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Mix Design Considerations and Performance Characteristics of Foamed Bitumen Mixtures (FBMs)

By

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ABSTRACT

The sustainability issues in pavement materials and design form a strong incentive for the present work. Using recycled materials in pavements is a sustainable practice that is gaining adoption, particularly for flexible (bituminous) pavements. One approach is to incorporate large quantities of Reclaimed Asphalt Pavement (RAP) into base and sub-base applications for pavement construction. Numerous studies have reported that RAP can be reused as an aggregate in Hot Mix Asphalt (HMA) as well as in cold mix asphalt, granular base, sub-base, and subgrade courses. Cold recycling technology, like hot mix technology, has also become popular in various countries for rehabilitation of damaged bituminous pavements. RAP stabilized with bitumen emulsion and foamed bitumen has been used as a base layer. The present study focuses on Foamed Bitumen treated Mixes (FBMs). Most of the agencies which use FBMs have their own mix design procedures which are the result of numerous efforts over decades. In spite of all these efforts, Foamed Bitumen application in cold recycling in the United Kingdom suffers from the lack of a standardised mix design procedure. To overcome this, the present research objective was to develop a mix design procedure by identifying critical mix design parameters. The mix design parameters that were optimised were Foamed Bitumen content, mixing water content (MWC), and compaction effort. Special attention was given to the simplest yet crucial mix design consideration of FBMs; curing. The thesis also attempted to simulate what should be expected in terms of the performance of flexible pavements containing FBMs as road base.

The mix design parametric study was initially carried out on FBMs with virgin limestone aggregate (VA) without RAP material and a mix design procedure was proposed. Optimum MWC was achieved by optimising mechanical properties such as Indirect Tensile Stiffness Modulus (ITSM) and Indirect Tensile Strength (ITS-dry and ITS-wet). A rational range of 75-85% of Optimum Water Content (OWC) obtained by the modified Proctor test was found to be the optimum range of MWC that gives optimum mechanical properties for FBMs. The proposed methodology was also found to apply to FBMs with 50% RAP and 75% RAP. It was also found that the presence of RAP influenced the design FB content, which means that treating RAP as black rock in FBM mix design is not appropriate. This work also evaluated the validity of the total fluid (water + bitumen) concept which is widely used in bitumen-emulsion treated mixes.

The present work was also intended to better understand the curing mechanism of FBMs and to lessen the gap between laboratory curing and field evolution of these mixtures. This was achieved by evaluating different curing regimes that are being followed by different agencies and researchers, as well as identifying important parameters that affect curing. In achieving this, a link was established between laboratory mix design and field performance by evaluating applicability of the maturity method. The curing regime study provided a valid investigation into the behaviour of FBM taking into account the effect of temperature, curing conditioning (Sealed or Unsealed), curing duration and the influence of cement with different curing regimes. It was found that the temperature is as important a parameter as time, as temperature has a greater influence on curing rate and also on bitumen

properties. Moreover, higher curing temperatures resulted in higher rate of stiffness gain. This trend is not only because of rapid water loss but also implies an increase in binder stiffness at higher curing temperatures. Though the presence of RAP improved the early stage stiffness of FBMs, it slowed down the rate of water loss from the specimens which resulted in smaller stiffness values at a later stage. The experimental results also indicated that cement addition has no influence on water loss trends, but improved the stiffness significantly during all stages of curing.

The study also evaluated the applicability of the maturity method as a tool to assess the in-situ characteristic of FBM layers in the pavement. It was found that replacing the time term with an equivalent age term in the maturity function aided in estimating stiffness rather than relative stiffness. This was possible because of the characteristic curing of FBM in which the limiting stiffness these mixtures reach strongly depends on the curing temperature at least for the length of the curing stages considered in the present study. A strong correlation was found between maturity and the stiffness values obtained from the laboratory tests which resulted in development of maturity-stiffness relationships. The application of the method to assess the in-situ stiffness was presented using three hypothetical pavement sections. The results showed the influence of ambient temperature and the importance of cement addition to FBMs.

The permanent deformation resistance was assessed by performing RLAT tests on cylindrical specimens compacted by gyratory compactor. The RLAT test results indicate that both test temperature and stress level have significant influence on permanent deformation characteristics as expected. The effect of stress on permanent deformation was increased with increase in test temperature. It was also found that from limited tests and mixture combinations, RAP content has only a slight influence on permanent deformation of FBMs. However, the presence of cement led to significant improvement. FBMs were also found to be less temperature susceptible than HMA in terms of permanent deformation and, within FBMs, mixtures with cement were found to be more sensitive than FBMs without any cement.

For assessing the fatigue performance of FBMs, the ITFT was initially used to investigate the effect of cement on the fatigue life. The ITFT tests results showed that the FBMs without cement (50%RAP-FBM) have lower fatigue life than HMA (DBM90) at any initial strain level. Nevertheless, similar to permanent deformation, the fatigue life was improved with the addition of 1% cement to FBMs. However, the above discussion was not found to be completely valid when uniaxial tests were carried out. In stress controlled uniaxial tests, a sinusoidal load of 1Hz frequency was applied axially to induce tensile strain in the radial direction. The failure criterion considered in the study was the number of cycles to reach 50% stiffness and this was plotted against the measured initial strain values. Results indicated that there was not much difference in fatigue life among different mixtures and also between FBM and HMA. However, stiffness evolution curves showed that FBMs fail in a different pattern compared to HMA. Unlike HMA, which showed a three stage evolution process, for FBMs the stiffness actually increased initially to reach a maximum and decreased at a slower rate until failure. It was also found that by plotting curves according to Hopman et al.,(1989) which identifies the fatigue failure transition point, use of the 50%

stiffness criterion for fatigue life evaluation is not a conservative approach. Uniaxial tests also revealed that, although in fatigue the FBMs were found to behave differently from HMA, in terms of permanent deformation, FBMs behave similarly to HMA in that a steady state strain rate was achieved.

Keywords: Foamed bitumen, Reclaimed Asphalt Pavement (RAP), Recycling, Mechanical Properties, Performance Characteristics

I would like to dedicate my thesis to my beloved parents and all my teachers

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DECLARATION

The research described in this thesis was conducted at the Nottingham Transportation Engineering Centre, University of Nottingham between October 2011 and October 2014. I declare that the work is my own and has not been submitted for a degree of another university.

