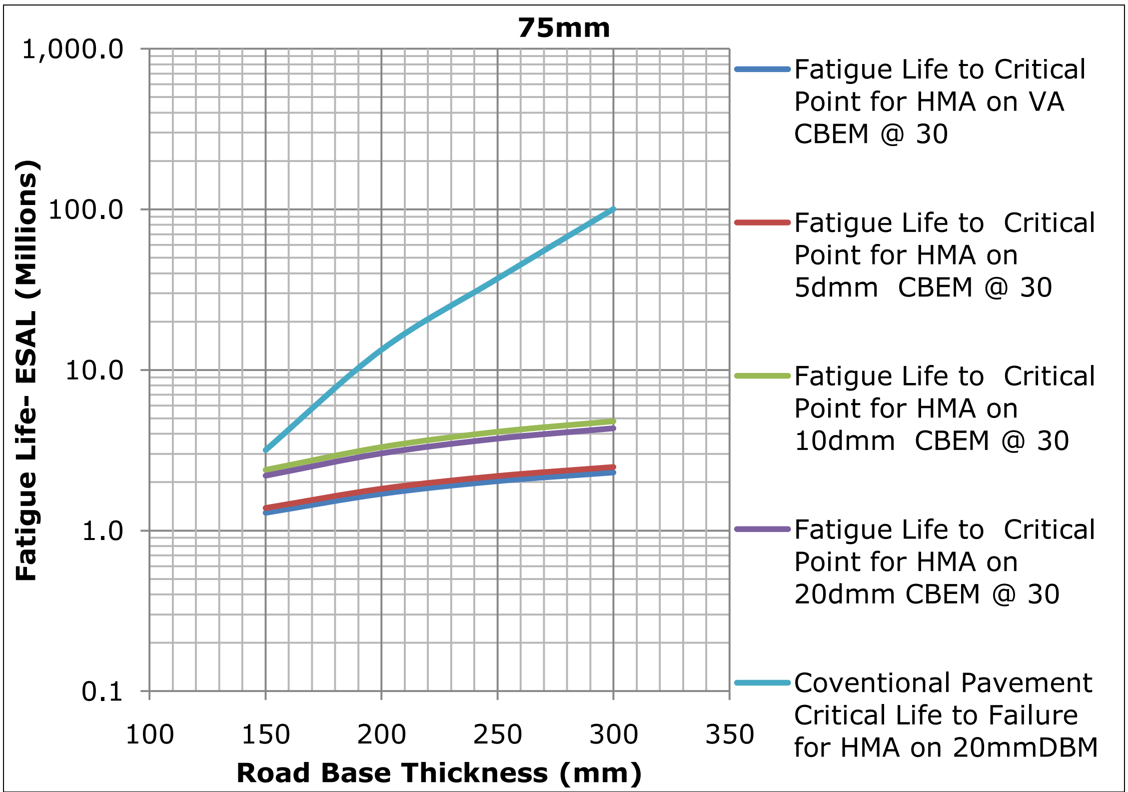


Figure 7.23: Effect of Road Base Layer Thickness and Material Type on Fatigue Lives to Critical Condition for Surfacing Thickness of 75mm HMA

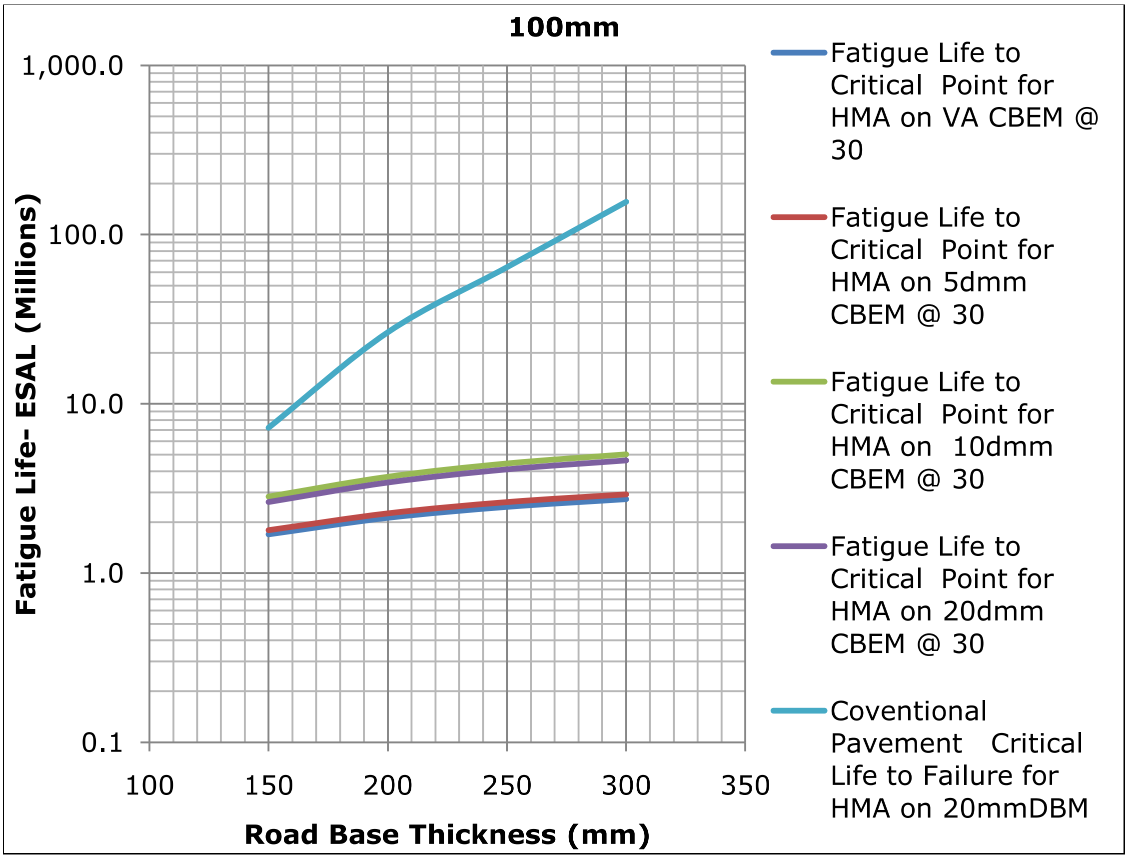


Figure 7.24: Effect of Road Base Layer Thickness and Material Type on Fatigue Lives to Critical Condition for Surfacing Thickness of 30mm HMA

suggests that thicknesses as low as 20mm HMA surfacings are achievable in pavements containing cold asphalt. It is worth noting that the trend for the conventional pavement containing HMA (28mm DBM) as road base had a positive trend for the 30mm HMA surfacing. The trends observed for the other cases i.e. 50mm, 75mm and 100mm surfacing layer thicknesses were positive i.e. as road base layer thickness increases, fatigue life (ESALs) increases irrespective of the material type used for the road base. The HMA 28mmDBM was significantly better than the CBEMs although for all the cases considered, the ESAL for HMA surfacings supported by the CBEMs were all clearly above 1 million ESALs for critical condition. This was particularly obvious for the 50mm HMA surfacing as the predicted lives were consistently higher than those for the 75mm and 100mm HMA surfacing layers. The modular ratio of the surfacing layer and the underlying CBEM layer might be partly responsible for these unexpected results. The pavements containing the CBEM layers as road base are inverted compared to the HMA in this regard i.e. modular ratio for CBEM pavements are ≥ 2, while modular ratio for the HMA pavement is < 1. Although the rutting lives of the subgrade have to be checked, the results suggest that it is not structurally and also economically viable to use surfacing thicknesses higher than 50mm in this regard. Asphalt Academy (2009) advised that the maximum HMA surfacing for pavements containing cold asphalt layers should be limited to 50mm.

Similarly, the observed fatigue lives of the 50mm HMA surfacing for critical condition which is between 2.8 and 10.5 million ESALs seem to be close to the observations of Liebenberg and Visser (2004) for similar pavements in which they recorded 30 million ESALs to failure. Overall, the RAP CBEMs performed better than the VACBEM with the 10dmmCBEM and 20dmmCBEM offering the best performances in ESALs.