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GLOSSARY

APT	Accelerating Pavement Testing
CBM	Cold Bituminous Mixtures
CBR	California Bearing Ratio
CSS	Creep Strain Slope
CWC	Compacting Water Content
DBM	Dense Bitumen Macadam
DISR	Deep In-Situ Recycling
ER	Expansion Ratio
FB	Foamed Bitumen
FBM	Foamed Bitumen Mixture
FI	Foam Index
FWC	Foaming Water Content
FWD	Falling Weight Deflectometer
GPR	Ground Penetration Radar
HL	Half-Life
HMA	Hot Mix Asphalt
HVS	Heavy Vehicle Simulator
ITFT	Indirect Tensile Fatigue Test
ITS	Indirect Tensile Strength
ITSM	Indirect Tensile Stiffness Modulus
MDD	Maximum Dry Density
MWC	Mixing Water Content
NAT	Nottingham Asphalt Tester
N_{design}	Design number of gyrations
NDT	Non Destructive Testing
NPTF	Nottingham Pavement Test Facility
OBC	Optimum Bitumen Content
OWC	Optimum Water Content
PD	Permanent Deformation
QH	Quick Hydraulic
QVE	Quick Viscoelastic
RAP	Reclaimed Asphalt Pavement
RLAT	Repeated Load Axial Test
SH	Slow Hydraulic
SVE	Slow Viscoelastic
UCS	Unconfined Compressive Strength
VA	Virgin Aggregate
VMA	Voids in Mineral Aggregates
WMA	Warm Mix Asphalt

1 INTRODUCTION

1.1 Incentive

The sustainability issues in pavement materials and design form a strong incentive for the present work. Sustainable development has been defined in many ways, but the most frequently quoted definition is from *Our Common Future*, also known as the Brundtland Report [1] "development that meets the needs of the present without compromising the ability of future generations to meet their own needs." Using recycled materials in pavements is a sustainable practice that is gaining adoption, particularly for flexible (bituminous) pavements. One approach is to incorporate large quantities of Reclaimed Asphalt Pavement (RAP) into base and sub-base applications for pavement construction. RAP material, obtained from milling of existing distressed bituminous surfacing in pavement construction and rehabilitation works, is being routinely used in many countries to conserve natural resources. This RAP often contains high-quality, well-graded aggregates that are coated with bitumen. Economy, ecology, and energy conservation are all served when bitumen and aggregate – the two most frequently used pavement construction materials are reused to provide a strengthened and improved pavement. The major advantages of the use of RAP are (a) lower cost, (b) reduction in use of natural resources, (c) reduction of damage to other roads from transportation of materials from a quarry site, (d) no increase in pavement thickness, very important for city streets and major highways, and (e) less dependence on added energy.

Numerous studies have reported that RAP can be reused as an aggregate in Hot Mix Asphalt (HMA) as well as in cold mix asphalt, granular base, sub-base, and subgrade courses. A large amount of literature is available on the use of RAP in HMA [2]. Research findings indicate that bituminous mixes containing RAP and a rejuvenator produce mechanical and rutting properties that are as good as or even better than those using conventional binder. Cold recycling technology, like hot mix technology, has also become popular in various countries for rehabilitation of damaged bituminous pavements. RAP stabilized with bitumen emulsion and foamed bitumen has been used as a base layer. The present study focuses on Foamed Bitumen treated Mixes (FBMs).

1.2 Problem Statement

Unlike hot mix asphalt (HMA), there is no universally accepted mix design method for FBMs. Most of the agencies which use FBMs have their own mix design procedures which are the result of numerous efforts over decades. In spite of all these efforts, Foamed Bitumen application in cold recycling in the United Kingdom suffers from the lack of a standardised mix design procedure specifically using the gyratory compactor. As a result, the mix design parameters such as Foam characteristics, mixing, compaction, curing and testing that are being adopted are far from being standardised. To overcome this, research was undertaken at the University of Nottingham by Sunarjono (2008) [3] to develop a mix design procedure by identifying critical mix design parameters.

The research by Sunarjono (2008) was focussed on the influence of the bitumen type, the foaming conditions, foam characteristics and mixer type on the mechanical properties of FBM. The major outcomes of the work were recommendations for producing an optimised FBM in terms of mixer type and usage, selection of binder type, bitumen temperature, and foam characteristics. Therefore this present study has focussed on other mix design parameters such as foamed bitumen content, Mixing Water Content (MWC), and compaction effort. The amount of water during mixing and compaction is considered as one of the most important parameters in FBM mix design; it helps in dispersion of the mastic in the mix. However, too much water causes granular agglomerations which do not yield optimum dispersion of the mastic in the mix. Because of the presence of the water phase, the compaction mechanism of FBMs is very different from that of HMA. Various laboratory compaction methods such as Marshall compaction, vibratory compaction, gyratory compaction have all been used in the past, and there are very well established guidelines for Marshall compaction and vibratory compaction. However, there are no established guidelines for a gyratory compaction method for FBMs in terms of compaction effort (number of gyrations, gyratory angle and gyratory pressure).

A lot of research has been carried out on development of a laboratory curing protocol for these mixes to assess when to carry out the test on a mixture. The curing protocol is especially important in structural capacity analysis in pavement design which is based on the laboratory measured stiffness values. Therefore, a laboratory mix design procedure needs to simulate the field curing process in order to correlate the properties of laboratory prepared mixes with those of field mixes. Simulation of the environment to which the pavement material will be exposed after compaction is the best approach. However, this approach is too complicated and time consuming as the process takes a long time and predominantly depends on the climatic condition that the material is exposed to in the field. An accelerated laboratory curing procedure which involves curing at elevated temperature is the best available option.

Various accelerated laboratory curing and conditioning regimes exist for FBMs and are used in a variety of ways by researchers and in material specifications although the underlying question remains: how realistic are these curing and conditioning regimes in simulating pavement conditions as experienced in the field? Most accelerated curing regimes fail to reproduce the actual condition of the mix that is expected in the pavement and tests are therefore conducted on specimens cured under unrealistically favourable conditions (i.e. higher temperatures). This results in overestimating the mix performance in the field. This is especially true when the fundamental properties of the FBM are compared with other mixes such as HMA and Warm Mix Asphalt (WMA). Moreover, considerable variation among different conditioning regimes is evident. Therefore it is necessary to develop a suitable curing protocol and establish criteria to evaluate how long a laboratory prepared FBM specimen needs to be cured before testing for mixture selection.

In the pavement layers, when bituminous mixtures are subjected to traffic loads, stresses are induced which are usually very small when compared to mixture strength. These stress pulses cause strain most of which is recoverable. The irrecoverable strain is very small for a single load application. However, for repeated loads which are the actual case in pavements, these small irrecoverable strains accumulate. This accumulated permanent strain induced by traffic loads manifests itself as permanent deformation in pavement layers. A number of researchers have studied the effect of different factors on permanent deformation in FBMs such as the grade of bitumen used for foaming, foamed bitumen content, active filler type and its content, RAP content and age, water content in the specimen during the test and applied stress.

From a review of the literature it was noted that, though most researchers have considered temperature as a major factor that affects the permanent deformation behaviour of FBMs little effort has been made in undertaking experimental investigation of this effect. However, a considerable amount of investigation has been carried out on mechanical properties such as stiffness and strength. In view of these findings, the present study aimed at studying the effects of the applied stress on the temperature sensitivity in terms of permanent deformation, which is essential to understanding the behaviour of FBM and to increasing confidence in pavement design using these materials.

1.3 Aim and Objectives

The primary objective of the present study is to propose a practical and consistent mix design procedure and to study the performance characteristics of FBMs. By achieving these objectives it is expected that the behaviour of FBMs with cementitious additives and RAP material will become well understood such that pavements with FBM layers can be designed with some confidence.

To achieve the aim the following objectives were considered;

1. Detailed literature review of mix design procedures that are being followed by different agencies.
2. Identifying critical mix design parameters and studying their influence on mechanical properties.
3. Understanding curing (water loss and strength gain) mechanisms in FBM.
4. Developing strength-maturity relationships for FBMs.
5. Studying fatigue behaviour and durability of FBMs with cement and RAP material.
6. Conducting mechanistic (non-linear elastic) analysis of pavements with FBM layers by using the results obtained in 5.

1.4 Thesis outline

There are **nine chapters** in all in this Thesis with this present **Chapter** being the **first**. This Chapter has 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.

Chapter 2 is a **literature review** of the previous research studies that have been conducted on FBMs. The chapter includes a brief introduction to foamed bitumen (history and production) followed by an explanation to foamed bitumen characteristics. The chapter also reviewed the parameters for mix design of FBMs and their influence on the mechanical properties and laboratory and field performance. A literature review of structural design aspects of FBMs such as failure mechanism and structural evaluation is also included in the chapter.

A **review of mix design methods** that are followed by different research and practising agencies around the world is discussed in **Chapter 3**. The review was carried out in terms of material properties (bitumen and aggregate), foam characteristics, RAP characterisation, fluid considerations, mixing, compaction and curing.

Chapter 4 in this thesis explains about the characteristics of the **materials** used in the study. Along with bitumen and virgin aggregates, special attention was paid to the characterisation of the RAP. In addition to basic tests such as recovered bitumen and aggregate properties of RAP, tests such as fragmentation and cohesion tests were also included as a part of this study.

In **Chapter 5** efforts were made to recommend a practical and consistent **mix design procedure** for FBM with the main focus being on the use of the gyratory compaction method in the proposed methodology. Results of the validation of the proposed methodology are also presented in the chapter.

Chapter 6 discusses one of the crucial mix design considerations for of FBMs, **curing.** The chapter also evaluates the applicability of the maturity method, which is commonly used to estimate in-situ compressive strength of concrete before removal of formwork, to FBMs.

Chapter 7 discusses the **performance characteristics** of FBMs in terms of their permanent deformation and fatigue life.

In **Chapter 8 structural design** charts were developed for pavements incorporating FBMs as base course material.

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

2 LITERATURE REVIEW

2.1 Foamed bitumen (FB) and FBMs

In recent years considerable experience has been gained by engineers and researchers with the construction of pavements incorporating FB treated layers. Development of this technique (bitumen foaming) has enabled an economic and environmentally friendly means of upgrading structural capacity of in-service pavements. This section overviews development FB technology and describes the microstructure formation of FB treated mixes.

2.1.1 Brief history of FB

The process of foaming bitumen was developed more than 50 years ago by Dr. Ladis Csanyi of Iowa State University. Availability of marginal ungraded aggregate and shortage of good aggregate in his state of Iowa inspired him in the invention of foamed bitumen technology. He studied different methods of producing foam from bitumen and its applicability to paving materials [4]. It was shown in his studies that producing foam from bitumen by injecting steam into bitumen was a simple and efficient technology [5]. However, this method (injecting steam) proved to be impractical for in-situ foaming, because of the need for special equipment such as steam boilers [6].

In 1968, Mobil Australia which had taken patent rights for Dr.Csanyi's process, modified the original process by injecting cold water rather than steam into hot bitumen [6]. Thus, bitumen foaming technology has become much more practical and economical. Subsequently, this technology has gained popularity in countries like Australia, Germany, New Zealand, South Africa and later in USA and UK [7].

2.1.2 Bitumen foaming process

The foaming of bitumen can be acknowledged to be a phenomenon caused by changing water from a liquid state to a vapour at high temperatures which is accompanied by an increase in volume around 1500 times at atmospheric pressure [8]. When water comes in contact with hot bitumen, the heat energy is transferred from bitumen to water. This result in water reaching boiling point and changing state and, in doing so, creating a thin-filmed bitumen bubble filled with water vapour. Accordingly, foamed bitumen is produced by injecting water into hot bitumen, resulting in spontaneous foaming in an expansion chamber [9-11]. However, the original process proposed by Dr.Csanyi consisted of introducing steam into hot bitumen through a specially designed nozzle such that the bitumen on ejection was temporarily transformed to foam. An improved production process was developed by Mobil Oil Australia Limited [6]. This improved method involves introduction of a controlled flow of water into hot bitumen (Figure 2-1). The expansion chamber was refined by the Mobil Oil organisation in the late 1960's and it is still the most commonly used system for producing foamed bitumen [10]. However, the system developed by Wirtgen in the mid-1990s injects both air and water into the hot bitumen in an expansion chamber as shown in Figure 2-2. Water (1% to 5% of the mass of bitumen) together with compressed air is injected into hot bitumen (140°C to 180°C) in the expansion chamber. This causes the water to turn into vapour which forms tiny bitumen bubbles.

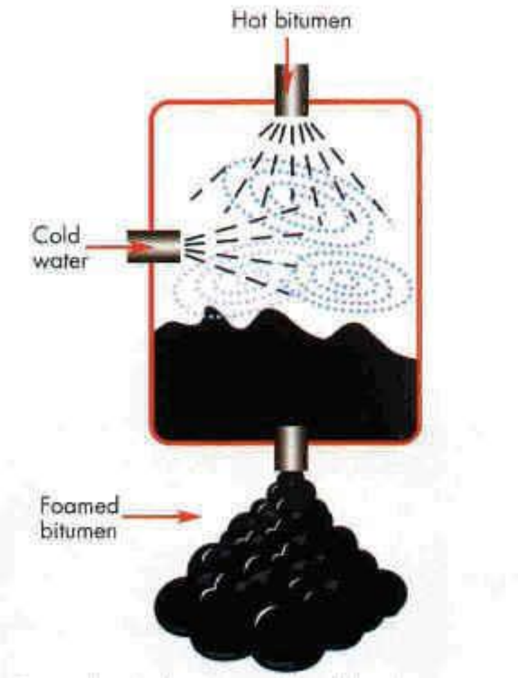


Figure 2-1 Foaming bitumen by Mobil Oil organisation technique [6]

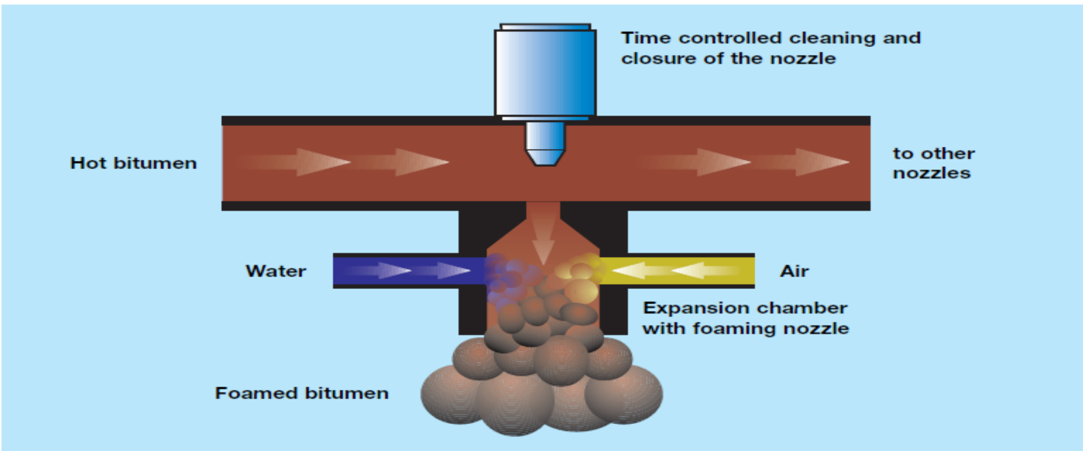


Figure 2-2 Foaming bitumen by Wirtgen [8]

2.1.3 Mixing mechanism and Microstructure of FBMs

Foaming bitumen increases the surface area of bitumen making it well suited for mixes with relatively cold and moist aggregate. To produce FBMs, the aggregate is incorporated into the bitumen while still in its foamed state. The greater the surface area of the foam, the better the distribution of the bitumen in the aggregate. An analogue to this mixing process is a baker beating the white of an egg into foam of low viscosity before mixing with flour [12, 13]. Through the beating process, the egg-white turns to air-filled bubbles of thin films occupying a far greater volume. This helps in distribution of flour in the volume and thereby making it possible to achieve a consistent mix. Particles stick together and form a paste which does not get harder immediately.