7.9 The Effect of CBEM Road Base Layer Property on Subgrade Performance

The rutting potential of the subgrade under the various pavement structures analysed were also studied. The governing criterion here is the vertical strain on top of the subgrade. Equations 7.1 and 7.2 were used for the computations of ESALs as earlier described. Figures 7.25 to 7.27 are relevant here. The figures which are for life (ESALs) to critical condition, generally indicate that the HMA (28mm DBM) used for the road base also offers the best protection for the subgrade of all the materials studied. All the CBEMs showed similar responses i.e. positive trend such that ESALs increase with corresponding increase in road base

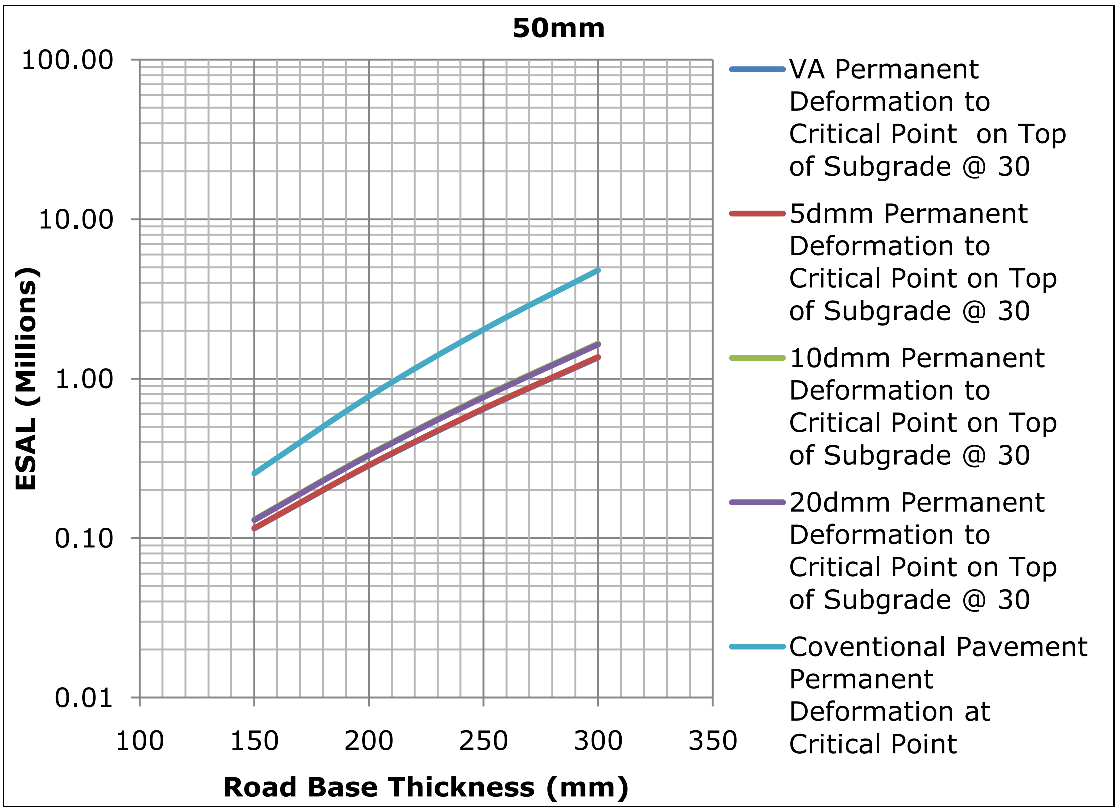


Figure 7.25: Effect of Road Base Layer Thickness and Material Type on Permanent Deformation to Critical Condition on top of Subgrade for Surfacing Thickness of 50mm HMA

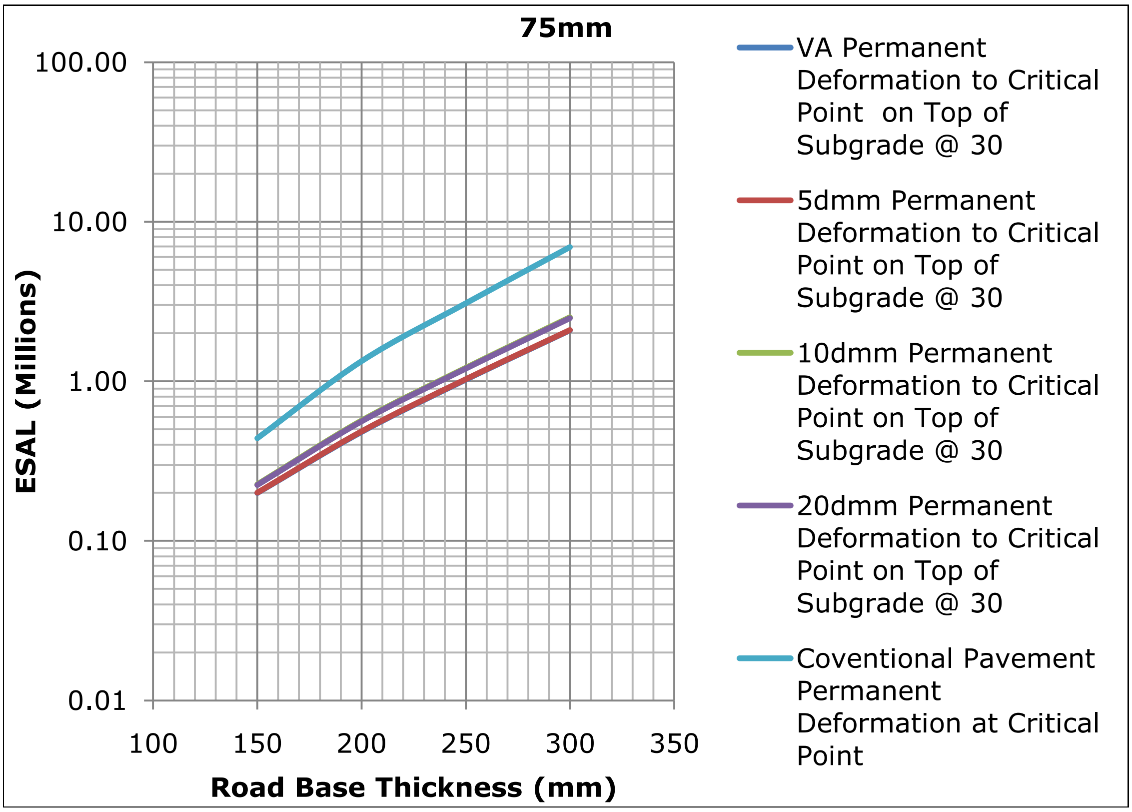


Figure 7.26: Effect of Road Base Layer Thickness and Material Type on Permanent Deformation to Critical Condition on top of Subgrade for Surfacing Thickness of 75mm HMA

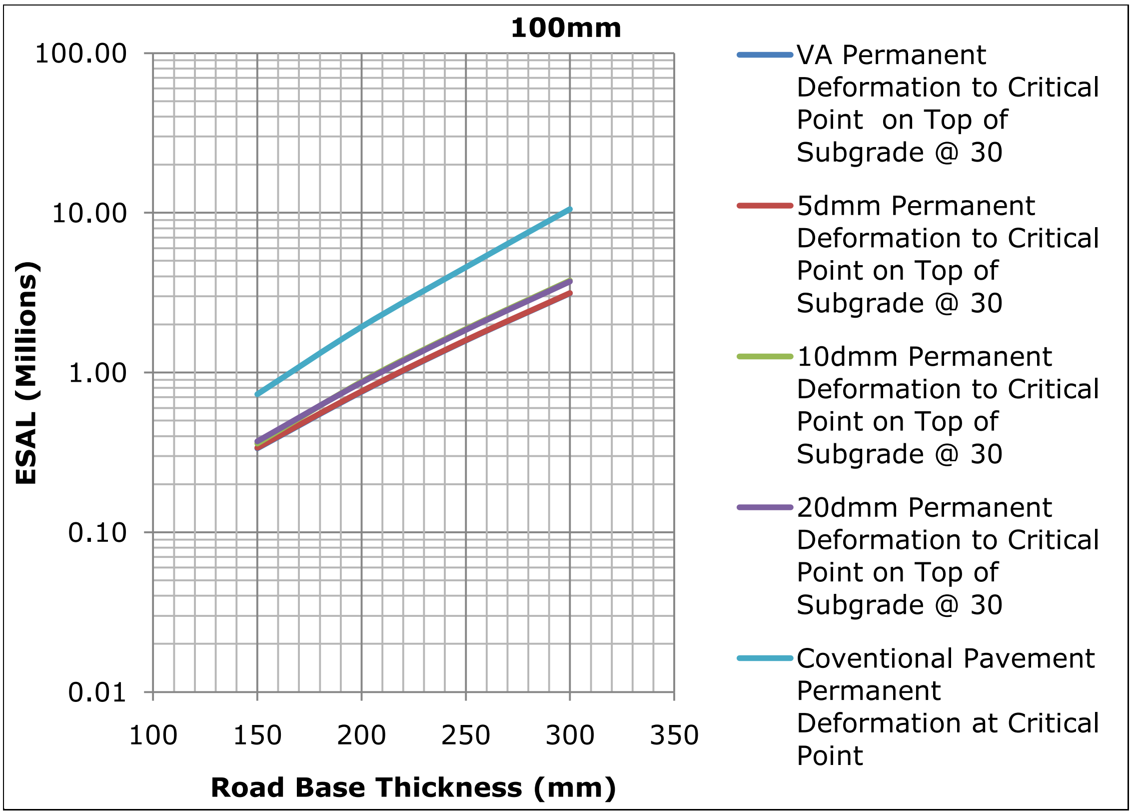


Figure 7.27: Effect of Road Base Layer Thickness and Material Type on Permanent Deformation to Critical Condition on top of Subgrade for Surfacing Thickness of 100mm HMA

thickness just as was the case for the conventional pavement. Both the 20dmmCBEM and 10dmmCBEM indicated the best protection for the subgrade of all the CBEMs studied.

The results showed that the surfacing layer had little or no significant influence on the response of the subgrade in the limited study carried out here whereas the road base thickness significantly influenced such responses. It is worth noting that the ESALs to critical condition for all the cases considered here were generally low compared to the ESALs obtained for the other parameters discussed earlier. The subgrade life also of course depends on the stiffness properties of the sub-base and subgrade. Although the design charts are hypothetical, they therefore suggest that the properties of the subgrade must be improved on in order to improve on the rutting responses of the subgrade and not necessarily the HMA surfacing and the CBEM road base layers.

7.10 Summary

In this chapter, Structural Design and Modelling of Flexible Pavements have been carried out. Although these examples are hypothetical in nature (sub-base and subgrade not varied in this work), the results are good indicators for ranking the road base materials. Along-side pavements containing CBEMs, conventional pavements were considered for comparative analysis. The following points can be drawn from the results obtained in this chapter:

- KENLAYER though time consuming, is a good analytical tool for analysis of flexible pavements involving non-linear elastic materials while BISAR 3.0 and OLCRACK are both good for linear elastic layers in multi-layer analysis of flexible pavements.
- Vertical stress distribution for pavements containing non-linear materials (CBEM) is similar irrespective of material type. Strains (vertical and horizontal) are however different.
- BISAR 3.0 has the limitation that it can only give similar results for stresses and strains as KENLAYER when the correct modulus values for each sub-layer considered are used.
- Except for ranking CBEMs, the use of maximum horizontal strains anywhere within the CBEM layers for design purposes is not sensitive to changes in thicknesses of the surfacing and CBEM layers. It is thus better to focus on checking the fatigue lives of surfacing and the rutting lives of subgrade for pavements containing CBEM layers.

- From the analysis done here and the subsequent design charts produced, the 28mm DBM and 5dmmCBEM indicated the best fatigue lives followed by 20dmmCBEM and 10dmmCBEM. VACBEM indicated the worst performance.
- Fatigue lives of the 50mm HMA surfacing for critical condition which is between 2.8 and 10.5 million ESALs suggest longer lives to failure although it is difficult to make predictions. Liebenberg and Visser (2004) recorded 30 million ESALs to failure for similar pavements.
- 28mm DBM gave very good support to 30/14 HRA surfacing layer compared to the CBEMs. A surfacing layer on HMA road base is highly sensitive to changes in thickness of the road base compared to CBEM road base. Such changes in HMA road bases resulted in significant increase in ESALs.
- The low stiffness values of the CBEMs did not affect their rutting resistance since road base made of either HMA or CBEM indicated similar responses in rutting resistance.
- The analysis showed that pavements containing HMA materials as surfacing and road base are better in performance compared to those made with CBEMs and thus confirms the findings/views of experts in the field.
- However, with all things being equal, pavements containing 5dmmCBEM/ 20dmmCBEM and probably 10dmmCBEM as road base should conveniently accommodate about 5million ESALs before reaching the end of their service lives. The results presented here are generally for critical conditions, which imply failure would actually happen at larger ESALs values than presented in the design charts produced.
- The RAP CBEMs overall performed better than the VA CBEM in terms of ESALs applications that they offer.
- These RAP CBEMs would be applicable in low to medium volume traffic scenarios as road base materials.

7.11 Conclusion

The hypothetical examples presented in this chapter showed that KENLAYER, though time consuming, may be better than BISAR 3.0 for analysing pavements containing CBEMs as road base layers since the materials are non linear in elastic response. When KENLAYER seemed not to be sensitive enough in analysing the pavements with thin surfacing layer thicknesses of 30mm for horizontal strains at the bottom of the surfacing layer, OLCRACK proved in this regard, though the stiffness values calculated in the KENLAYER had to be used. However, in order for OLCRACK to have better application in modelling pavements containing non linear elastic layers, experimental modification reflecting the non linearity of the CBEMs would have to be introduced to the software. OLCRACK is basically good for estimating fatigue lives to failure for linear elastic layers in multi-layer analysis of flexible pavements.

The fundamental design criterion arrived at in this hypothetical design exercise was the horizontal strain at the bottom of the HMA surfacing layer for pavements containing CBEMs as road base layer. The less fundamental design criterion was the vertical strain on top of the subgrade. These two design criteria were chosen since, when using them, the type and thicknesses of CBEMs were reflected in the design ESALs. Though the result from the less fundamental criterion informs the use of improved subbase and subgrade, the RAP CBEMs offered better protection to the subgrade than the VACBEM. Similarly, the RAP CBEMs offered better support to the surfacing than the VACBEM.

Other design criteria that should have complemented those earlier stated are the fatigue and rutting lives of the CBEM layers. Common practice requires the use of the maximum strains in the layer. Although the CBEMs seemed to have been ranked, however, the use of such maximum strain values along with changes in the thicknesses of the layers were of no effect on the ESALs probably because of the modular ratio of the surfacing and CBEM layers, whereas it did for strain values at the bottom of the CBEM layers. It is not advisable to use the fatigue and rutting lives of the CBEM layers as design criteria for now. It is however possible that the representative strains required for considering CBEM layers in structural design of pavements can be ascertained through full scale pavement tests and this will be an interesting area for further research work.

8 Some Guidelines for the use of CBEM in Hot Tropical Climates

8.1 Overview

This chapter is deemed important to this study as it details some guidelines for producing and using cold bituminous emulsion mixtures (CBEM) containing severely aged reclaimed asphalt pavement (RAP) in hot tropical climate. Such guidelines are lacking at the moment and have been identified as major barriers preventing developing countries with hot tropical climates from integrating recycling into their road maintenance/rehabilitation programmes. It is envisaged that employing such methods and materials for road rehabilitation/reconstruction would assist in stretching road funds, budgets for which in recent times have increasingly become inadequate in such developing countries.

Although there are a number of methods for recycling RAP materials, the cold in-plant recycling approach using bitumen emulsion has been chosen and considered more appropriate for such countries in order to ensure good quality control of the mixtures produced and so as not to require expensive equipments. It is by ensuring such quality control that the full benefits of recycling can be harnessed.

This chapter essentially focuses on two fundamental issues affecting mix (job mix), and structural design considerations for cold recycled bitumen emulsion mixtures. These are considered germane towards achieving success for such recycling programmes. Issues affecting construction considerations which are also very important are well detailed by ARRA (2001), Wirtgen (2004) and Merrill et al (2004).

8.2 Laboratory Mix Design Considerations for the Use of Cold Recycled Bituminous Emulsion Mixtures

Mix design as earlier described in Chapter 5 of this thesis essentially entails the formulation in the right proportions of materials (aggregate + binder + filler) that will constitute the asphalt mixture whether cold, warm or hot mixed. While such an exercise which involves mainly virgin materials prepared hot is straight forward though still regarded under some circumstances as a complex material by some researchers (Collop, 2009), the inclusion of RAP as full or partial substitute for virgin aggregates makes the case more complex still. This is because RAP in itself is a composite (conglomerate of aggregate and aged binder which could even be polymer modified type) material that must be characterised and checked for its integrity or nature before it is used in the mix. This indeed is an additional but very important exercise most especially for developing countries where information on the history of such reclaimed pavements might be difficult to come by. Furthermore, it is considered important to do this even when such history is known since RAP though depending on its nature/integrity and the percentage it constitutes in the mix, can significantly affect/alter the overall mechanical behaviour of the mix contrary to the desired expectations of the pavement designer when the true nature of the RAP is not considered.