During the mixing process, the bitumen bubbles burst, producing tiny bitumen particles that disperse throughout the aggregate by adhering to the fine particles to form mastic. In un-compacted foamed asphalt mixes, the asphalt mastic phase exists in the form of isolated droplets with various sizes. Since the workable duration (before asphalt bubbles burst and lose workability) of this mixing process is as short as a few seconds, only a fraction of the mineral fillers can be coated by bitumen, while a considerable amount of mineral filler remains "free" and eventually forms the mineral filler phase. On compaction, the bitumen particles in the mastic are physically pressed against the large aggregate particles resulting in localised non-continuous bonds [12, 14]. Jenkins (2006) [7] termed the non-continuous bonds in FBMs as ''spot welding''. A conceptual description of FBM microstructure is shown in Figure 2-3.

According to Fu (2009) [14], FBMs have three phases.

1. The aggregate skeleton formed by large aggregate particles,
2. The asphalt mastic phase existing in the form of droplets bonding the aggregate skeleton together, and
3. The mineral filler phase partially filling the voids in the skeleton.

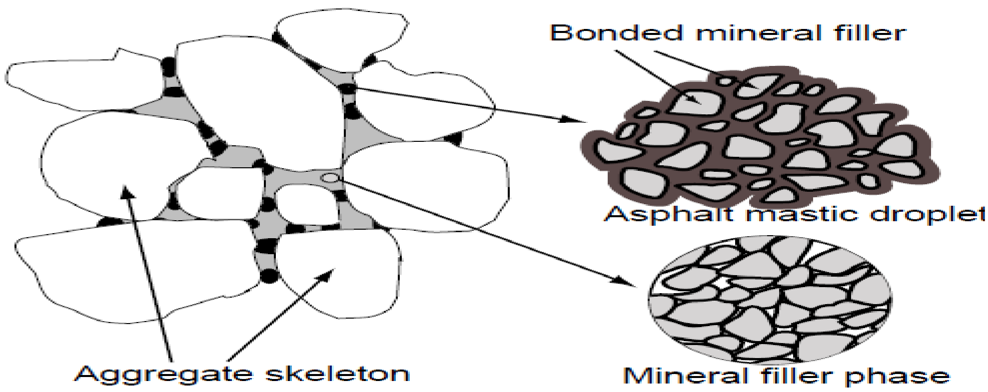


Figure 2-3 Microstructure of FBMs [14]

2.2 Foamed bitumen characteristics

Studying the use of foamed bitumen for cold-in situ recycling includes two aspects: one is to study the foamability of bitumen and the other is to study the property of the foamed asphalt mixture. Foamability (foaming potential) is the ability of binder to produce foam of superior characteristics. It is expected that foam characteristics are important for better mix performance. This section discusses different foamed bitumen characteristics and factors that affect these characteristics.

2.2.1 Maximum Expansion Ration (ER) and Half –Life (HL)

During the foaming process the foam expands rapidly and reaches its maximum volume. The ratio of maximum volume of foamed bitumen to the volume of liquid bitumen used is termed the maximum expansion ratio [15]. The expansion ratio is a measure of viscosity of the foam and indicates how well the bitumen will disperse in the mix (Figure 2-4). The time that elapses from the moment that the foam is at its maximum volume to the time that it reaches half of this volume is termed the half-life of the foam (Figure 2-4). Accordingly, expansion ratio and half-life have been used to compare the foaming characteristics of bitumens [12, 15-22].

Brennan (1983) [15] identified that expansion ratio and half-life are affected by (1) the amount of foam produced, (2) the foaming temperature of the bitumen. However, these three factors are not enough to explain the formability of the bitumen [7]. One of the dominant factors (other than the two mentioned by Brennan) influencing the foam properties is the water that is injected into the expansion chamber to create the foam. Higher application rate of water results in higher expansion ratio but leads to rapid collapse of foam i.e. shorter half-life (Figure 2-5). In the laboratory, the size of the container was found to affect foam parameters [18]. Jenkins (2000) [7] and Sunarjono (2008) [3] have done extensive research on foamed bitumen characteristics and concluded that water application rate and bitumen temperature are the most significant factors affecting expansion ratio (ER) and half-life (HR).

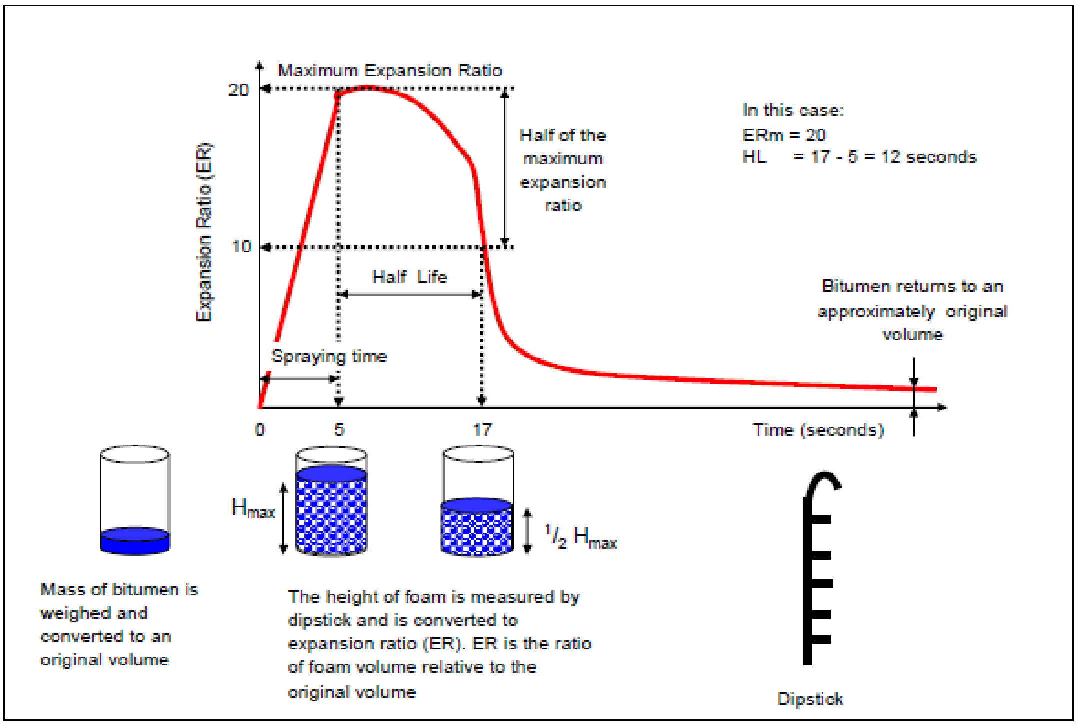


Figure 2-4 Characteristics of Foamed bitumen [8]