As earlier mentioned, the cold recycling *ex situ* (in-plant) approach chosen in this work allows for proper assessment of RAPs before they are used. Other factors responsible for this choice of recycling have been previously discussed in Chapter 2 of this thesis. Although factors that must be considered for pavement recyclability have likewise been dealt with in Chapter 2 of this thesis, and extensively in the Basic Asphalt Recycling Manual (ARRA, 2001), Wirtgen Cold Recycling Manual (2004) and Merrill et al (2004), it is believed that this method is applicable in both new construction and rehabilitation works. This is because cold asphalt produced in-plant has better potential for consistency in properties compared to the *in situ* approach since there is room for proper quality control. Furthermore, cold recycling whether *in situ* or *ex situ* is almost always found applicable where there is a major need for the improvement of structural capacity of road pavements (ARRA, 2001), which in procedure, is closely related to new construction.

It implies thus that once the project evaluation, economic analysis and preliminary selection favour cold recycling, then proper pavement assessment, structural capacity assessment, material properties assessment, traffic assessment, constructability assessment and environmental implications as suggested by ARRA (2001) and Wirtgen (2004) must be carried out. When all these necessary steps have been taken, then the mix design which is the focus of this section is conducted.

These guidelines have been produced in concert with the suggestions of ARRA (2001), AI & AEMA (1997), Wirtgen (2004), Merrill et al (2004), Ibrahim (1998), Thanaya (2003) and Smith and Jones (1998) just to mention a few. However they have been done in the light of countries located in hot tropical climates. It should also be noted that the cold asphalt materials in question here are applicable in the construction of road base of flexible road pavements and hydraulic binders though advantageous, have not been considered.

8.2.1 RAP Characterisation

The importance of proper RAP characterisation cannot be over emphasised whenever RAP materials are to be used in the production of cold mixtures. This obviously is the first step in the process of job mix design as the integrity of the RAP is ascertained in the process. This characterisation is done by conducting a composition analysis using relevant standards (BS598-102:2003, BSEN 933-1 and BS598-101:2004). This will give information on the content of residual bitumen in RAP and as well the gradation of extracted aggregates from RAP. It is advised that the properties of the recovered bitumen be ascertained subsequently in order to know the design approach to follow i.e. whether to treat RAP as black rock or not. It is envisaged from the findings of this work that RAP with residual bitumen of 5dmm penetration is active under hot tropical conditions although the extent of the activity is difficult to quantify. A suggested method for recovering the aged residual binder in RAP is BS EN12697-4:200. Some of the suggested tests are:

1. Physical properties of recovered bitumen from the mix
    a. Penetration at 25ºC
    b. Softening point
    c. Viscosity
2. If possible the stiffness properties in the form of Complex Modulus at 25ºC

If the penetration is below 5dmm, there is no need to conduct the other tests on the binder. The RAP should be treated as black rock i.e. as virgin aggregate. Processing of the RAP follows i.e. crushed and screened to required sizes suitable for a 20mm DBM as stated in ORN 31 & 19 (see Table 5.4 in Chapter 5 of this thesis). Smith and Jones (1998) suggested that such mixes are able to give satisfactory support to road pavements in hot tropical climates where overloading is a problem. In order to meet the gradation requirements of 20mm DBM virgin aggregates could be introduced to make up for the deficient sizes.

When the penetration of the residual bitumen is 5dmm and up to 20dmm, the other tests listed above are worth conducting in order to gain a fuller understanding of the behaviour of the residual bitumen. These are useful in determining the proper grade of recycling additive i.e. recycling agent for the cold recycling. However, caution must be taken in deciding on the grade of additive (recycling agent) since the extent of possible rejuvenation or not of the residual binder cannot be ascertained at the moment. For residual bitumen with the range of penetration 5dmm – 20dmm, bitumen emulsion containing base bitumen of

50dmm was found suitable. Meanwhile, the determination of the gradation of the extracted aggregate is necessary since the residual binder is considered to be active under hot tropical conditions. It is believed that fines and fillers alike which are trapped in residual aged bitumen begin to contribute to the properties of the cold mixture upon the softening or possible rejuvenation of such residual aged binder thus potentially affecting the volumetrics of the mixture. This is considered appropriate.

The deficient sizes in the extracted aggregate from the RAP aggregates are supplemented with virgin aggregates to suit the requirements for 20mm DBM earlier suggested. ARRA (2001) in buttressing this cautioned that the addition of new aggregates should not be based on the gradation of the aggregate recovered from the RAP (RAP aggregates) alone, because coarse angular particles of conglomerated fines can be manufactured by cold milling that are not broken down by traffic. However, the addition of virgin aggregates can be beneficial and justified when excess binder is present or when it is deemed desirable to increase structural capacity of the mix. No value is given here for percentage addition of virgin aggregates to mix since each individual project merits individual treatment.

It is similarly important to determine the particle density and water absorption properties of both the RAP and virgin aggregates in order to make the appropriate decision with regard to bitumen emulsion content required for the mix.

8.2.2 Considerations for the choice of bitumen emulsion

Although there are a number of recycling agents which have been successfully used in cold recycling as discussed in Chapter 2 of this thesis, this guideline focuses on bitumen emulsions. Factors to consider for choosing other recycling agents have been properly addressed by ARRA (2001), Wirtgen (2004) and Merrill et al (2004).

The choice of suitable bitumen emulsion type i.e. cationic or anionic is determined by the aggregate type i.e. whether it is siliceous or calcareous. Cationic bitumen emulsion is appropriate for siliceous aggregates (granite for example), and anionic bitumen emulsion is considered appropriate for calcareous aggregates (e.g. limestone aggregates). This is important as it strongly affects the adhesion of the base bitumen in the emulsion to the surface of the aggregates. Medium to slow setting emulsions are considered appropriate for hot tropical conditions so as to guard against setting before compaction is carried out.

8.2.3 Estimation of pre-wetting water content and binder demand

Once a decision on the type of bitumen emulsion suitable for the project is taken, the determination of the quantity of bitumen emulsion required for the mix is next. Some important parameters are relevant here. These are pre-wetting water content, initial emulsion content (IEC), total fluid content (TFC) and the optimum bitumen emulsion content (OBEC) in the optimum total fluid content (OTFC).

1. The whole exercise starts by estimating the trial/initial emulsion content. The empirical formula suggested by Asphalt Institute method (MS-14, 1989) is considered appropriate for this thus:

$$P = (0.05A + 0.1B + 0.5C) \times (0.7) \qquad 8.1$$

Where,

P= % by weight of binder demand by mass of total mixture

A= % of mineral aggregate >2.36mm

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

C=% of mineral aggregate < 0.075mm

The bitumen content in the emulsion must be considered.

2. Apart from helping to activate the charges on the surface of aggregates which enhances attraction of emulsion to the aggregates, the addition of pre-wetting water enhances the compaction characteristics of the cold mix when optimal values are used. Pre-wetting water is determined with due regard to the natural moisture content of the aggregates (RAP or virgin) by determining both the Optimum Moisture Content (OMC) and Maximum Dry Density (MDD) of the mix using a range of moisture content (say 0-12% at convenient intervals) for the moisture-density relationships. Though the gyratory compactor was found suitable for this exercise other convenient methods most especially as obtained in the modified proctor test can also be used. No specific value will be given here for these values (OMC and MDD) since each project should be given individual treatment.

3. Once the OMC is determined, the IEC value obtained in equation 8.1 is used to replace the same moisture proportion in the OMC to constitute the TFC. The difference between OMC and IEC is the pre-wetting water content. These are simply expressed as follows:

TFC = Natural Moisture Content in Aggregates + IEC +

Pre-wetting Water Content 8.2

Further optimisation must be conducted to determine the optimum bitumen emulsion content (OBEC) to give the optimum total fluid content (OTFC) for the mix.

The OBEC in OTFC is determined in a similar fashion as is normally done for the determination of OMC and MDD but now by checking the indirect tensile strength (ITS using the Universal Testing Machine) and/or the indirect tensile stiffness modulus (ITSM using the Nottingham Asphalt Tester - NAT) of the mixtures in dry and wet conditions using a range of residual bitumen content (RBC). The relationships generated in the process are the TFC – ITS and or TFC – ITSM relationships. Meanwhile, the OBEC must be chosen giving allowance for the absorption properties of the mix, and considering the fact that a significant portion of the moisture will evaporate upon curing. For example, while keeping the emulsion content constant at 'P%' by mass of the aggregate (as empirically determined in equation 8.1), the pre-wetting water content can be increased at intervals of 1.0% by mass of the aggregate starting from total fluid content equal in value to the OMC and ending at a convenient point, e.g. 10%. The procedure followed for preparing the specimens should be such that the pre-wetting water is first added to the dry batched mixture and then mixed using speed II on the Hobart Mixer for 60s. The required emulsion is subsequently added and the mixture mixed for another 60s. It is suggested that each batch of mixing should have sufficient materials for 4 specimens. The required materials are subsequently placed in 100mm diameter split moulds and then compacted applying the same compactive effort. These are left in the moulds for 24hrs (in a sealed condition) at the same ambient temperature before they are carefully extruded.