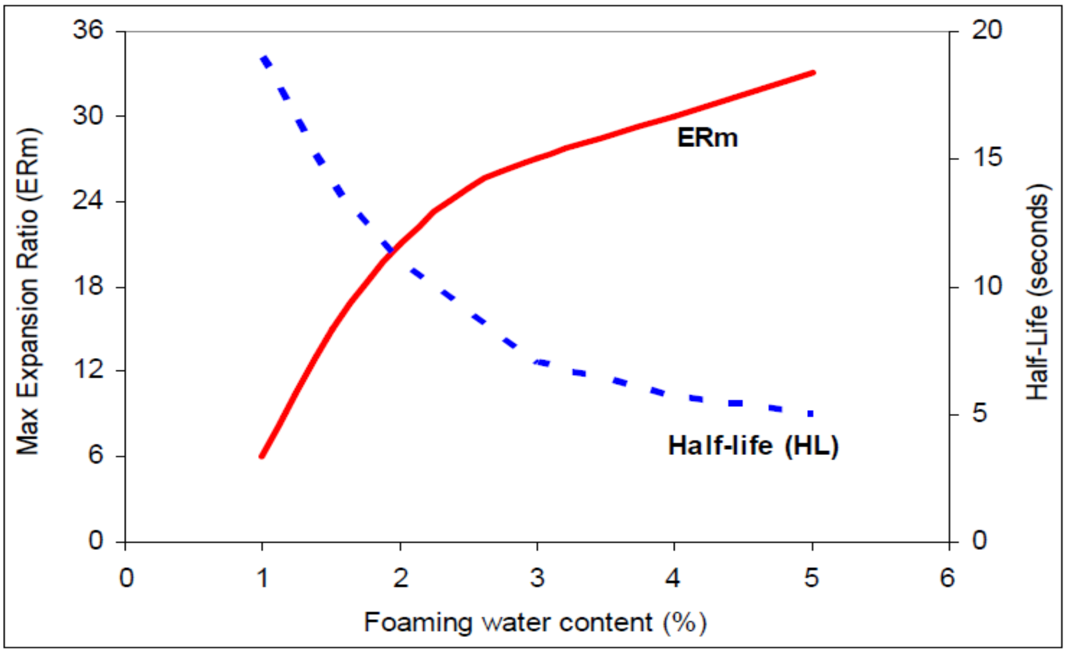


Figure 2-5 Relation between ER and HL

2.2.2 Foam Decay and Foam Index (FI)

Higher ER and a longer HL are currently understood as foam characteristics that result in better FBM properties. But, as discussed in 2.2.1 ER and HL show opposite trends and this makes the selection of optimum foam a compromise between the two. After intensive study by Jenkins et al., (1999) [12], on foam characteristics, it was concluded that these two parameters and the manner in which they are determined are insufficient for adequate characterisation of the foaming potential of bitumen. For this reason the concepts of foam decay and Foam Index (FI) were introduced.

Foam decay is the collapse of FB with time. The primary cause of foam decay is reduction of temperature due to contact of foam bubbles with the container and aggregates of relatively lower temperature. A decay curve records the change of the expansion ratio with time (Figure 2-6). The decay curves are a function of both ER and HL. If the decay function of a FB is obtained, its ER and HL can be calculated from the decay function. Jenkins et al., therefore developed a foam decay model by adapting equations for isotope decay. He and Wong (2006) [23] carried out foam characterisation to investigate decay properties of bitumen. A four-parameter power function and a three-parameter exponential function were selected to simulate the bitumen's decay properties. It was found that water content has a significant effect on bitumen decay.

Jenkins et al., (1999) [12] intoduced the FI concept to describe the foaming potential of the bitumen. FI was defined as the area under the decay curve (Figure 2-6). The area is calculated for the purpose of optimising the foamant water application rate. In general, it is expected that a maximum FI occurs at an optimum FWC giving optimum foam quality. Finally, it was proposed to select bitumen based on the value of the FI.

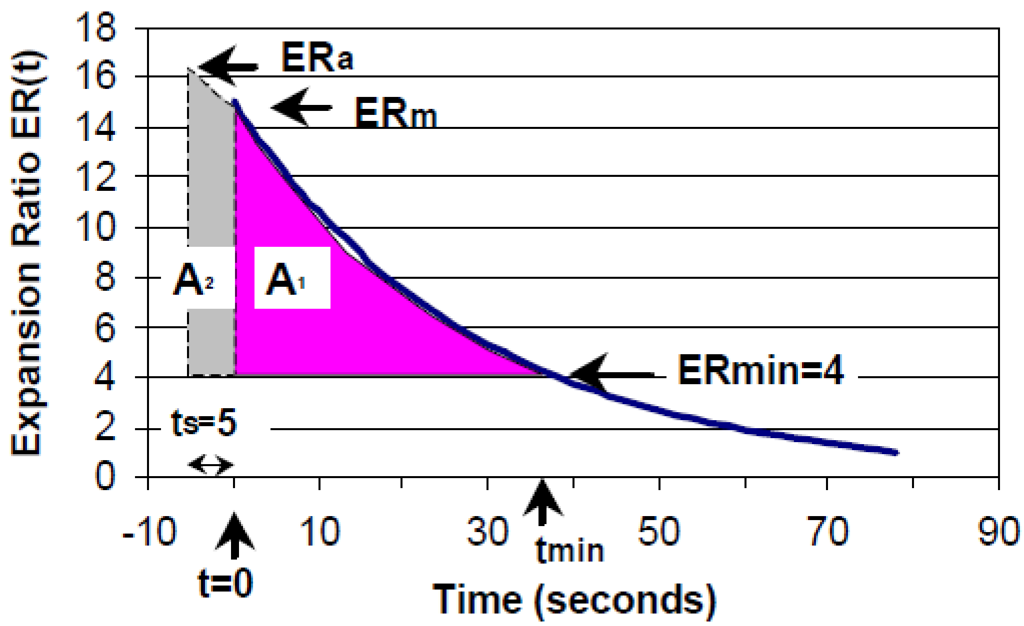


Figure 2-6 Foam decay and FI (A1+A2) of a bitumen [12]

2.2.3 Viscosity

Lesueur et al., (2004) [24], He and Wong, (2006) [23], Saleh (2006) [25] and Sunarjono (2008) [3], found that HL values decrease up to a certain FWC and then remain constant. Such a trend results in FI values increasing continually with increasing FWC and hence an optimum FWC value becomes impossible to locate.

Saleh (2006) [25] proposed an alternative method in which foam viscosity was measured using a Brookfield rotational viscometer at several times, and an average foam viscosity over the first 60 seconds was calculated. The relationship between FWC and average foam viscosity was plotted and the minimum viscosity value was selected as the best foam composition. He also found that foams that were categorised as poor performing, according to their FI values, can still be effectively mixed with aggregates. It was thus proposed that the ability of foam to form a well coated asphalt mix was directly related to its viscosity.

2.3 Mix design considerations

As discussed in 2.1.1, the bitumen foaming process dates back to 1957 when Dr. Ladis Csanyi first injected steam into bitumen. Since then the FB production process has expanded to a diversity of countries and research has been undertaken on a variety of materials in various climates. In this section, available literature on different mix design considerations is reviewed together with factors that are affected by these considerations.

2.3.1 Bitumen properties

Bitumen with penetration values between 80 and 150 is generally used for foaming. Although harder bitumens that meet the minimum foaming requirements have been successfully used in the past [8], for practical reasons harder bitumen is generally avoided as it produces poorer quality foam, leading to poorer dispersion.