Since the mixtures produced in the process are still very fragile even after 24hrs curing in the mould, the produced specimens must be further cured in a forced draft oven. Curing at 60ºC for 96hrs is considered appropriate here for the specimens that are tested dry to represent typical hot tropical conditions. Wet vacuum saturated specimens at a pressure of 140mbar should be further conditioned in water for 24hrs at 30ºC (average maximum ambient temperature in the tropics) and followed subsequently by further conditioning at 20ºC (average minimum ambient temperature in the tropics) before the ITS and ITSM tests are conducted at 20ºC. This procedure should fully account for the recovery potentials of the mixture after exposure to severe conditions. Testing for ITS and ITSM at

20ºC ranks CBEMs better than testing at 30ºC which is the more appropriate temperature for hot tropical conditions. The results are used to generate TFC-ITS/ITSM, and TFC-ITSR/ITSMR relationships where the optimum total fluid content (OTFC) is determined.

In order to determine the OBEC in the OTFC, further optimisation must be conducted just as explained above but now keeping the OTFC constant. Ideally, and if possible the best way of establishing this is by varying the bitumen/water ratio in the emulsion, but since most emulsions are proprietary, varying the emulsion/ pre-wetting water content ratio should be adopted.

4. Now that the OTFC is known, it is important to ensure that the appropriate mixing time for the mixture is ascertained. 2-4 minutes using the sun and planet mixer is considered adequate and similarly in a suitable Hobart mixer. For example, in a 4-minute mixing time, the first 2minutes is used for the pre-wetting water addition/mixing, while the remaining 2minutes is used for the bitumen emulsion addition/mixing.

5. Further investigations must be conducted on the designed (fully optimised) mix to ascertain its mechanical performance. The results of such will be useful for the structural design of the layer. Of importance are the ITSM and fatigue tests. If possible, permanent deformation and resilient modulus tests can be conducted although not always practical. The more the traffic volume anticipated, the more important it is to conduct these tests. With low-medium volume roads, ITSM and fatigue tests should suffice. The durability (ageing and water susceptibility) of the mixture must also be considered. All these will give an indication on the performance of the mixture in service. Figure 8.1 and Table 8.1 are summarised procedures for the cold mix design.

8.3 Structural Design of Cold Recycled Bituminous Emulsion Mix Layers

A number of methods are available for pavement structural design. These include, catalogue design methods, structural number method, deflection based methods and the mechanistic methods (Wirtgen, 2004). Wirtgen (2004) stated that the use of a mechanistic pavement design method for rehabilitation works is the most fundamentally sound approach since the method is based on engineering principles, using the performance data developed through research on different materials.

From the work done in this thesis, it is important to consider the non-linear elastic

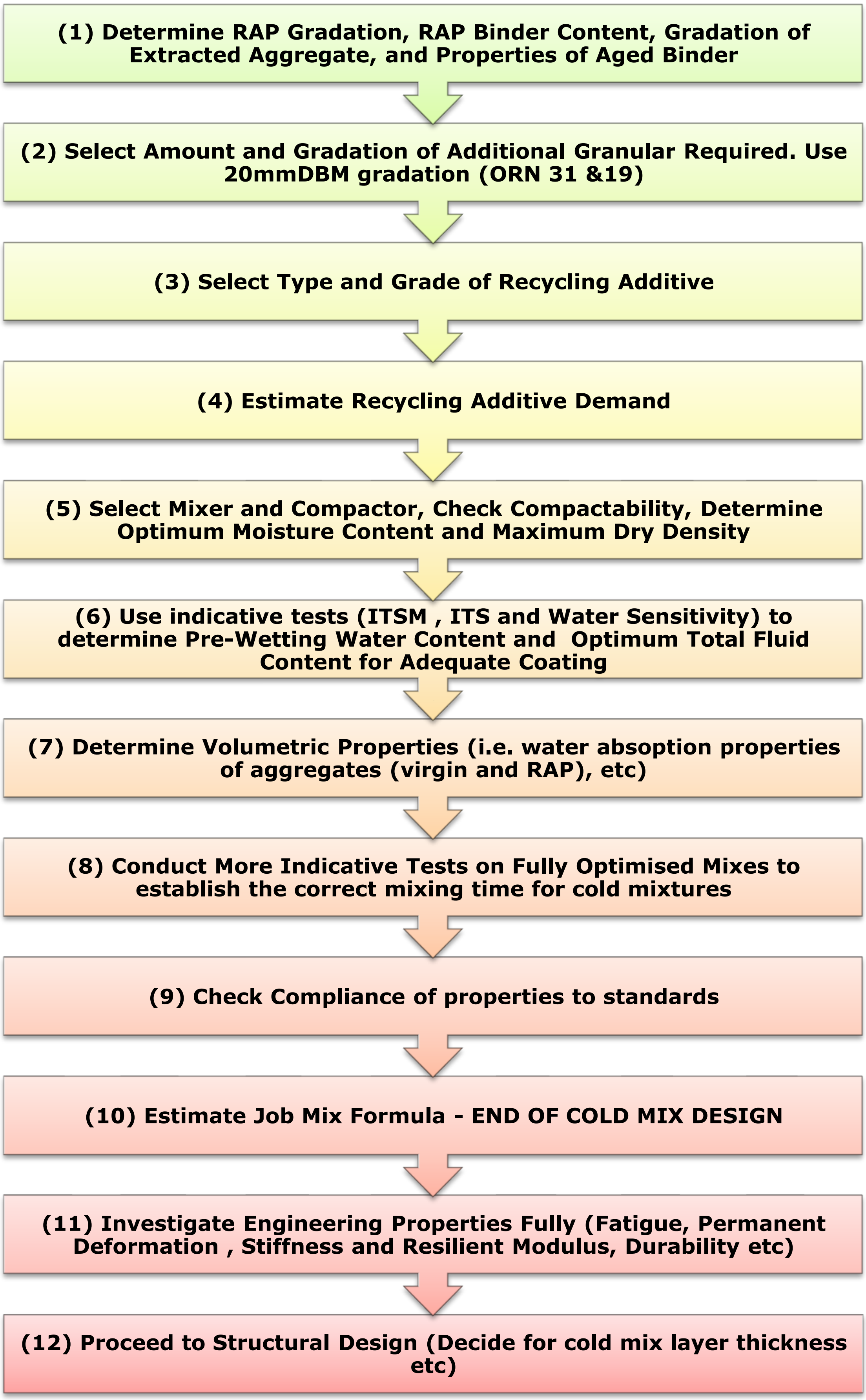


Figure 8.1: Flow Chart for Cold Recycled Bituminous Emulsion Mix Design

Table 8.1: Step by Step Approach to Cold Recycled Bituminous Emulsion Mix Specimens Preparation

<table>
<tr><th colspan="3">1.Materials for Cold Bituminous Emulsion Mixture (CBEM)</th></tr>
<tr><td>RAP and Virgin Aggregates</td><td colspan="2">Characterise RAP (check residual bitumen content and properties and the gradation of extracted aggregate). Determine quantity and sizes of virgin aggregates required to fill deficiencies. Use 20dmm DBM gradation</td></tr>
<tr><td>Optimum Total Fluid Content</td><td>Start by estimating trial emulsion content. Determine the pre-wetting water content using moisture-density relationships.</td><td>Determine OTFC using TFC – ITS and or TFC –ITSM relationships.</td></tr>
<tr><td>Batching</td><td colspan="2">Each batch contains enough materials for 4 cores at 910g per sample</td></tr>
<tr><th colspan="3">2. Mixing</th></tr>
<tr><td>Material Conditioning prior to mixing</td><td colspan="2">- Materials (aggregate mix, bitumen emulsion in air tight container and demineralised water) preconditioned over night at the desired mixing and compaction temperatures (32°C)
- Set Sun and Planet Mixer also to the desired temperature over night (or 3hrs to the time of mixing)
- Precondition prepared moulds at the required temperature</td></tr>
<tr><td>Fluid Addition and Mixing Technique</td><td colspan="2">- Add the prescribed amount of pre-wetting water to mix in the mixing basin.
- Mix continuously for 60s and stop the mixer. Stir mix manually using a spatula. Mix again for 60s
- Add the prescribed bitumen emulsion content to water pre-wetted mix
- Mix for 60s and stop the mixer. Also stir manually to obtain homogeneous mix. Complete operation by mixing for 60s more. Place loose CBEM in moulds and subsequently condition for 30minutes</td></tr>
<tr><th colspan="3">3. Compaction</th></tr>
<tr><td>Moulds</td><td colspan="2">- Prepare 100mm diameter moulds and precondition overnight
- Upon mixing, place 900g of CBEMs in each mould
- Condition for about 30minutes</td></tr>
<tr><td>Compaction Information</td><td colspan="2">- Set angle of gyration to 1.25°
- Compaction pressure to 600kPa
- Target 200gyros for good compaction
- Target 100gyros for mild compaction</td></tr>
<tr><td>Specimen Size</td><td colspan="2">- Diameter = 100±1mm
- Height = 50±2mm (requires no cutting)</td></tr>
<tr><th colspan="3">4. Curing</th></tr>
<tr><td>Stage 1 of Curing</td><td colspan="2">First 24Hrs after compaction
- Keep in mould (sealed) condition for 24hrs at ambient condition (temperature of about 20°)
- De-mould or extrude specimens carefully after 24hrs
- Check mass</td></tr>
<tr><td>Stage 2 of Curing</td><td colspan="2">Curing in the Forced Draft Oven After 24Hrs of Stage 1 Curing
- Cure @ 40°C over 12hrs (Early Life)
- Cure @ 40°C over 72hrs (Intermediate Life)
- Cure @ 60°C over 96hrs (Fully Cured Condition)</td></tr>
<tr><th colspan="3">5. Handling of Specimens After Curing</th></tr>
<tr><td>Storage After Curing</td><td colspan="2">If specimens are not tested immediately after curing, store in the cold store at 5°C until needed for testing</td></tr>
<tr><td>Conditioning Prior to Test</td><td colspan="2">- Prior to any dry test (i.e. ITSM), place specimens in the conditioning cabinet at the desired test temperature for about 8hrs (preferably over night)
- Specimens for wet tests (for water sensitivity) should be conditioned for 2hrs in water at the desired test temperature before testing</td></tr>
</table>

behaviour of the Cold Recycled Bituminous Emulsion Mixtures (CRBEM). This informs that the resilient modulus test results are required in this regard for the input parameters in the mechanistic analyses. The KENLAYER computer software developed by Huang (2004) which is commonly available is considered appropriate in this regard for analysing the response of the non-linear layer to load. The desired outputs from the analyses are:

- Horizontal tensile strain at the bottom of the HMA (surface layer)
- Vertical strain on top of the subgrade

A range of layer combination and thicknesses for surfacing layer (HMA), road base layer (CRBEM), sub-base layer (unbound granular materials- lateritic gravel) and subgrade layer must be tried until the desired layer combination is found that satisfactorily meets the requirements for the desired pavement design life. This must be economically viable compared to when the conventional pavement is used. Experience shows that with a well designed CRBEM road base layer, and good sub base and subgrade layers, a surfacing layer (HMA) thickness of 20mm – 50mm is achievable.

For the prediction of Equivalent Standard Axle Load applications, transfer functions are required for realistic values. It is advised that transfer functions or shift factors for critical conditions for fatigue and permanent deformation are used for such estimations as follows:

Fatigue;

Values obtained when the horizontal strains from multi-layer analysis have been substituted in fatigue life equations obtained from fatigue tests should be multiplied by 77

Permanent Deformation;

N_{cr} =fr $\left(\frac{7.6\ x\ 10^8}{\varepsilon_z^{3.7}}\right)$

Where:

- N_{cr} = life to critical condition (10mm rut)
- Fr = a rut factor to account for the hot mixture type- 1.56 for dense bitumen mixtures

Such shift factors are to be used with caution for the CBEMs (cold mixtures) since they were developed for HMA. At present, no such values are available for cold mixtures. The adopted factor of 77 may be too low to describe total pavement life. Considering the fact that cold mixtures are not as good as HMA as observed in this thesis, it is believed that the appropriate value for life to failure should be somewhere in between 77 and 440 for critical and failure conditions respectively. These shift factors are based on the Nottingham Asphalt Design method for hot mixtures.

For the construction of pavements which contain cold recycled bituminous emulsion mixture (CRBEM) layers, the guidelines for such given by ARRA (2001), Wirtgen (2004) and Merrill et al (2004) are relevant. However, it is advised that such constructions should be done in the harmattan (dry) season in hot tropical countries such as Nigeria, and where very thick layers of road base CBEMs are considered i.e. from 200mm to say 300mm in order to minimise the thickness of the HMA surfacing, such a CBEM layer should not be laid in one lift but in layers (of say 100mm thick) to allow for adequate curing. This can be done at intervals of 12hrs for good performance in a typical hot tropical climate considering the stiffness values observed in this limited study. It follows therefore that 12hrs after being laid such CBEM layers can be opened to vehicular traffic. This is believed to give room for consolidation of the CBEM layers before they are sealed with a HMA surfacing layer.

9 Conclusions and Recommendations for Future Research

9.1 Conclusions

A summary of the major findings for the research work presented in this thesis in the form of conclusions are drawn as follows:

1. Major Conclusions from Literature Review on Flexible Pavement Engineering and Current Practices

a. The road Network in Nigeria and throughout Sub-Sahara Africa (hot tropical belt) is in a deplorable state due to long neglect as a result of lack of funds for maintenance and rehabilitation. The situation can only get worse under the present global economic recession.
b. There is urgent need to introduce alternative but sustainable methods and materials in order to significantly cut down on costs and also stretch available funds.
c. One out of the sustainable methods that appeals and which can be applied without spending huge amounts on machinery is cold recycling RAPs with bitumen emulsion. This offers a potential cut in costs in the region of 40-60%. Cold recycling *ex situ* using bitumen emulsion should not pose any problems since cutbacks which are similar in operation to emulsions have been used in the industry for a long time and thus are easily adaptable. Such would also ensure good quality control.
d. There is a need to develop guidelines that will assist in making provisions for the use of such alternative sustainable methods and materials in the highway design manual of Nigeria which is lacking at the moment.

2. Major Conclusions from Literature Review on Laboratory Investigations on Cold Mixes

a. Ageing of bitumen whether in the short or long term is inevitable in bituminous road pavements. Ageing in bituminous materials is evidenced by physical hardening which could either be advantageous or otherwise in a road pavement.
b. Laboratory protocols (accelerated) for simulating ageing of bituminous road pavements in temperate climates have been developed. Such protocols are useful for predicting performance in service of bituminous road pavements. However there is a need to develop similar laboratory protocols for road pavements in hot tropical climates.
c. Cold mixes are cost effective, energy efficient and environment friendly, although inhibiting factors such as low early life stiffness and strength development, high air void contents and presence of moisture in the mix are

still preventing such mixtures from being fully embraced in some developed countries.

d. Most of the studies reported in literature on cold mixes have focused on temperate climates, though with good results in most cases. There is lack of evidence in literature on similar studies conducted in hot tropical climates and therefore there is a need to urgently look at this area for such sustainable materials to be found applicable in such regions.
e. There is no universally accepted method for cold mix designs (job formula) although most of the methods which have been advanced in literature are based on the Hveem and Marshall methods of mix design.
f. Stiffness modulus, fatigue response, deformation properties, resilient modulus, and water susceptibility are good means for assessing the performance of cold mixes.
g. Knowledge about the interaction between virgin binders and aged residual binders in cold recycled mix is still ambiguous and needs further exploration.
h. RAPs are still regarded as black rocks when they are used in cold recycling.
i. The effect of cold mix preparation temperature (mixing and compacting) of cold mixes has not been fully accounted for in literature.