Unlike Hot Mix Asphalt (HMA) mix design in which climate is a major factor of consideration in selection of binder, in FBM mix design one has to consider the foamability (foam potential) of the binder as well. Foam potential is the capability or potential of bituminous binder to produce good foam characteristics [15]. As discussed in 2.2 expansion ratio and half-life are characteristics of foam (in this case foamed bitumen), not the properties of bitumen. So identifying the right bituminous binder with good foam potential is an important task.

Foamability of bitumen was found to be a primary factor determining FBM quality [26]. Studies on the effect of bitumen properties on FB properties were carried out by Jenkins (2000) [7], Saleh (2006) [21], Sunarjono (2008) [3]. Most of these studies concluded that bitumen with better foam potential tends to yield better quality FBM. Referring to Castedo et al., (1982) [27] any binder irrespective of grade can be used with an appropriate combination of nozzle type, water, air and bitumen injection pressure. Abel (1978) [28] stated that temperature of the bitumen during foaming is an important factor to be considered during the foaming process. A given bitmen can be foamed under different temperatures to produce foam with different characteristics [27-29]. Moreover bitumen of the same grade from different sources may not always give the same characteristics for the same foaming conditions (Table 2-1). Therefore, one batch of bitumen having excellent foaming characteristics does not necessarily mean bitumen with the same brand and grade will always give comparable characteristics [26].

Table 2-1-Effect of bitumen source on foamability [26]

Bitumen ID	Foam	PG grade	G*/sinδ	Foaming temp. (°C)	Foaming water content (%)	Expansion ratio	half-life (Secs)
A	A-1	64-16	4632 at 28°C	158	4	24	20
A	A-2	64-16	4632 at 28°C	149	3	20	30
B	B	64-22	3276 at 25°C	145	3	11	25
C	C	64-16	3760 at 31°C	158	3	6	6

In general it has been assumed that softer bitumen types and higher bitumen temperatures generally produce better foam quality. However, experiments conducted by He and Lu (2004) [22] on the foaming potential of bitumen grades commonly used in road construction in China concluded that softer bitumen and higher temperature of bitumen do not always give superior foam characteristics. He and Wong (2006) [23] supported this statement (Figure 2-7). A bituminous binder with good foam potential can produce foam with inferior foam characteristics if the foam conditions are not optimised. Consequently, in FBM design it is not enough to select binder with good foam potential but also selection of the optimal foaming conditions for making good foam out of the selected binder is important.

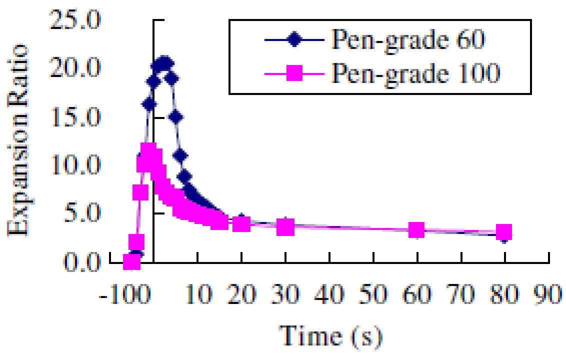


Figure 2-7 Foam potential of two different bitumen grades[23]

2.3.2 Foaming Water Content and Temperature

The viscosity of bitumen has an inverse relationship with temperature; as the temperature increases, its viscosity reduces. Since the foaming process draws heat energy from the bitumen, the temperature before foaming typically needs to be more than 140°C (depending on binder grade) to achieve a satisfactory product.

Csanyi (1960) [30] had mentioned that bitumen temperature affects the foam production. An increase in the water rate also had the same effect as that of temperature [3, 11, 15, 19, 31]. Brennan (1983) [15] recommended a foaming temperature of 160°C and water content of 2% for optimum expansion ratio and half-life. Maccarone et al., (1995) [32] achieved optimum FB characteristics by adding 0.7% of additives at 2.6% water content. Nataatmadja (2001) [33] concluded that the foam water content generally ranges between 2 and 2.5%. However, optimum water content can be possible outside this range [3, 23, 31].

2.3.3 FB characteristics

In his study on FB characteristics Professor Csanyi (1960) [30] mentioned factors affecting foam production such as nozzle tip dimensions, nozzle adjustments, the asphalt temperature, the relative pressure of bitumen and steam. He was silent about the type of bitumen to be used for foaming and there were no recommendations to judge whether FB was satisfactory except by visual examination of aggregate particle coating. Qualitative characterisation of FB first resulted from a Mobil Oil study [6] in Australia. In their work quality of foam was characterised by half-life and expansion ratio. Later quantitative recommendations were also given by other studies [11, 16, 18].

In research carried out in Australia [16] in collaboration with Mobil Oil numerous FBMs were produced and tested. This research found that FB characteristics influence cohesion, stability and unconfined compression strength (UCS) of the FBMs and these properties of mixes were significantly improved when high ER was used. In this research, the authors used ER from 3 to 15 and the influence of ER on these properties was evaluated. Maccarone et al., (1995) used active additives in the FB production process to obtain high expansion ratio and longer half-life [32]. This high expansion and half-life of FB resulted in improved aggregate coating and hence in improved mix properties. However, the influence of FB characteristics on the FBM properties was not supported by a few other researchers [3, 17]. This contrary evidence, which did not find any difference between the measured properties of FBMs produced with different grades of bitumen, is probably related to the fact that much of the shear strength of FBMs is due to aggregate interaction rather than bitumen cohesion.

At Iowa State University [17], the effect of foam half-life and foam ratio (ER) on Marshall stability was studied. From experimental data it was revealed that there were differences between FBMs of high foam ratio (ER=20) and low foam ratio (ER=5). However, there were no differences between mixes of high (HL=139 sec) and low (HL=11 sec) half-lives. In research at the University of Nottingham [3], the relation between ER/HL and ITSM was analysed. This study concluded that FB characteristics were not the major influence factors on mixture properties. Instead, mixing protocol and binder type were found to have a more dominant effect (Figure 2-8 and Figure 2-9).

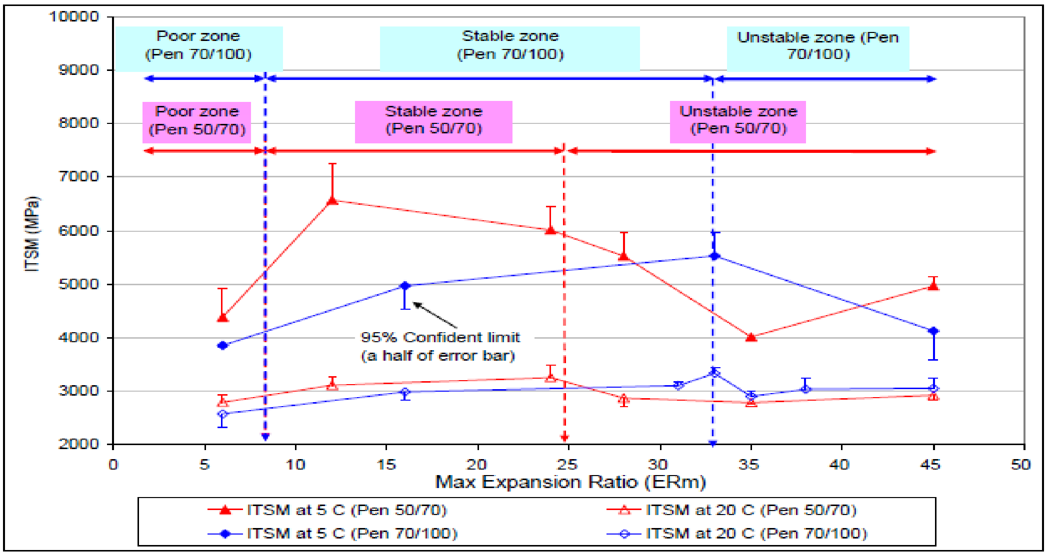


Figure 2-8 Effect expansion ratio on ITSM [3]