3. *Major Conclusions from Laboratory Ageing Protocol for Producing Severely Aged RAPs*

a. Penetration values as low as 6dmm for wearing course and 5dmm for binder course have been reported in literature for asphalt pavements in East Africa-typical hot tropical developing region.
b. Laboratory ageing protocols for producing RAPs are needed to reflect what obtains in road asphalt pavements in such hot tropical developing regions of the world.
c. Generally, as the binder used in this study (straight run paving grade binder 40/60 of Venezuelan origin with a penetration of 53dmm) ages, complex modulus, asphaltene content and softening point values increase while penetration values reduce irrespective of the ageing protocol i.e. either as bulk binder ageing or mixture ageing, in line with other researchers' findings.
d. Ageing bituminous materials at 85ºC over 120hrs (5days) in the forced draft oven did not yield the severity of ageing (below 6dmm) required to study heavily aged pavements typical of the tropics. This standard protocol (for temperate climates) yielded a residual binder of 23dmm penetration for the original 53dmm penetration binder used. However, it was able to age to 11dmm penetration after 840hrs (35days).
e. Ageing bituminous mixture at a higher temperature of 125ºC in the forced draft oven accelerates ageing, such that it only took 48hrs to reach a penetration of 11dmm compared to 840hrs (35days) for the standard protocol (for temperate climates), thus saving laboratory time.
f. Ageing loose bituminous mixtures in the forced draft oven accelerates ageing. For example, ageing such mixtures at the standard (temperate climate protocol) temperature of 85ºC over 24hrs yielded a pen of 31dmm in this work. Nguyen (2009) in his work in which he used compacted specimens with similar binder reported a similar penetration level of 31dmm after 120hrs of ageing.

g. Ageing asphalt mixtures at 125ºC is comparable to the standard temperature of 85ºC though with minor differences in physical properties at a given penetration.

4. Major Conclusions from Cold Mix Design of CBEMs

a. Bitumen emulsion compatibility with aggregate, pre-wetting water content and optimum total fluid content (OTFC) are important parameters that must be ascertained for cold bituminous emulsion mixtures (CBEMs) to achieve a successful cold (job) mix design.
b. The Indirect Tensile Stiffness Modulus (ITSM) test proved useful alongside the Indirect Tensile Strength (ITS) test as an indicative test for ranking CBEMs during mix design. With ITSM (a non destructive test), materials can be conserved as the same set of specimens tested dry can also be tested for water susceptibility. ITS tests (destructive) are common for mix designs of this nature.
c. Just enough mixing time for both the addition of pre-wetting water and bitumen emulsion are recommended for CBEMs in order to ensure adequate coating, while also ensuring at such timings that the bitumen emulsion in the mix has not started breaking. All these together facilitate good compactability and stiffness properties for the CBEM. Inadequate mixing time can significantly reduce compactability and stiffness response. A 36.1% reduction in stiffness property as a result of such inadequate timings was observed in this study.
d. As observed in this study, limestone filler significantly reduces the water susceptibility of CBEMs. Limestone has been known to be good at inhibiting stripping. However, granite filler gave a better dry stiffness property compared to limestone. Granite aggregate was adopted for subsequent CBEMs mainly because it is often used in Nigeria which is the study area of this work.
e. Mixing and the compaction of CBEMs at a temperature of 32ºC gave better results in compactability, stiffness properties and water susceptibility when compared to those similarly done at 20ºC. The lower viscosity value of the bitumen emulsion at 32ºC as observed in this work was considered partly responsible for this. However such slightly elevated temperature simulates a typical average maximum temperature in the tropics. Improvement in stiffness as high as 39.5% was achieved at a mixing and compaction temperature of 32ºC over 20ºC in such circumstance.
f. As observed in this study, the use of parameters of 600kPa pressure and 1.25º gyration angle in the Cooper Research Gyratory Compactor enhanced the CBEM durability especially the water susceptibility properties compared to other higher compactive efforts such as pressure of 800kPa and gyration angles of 1.5º and 2º. The 600kPa pressure and 1.25º gyration angle parameters were subsequently adopted for the manufacture of all the specimens used in this study.
g. Characterisation of the recovered binder from CBEMs cured at 40ºC over 24hrs in the forced draft oven compared to those cured at 60ºC over 96hrs also in the forced draft oven, showed similar physico-chemical properties. The results obtained from the two protocols also showed that the bitumen in the

mix had not significantly degraded compared to the same properties of the CBEMs characterised prior to curing. Based on the study therefore, curing protocols of 40ºC over 12hrs for early life, 40ºC over 72hrs for intermediate life and 60ºC over 96hrs for fully cured conditions were adopted for the studies on the CBEMs reported in this thesis.

h. As observed in this limited work, filler type does not influence curing, though it is recognised that the use of active fillers such as lime and Ordinary Portland Cement (OPC) can alter this view. Such active fillers were not considered in this work because they are expensive in Nigeria and can render cold recycling unattractive.

5. Major Conclusions from Detailed Investigation on the Mechanical Properties of CBEMs under Laboratory Simulated Tropical Conditions

a. CBEMs as observed in this study generally have high air void contents compared to HMA. They in some instances displayed values as high as 23% with the minimum experienced in this study being around 8%. However, the air void content depends on the condition of curing which essentially has to do with moisture content. In consonance with logic i.e. as cold mixture cures it gains stiffness, early life conditioned specimens displayed the lower values due to the presence of moisture, while the fully cured specimens had the high values. The moisture content at early life condition for all the CBEMs ranged from 1.0% to 1.4% while at fully cured condition it ranged from 0.2% to 0.8%.

b. Mixing and compaction temperature is very important for achieving low air void contents in CBEMs. For example, mixing and compacting CBEMs at 32ºC gave better results than at 20ºC.

c. The RAP CBEMs compact better compared to the control mix i.e. the VACBEM. The 20dmmCBEM had the best compactability.

d. The results showed that the consistency and reproducibility of compactability of CBEMs mixed and compacted at 32ºC is uniform and significantly better compared to those mixed and compacted at 20ºC. The results also confirmed that there is only a little difference between applying a compactive effort of 100gyros or 200gyros (at similar temperature) in compactability of CBEMs when compared to the effect of mixing and compaction temperatures of 32ºC and 20ºC (at similar compactive effort) in which significant difference was observed.

e. Compactability affects the rate of moisture loss in the CBEMs. The higher the initial air void content, the higher the rate of moisture loss from the CBEM. In this regard, the VACBEM showed the highest loss of moisture content.

f. The better the compactability of CBEM the higher the tendency of retaining more moisture content. This however seemed not to have affected the stiffness properties of the CBEMs.

g. The stiffness modulus responses referred to as ITSM for the CBEMs generally showed that the curing protocols used in this study sufficiently simulated the desired conditions in a logical order as stiffness improved from early life to intermediate life and fully cured condition.

h. At early life the ITSM test clearly ranked the CBEMs in a logical order and the results were in agreement with findings on compactability of the CBEMs.

5dmmCBEM had the least percentage improvement at 8.8% while 20dmmCBEM had the highest at 32% compared to the VACBEM. As the penetration of the residual binder in the severely aged RAP increases, there is a corresponding increase in ITSM. The trend line of the ITSM however indicates that an optimum value is very close to that for the 20dmmCBEM.

i. VACBEM as observed in this study shows the highest temperature susceptibility.
j. The RAP CBEMs performed better than the VACBEM in water susceptibility.
k. The results suggest that the RAP CBEMs were not completely acting as black rock most especially when constituted at 32ºC. This view is expressed also being mindful of the fact that the viscosity of the bitumen emulsion at 32ºC as observed in this work was far less than (almost by a factor of 3) that at 20ºC. The increase in stiffness modulus up the range from 10dmmCBEM to 15dmmCBEM and to 20dmmCBEM, with higher values than the VACBEM as observed in this work gives a clue to this. The logic is that if the RAPs acted as black rock, in which the residual binders in the RAPs were inactive, the stiffness modulus of the RAP CBEMs should be similar to the value observed for the VACBEM since all the CBEMs have been given similar treatments. More importantly, it implies that some portion of the residual aged binder in the respective RAPs is possibly rejuvenated or softened and this consequently enhanced the stiffness responses of the RAP CBEMs compared to that of the VACBEM. It is reasoned therefore that under such circumstances, the RAP CBEMs now contain more active and in effect stiffer binder than the VACBEM which subsequently reflected in the wet and dry stiffness modulus values of the RAP CBEMs. Alternatively, the better stiffness performances recorded for the RAP CBEMs over the VACBEM could also have been influenced possibly by changes in volumetrics of the RAP CBEMs which influenced their better compactability (compaction).
l. The poor water susceptibility performance of the VACBEM is believed to be partly due to its high air void content, while the lower air voids of the other CBEMs could have been partly responsible for their better performance.
m. There is a negative relationship between ITSM and air voids i.e. as air voids increase, stiffness modulus reduces for CBEMs.
n. Fatigue lives of the CBEMs generally are lower than those for HMA.
o. Overall, the RAP CBEMs prepared at a mixing and compaction temperature of 32^{o}C performed better than their counterparts prepared at 20^{o}C in fatigue response.
p. Considering the behaviour of both the VACBEM and 5dmmCBEM, it seems that fatigue responses of CBEMs were not affected by air void contents although it was earlier established that air voids have a strong correlation with stiffness of CBEMs but depending on the level of curing attained. Generally, the two referred CBEMs have relatively high air void contents compared to the other CBEMs.
q. VACBEM performed better than the RAP CBEMs in permanent deformation response. The results also seem not to have been affected by the air void contents of the CBEMs as the VACEBEM which performed best in permanent deformation resistance consistently displayed the highest air void contents of all the CBEMs studied.

r. The results of the resilient modulus tests clearly indicate that the resilient moduli of the mixtures investigated in this work are stress dependent as they all fitted the k-Θ model.
s. The resilient modulus test as it was done in this study was able to discriminate the properties (CBEM type) of the mixtures with regard to temperature susceptibility.
t. The results show that the Indirect Tensile Stiffness Modulus (ITSM) is useful for quickly ranking CBEMs and resilient modulus may be more appropriate for pavement design involving such materials since they are non-linear in their behaviour as observed in this study. The resilient modulus values were generally lower than the ITSM stiffness modulus values as observed in this work. This is consistent with the observations of Ibrahim (1998) on VACBEMs.

6. ***Major Conclusions from Structural Design and Modelling of Flexible Pavement Containing CBEMs and Guidelines for the use of CBEM in Hot Tropical Climates***

a. KENLAYER is a good analytical tool for analysis of flexible pavements containing non-linear elastic materials while BISAR 3.0 and OLCRACK are both good for linear elastic layers in multi-layer analysis of flexible pavements.
b. Vertical stress distribution for pavements containing non-linear materials (CBEM) is similar irrespective of material type. Strains (vertical and horizontal) are however different.
c. BISAR 3.0 only gives similar results for stresses and strains to KENLAYER when the correct modulus values for each sub-layer considered are used.
d. From the analysis done here and the subsequent design charts produced, the 28mm DBM and 5dmmCBEM indicated the best fatigue lives followed by 20dmmCBEM and 10dmmCBEM. VACBEM indicated the worst performance.
e. Fatigue lives of the 50mm HMA surfacing for critical condition which is between 2.8 and 10.5 million ESALs suggest longer lives to failure although it is difficult to make predictions. Liebenberg and Visser (2004) recorded 30 million ESALs to failure for similar pavements.
f. 28mm DBM gave very good support to 30/14 HRA surfacing layer compared to the CBEMs. Surfacing layer on HMA road base is highly sensitive to changes in thickness of the road base compared to CBEM road base. Such changes in HMA road bases resulted in significant increase in ESALs.
g. The analysis showed that pavements containing HMA materials as surfacing and road base are better in performance compared to those made with CBEMs and thus confirms the findings/views of experts in the field.
h. With all other things being equal, pavements containing 5dmmCBEM, 20dmmCBEM and probably 10dmmCBEM as road base should conveniently accommodate about 5million ESALs before reaching the end of their service lives. The results presented here are generally for critical conditions, which imply failure would actually happen at larger ESAL values than presented in the design charts produced.
i. The RAP CBEMs overall performed better than the VA CBEM in terms of ESAL applications that they offer. These RAP CBEMs would be applicable in low to medium volume traffic scenario as road base materials.

In conclusion, the findings generally show that severely aged RAPs with residual bitumen of penetration values as low as 5dmm can be successfully used in the construction of road base for low traffic volume roads in developing countries with hot tropical climates. Cold recycling *ex situ* (in-plant) is recommended in this regard. In order to achieve the desired success in recycling projects involving such severely aged RAPs, the guidelines in Chapter Eight of this thesis are useful but each project has to be treated individually. Due to the high air voids observed in the CBEMs, it is advised that a surfacing layer of HMA be placed over the CBEM layer.

9.2 Recommendations for Future Research

Although the limited study reported in this thesis has demonstrated that severely aged RAPs stabilised with bitumen emulsion can be successfully used in road base construction in hot tropical climates, some further investigations are deemed necessary to validate the results. These have been compiled as recommendations for future research as follows:

1. From the limited tests conducted in developing the laboratory ageing protocols for the production of severely aged RAPs presented in this thesis, the results indicated that physico-chemical and rheological properties of more severely aged binders are independent of the mixture ageing protocol temperature. However, this has to be ascertained further by conducting more PAV ageing of binders beyond 35hrs i.e. the maximum used in this work up to the point at which severely aged binders comparable in penetration values to those obtained from severely aged mixtures are reached. The chemical and rheological properties should be compared with those reported in this thesis for validation. Similarly, the measurement of chemical composition should be extended beyond asphaltene content to a full SARA (Saturates, Aromatics, Resins, Asphaltenes) analysis, in order to further explore the relationship between softening points, penetration values and rheological properties of the severely aged binders in relation to the various protocols used in this thesis. This will further ascertain whether there are significant differences or not in the chemical composition of binders aged using the protocols described in this thesis.

2. Due to time constraints, only one gradation type - 20mm DBM suggested by previous researchers for road pavements in developing countries located in hot tropical climates was experimented with in this work. This obviously imposes the constraint of having to process and screen the RAPs in order to conform to the requirements. Future works should explore other gradations. Also, in the research reported in this thesis, only one aggregate proportion ratio was used i.e. 65:35:5 for RAP : Virgin aggregate : Filler, different ratios up to 100% use of RAP as aggregates should be experimented with in order to fully understand the role being played by RAPs in CRBEM.

3. The philosophy of mix design adopted in the research work reported in this thesis assumed severely aged RAPs to be acting as black rock in CBEMs as it is the general belief in the industry most especially in temperate climates. However, since it has been proven in this work that RAPs of residual binder with penetration values as low 5dmm may possibly be active under hot tropical conditions, future research should be aimed at quantifying the extent of transient activeness (or level of rejuvenation if at all) of such severely aged binders in CBEMs under hot tropical conditions. This should help in estimating the optimum total fluid content for CBEMs in a more realistic manner.

4. There were plans to conduct two point bending fatigue tests on the CBEMs studied in this thesis, however due to time constraints the attempt did not go beyond the cutting of the specimens which was successfully done on fully cured compacted CBEM slabs contrary to general expectations. Researchers have always believed that CBEMs are too fragile to be subjected to such cutting. Therefore future research should attempt to validate the results of the indirect tensile fatigue tests (ITFT) conducted on the CBEMs reported in this thesis with the results obtained from two point bending fatigue tests. Some researchers are of the opinion that two point bending tests, in contrasts to the ITFT give room for healing of the mixtures which is the reality onsite.

5. The resilient modulus test protocol developed in this research is worth improving further since it is easy to conduct and requires short duration of time. It is worth noting that the resilient modulus results obtained for the studied CBEMs reported in this thesis using the protocol are consistent with those obtained by previous researchers for similar materials. The test as was conducted subjects the specimen to a range of stress levels (low to high) which pavements are believed to be subjected to in service and it uses specimen sizes similar to those used for indirect tensile stiffness modulus tests (ITSM). The improvement required here should be done by experimenting with conventional specimen sizes used for such resilient modulus tests in future research. More importantly, the results should be validated against the results of tests conducted on similar CBEM materials using the conventional triaxial test protocol.

6. One of the weak points of the research work conducted on the CBEMs which is reported in this thesis is that no field trial was conducted due to time constraints. Similarly, only artificial RAPs were used in the study. The geographical location of the case study (hot tropical countries) in relation to that of the laboratory used for this study did not help matters. The results presented therefore are essentially from laboratory investigations and would require field trials for the validation of the laboratory results. Future laboratory studies should similarly be extended to RAPs obtained from the field.

7. The application of the design charts presented in this thesis is limited for a number of reasons. For example, only one foundation type was used in the analysis and as earlier observed, only one gradation type and a fixed ratio of aggregates proportions (RAP, virgin and filler) were considered. Future

research should attempt to develop design charts that are robust enough to account for all these parameters.

8. Since there is lack of information in the literature for road pavement temperature profile in the tropics, some of the decisions taken with respect to the choice of test temperature for assessing mechanical performance in this work were based on a recognised predictive equation which might not have given a total reflection of the conditions in the road base layer. Therefore, future laboratory tests on CBEMs with focus on the tropics should consider other equations (or means of determining base layer temperature profiles) such that in addition to temperatures of 20, 30, 32 and 40^{0}C that were used, mechanical performance at lower temperatures of 15 and 10^{0}C are considered.

9. As no direct measurements of binder rejuvenation have been undertaken in this work, future works should investigate issues such as viscosity measurement and adhesion properties of binders which should give better insight into the possibility of rejuvenation or not in recycled cold asphalt mixtures.

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