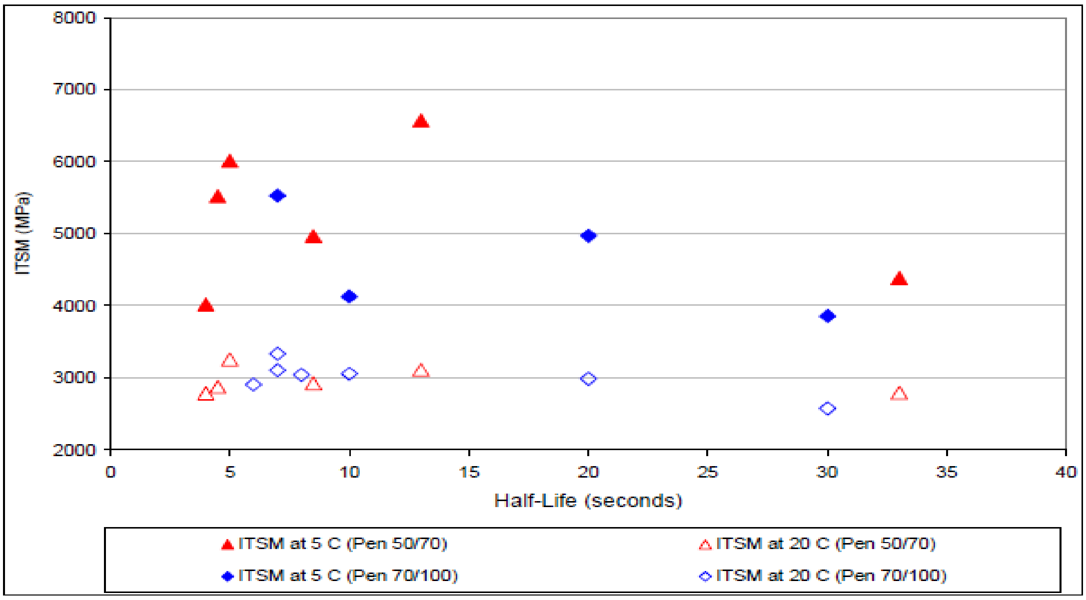


Figure 2-9 Effect of half-life on ITSM [34]

Fu et al., (2011) [26], using fracture face image analysis (FFIA) studied effect of FB characteristics, the mastic dispersion ability and mastic tensile strength in FBMs. FFIA was used as a quantitative indication of the bitumen dispersion quality. Higher Fracture Face Asphalt Coverage (FFAC) values generally represent better dispersion of bitumen mastic (uniformly distributed). The investigation proved that FBM produced with better foamability (high HL and ER) should have better dispersion characteristics and higher strength if all other variables are fixed (Figure 2-10 and Figure 2-11 and Table 2-1), where A, B, C are three different binder grades, A-1 and A-2 are of same binder grade from different refineries and State Route (SR)-33 and SR-88 are pulverised asphalt pavement material from two different sources.

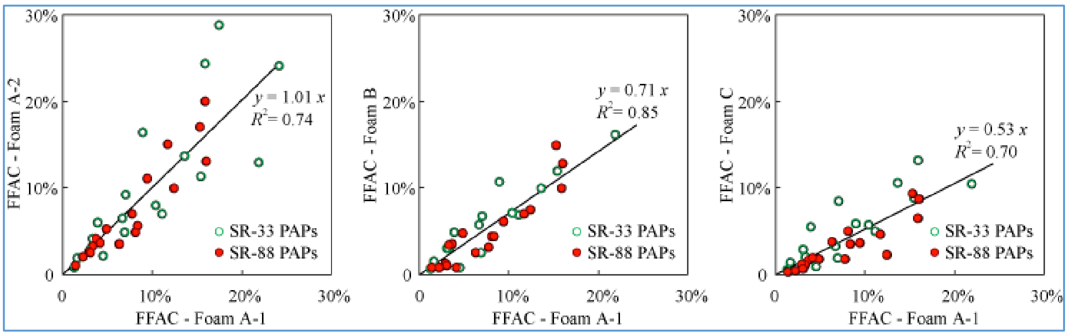


Figure 2-10 Effect of foam characteristics on mastic dispersion [26]

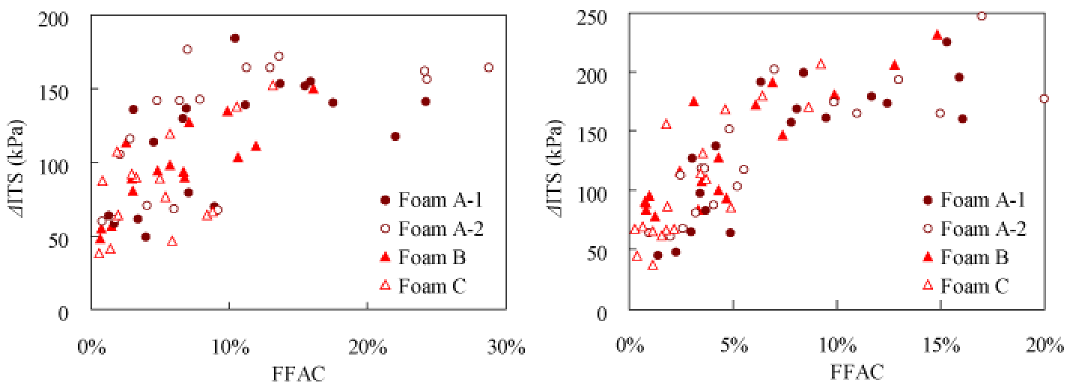


Figure 2-11 Effect of mastic dispersion on mix strength [26]

Apparently, the first work that discussed the importance of FB characteristics was work done by Mobil Oil Australia [16]. In their work, they recommended ER between 10 and 15 as effective FB characteristics. Later, Ruckel et al., (1982) [18] proposed limits of 8-15 for ER and a minimum HL of 20 seconds. These specifications were followed by many other researchers in their studies [35]. However, CSIR in South Africa recommended minimum limits of 10 ER and 12 seconds half-life [10].

2.3.4 Aggregate properties

2.3.4.1 Aggregate type

Previous research has shown that a wide range of aggregate can be used with FB ranging from crushed stone [12, 18, 21] to sand [35, 36]. Studies [15, 17] have demonstrated the feasibility of using marginal aggregates to produce foamed mixes with or without virgin materials. However, laboratory results reported by Little et al., (1983) [37] which used marginal aggregates in laboratory mixing of FBMs suggested that those FBMs have lower stability and poorer fatigue performance in comparison to FBMs with gravel. Bowering and Martin (1976) [16] tested FBMs comprised of 50 different materials and categorised and ranked them for use in FBMs (Table 2-2).

Table 2-2 Ranking of suitability for FB treatment [16]

Material type	Binder content (%)	Additional requirements
Well graded clean gravel	2 - 2.5	-
Well graded marginally clayey gravel	2 - 4.5	-
Poorly graded marginally clayey gravel	2 - 3	-
Clayey gravel	4 - 6	Lime modification
Well graded clean sand	4 - 5	Filler
Well graded marginally silty sand	2.5-4	-
Poorly graded marginally silty sand	3-4.5	Low pen bitumen; filler
Poorly graded clean sand	2.5-5	Filler
Silty sand	2.5-4.5	-
Silty clayey sand	4	Possible lime
Clayey sand	3 - 4	Lime modification

2.3.4.2 Aggregate gradation

Saleh (2006a) [21] investigated the effect of aggregate gradation on resilient modulus. It was clear from his results that aggregate gradation has a significant effect on engineering properties of FBMs. Mobil Oil Australia [38] from their experience established guidelines for suitable gradations for FB stabilisation. Figure 2-12 shows aggregate grading zones of suitability. Zone A is the envelope showing ideal material and zone C suitable only after adding lime and filler.

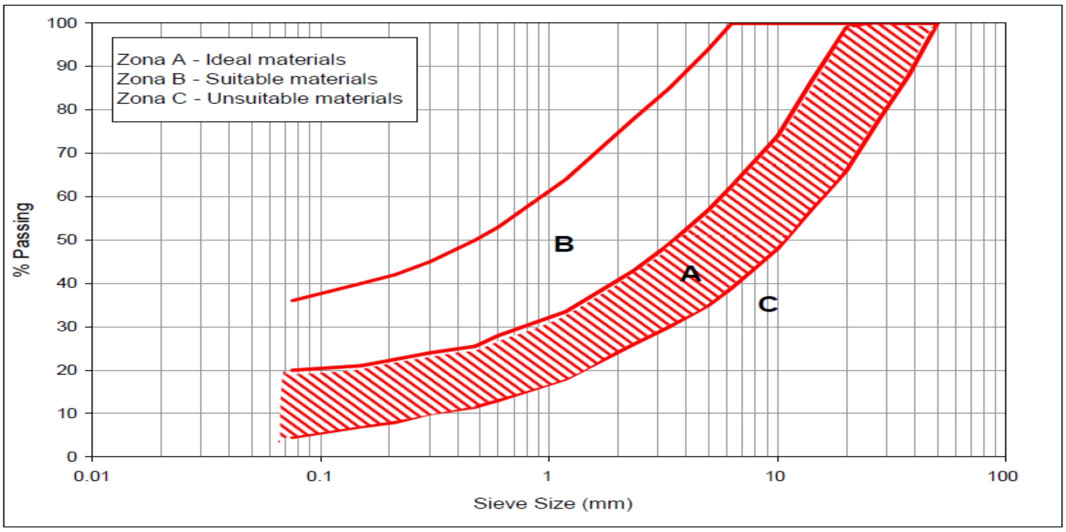


Figure 2-12 Suitability grading envelopes for FB stabilisation [38].

2.3.4.3 Fines

The importance of fines (passing through the 75 micron sieve) has been well documented in the literature. Materials with a relatively large percentage of fines have been considered better because the bitumen tends to coat the fines fully and only partly coat the larger aggregate [28]. Acott (1979) [39] also recommended adding of fines to improve the stability of FB treated sands. The presence of fines enhances the ability of the foam to produce better mix by forming mastic (bitumen + filler) [40]. Mastic has a significantly higher viscosity than raw bitumen; it acts as a mortar between the coarse aggregate particles and hence increases mix strength. Ruckel et al., (1982) [18] recommended that the fine aggregate content should be above 5%; whereas Maccarrone et al., (1994) [19] suggested a minimum of 8% fines. In a study in Oklahoma [41] on FBMs, it was found that mixes with a higher percentage of fines had higher stability. The same was supported by other researchers [19, 36]. This argument seems likely to be true, because foam bonds with fines to form mastic. So, more mastic will be formed if more fines are available. However, this argument has limitations [26]. Mixes with a higher percentage fines tend to have low soaked strength (Figure 2-13). The phenomenon can be explained by the presence of a continuous mineral filler phase.

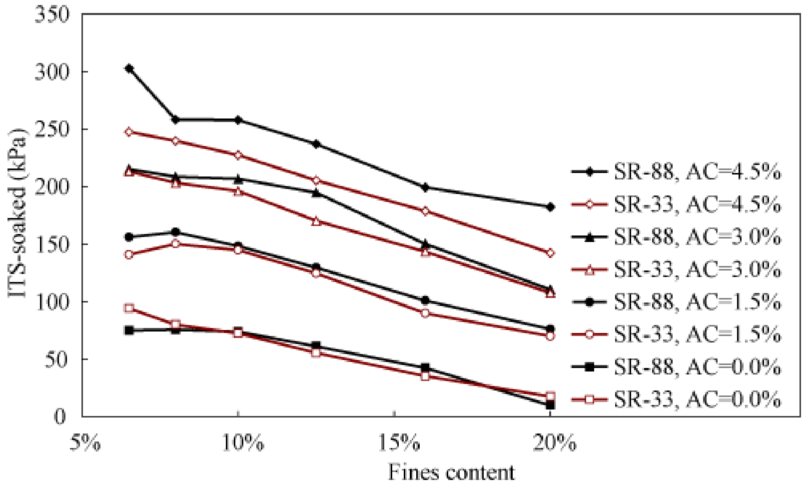


Figure 2-13 Effect of fines content on soaked strength[26]

2.3.4.4 Others

Sark and Manke (1985) [41] in their study found that, alongside percentage of fines, angularity of aggregate and plasticity index (PI) were also found to be significant factors affecting foam mix behaviour. The dominant effect of aggregate interlock on behaviour implied that FBMs are not as temperature susceptible as hot mix asphalt (HMA). Able (1978) [28] had indicated the importance of low plasticity of the material, while Sark and Manke (1985) [41] suggested a minimum PI of 10 to achieve a good and stable mix.

2.3.5 Secondary binder

It is a common practice to add active filler to the mix in conjunction with FB to achieve optimum performance. Bonding of bitumen in FBMs takes weeks to months to develop full bond where cementitious materials take a few hours after hydration. Therefore, addition of cement offers the capability to handle early traffic on pavements with FBM layers. Moreover, the tensile strength provided by active filler is much less water susceptible than that of the mineral filler phase. It may be possible that high cement content negates the flexibility offered by FB to the mix if maximum stiffness or maximum strength is used as the mix design criterion. Thus, the strategy has to be to optimize the use of bitumen and cement separately.

The types of active filler used in FBMs are cement [42-44], lime [11, 43, 45, 46] and fly ash [42, 43]. Different researchers have added active filler for different purposes such as to adjust the gradation [21], for early strength gain [19, 44], or to reduce the water sensitivity [47, 48]. Hodgkinson and Visser (2004) [42] evaluated the influence of active filler (cement, lime, fly ash) and inactive filler (rock flour) on dry and soaked ITS of FBMs. The results showed that the active filler had a dominant effect on ITS values (Figure 2-14). An increase in soaked strength with cement addition, which is an indicator of water resistance, was further confirmed [47]. Addition of cement helps in attaining early strength of mixes which occurs due to accelerated curing [19]. Further researchers [44, 49] also studied the mechanical behaviour of active filler added to FBMs mixes. These studies proved the increase in stiffness with active filler addition (Figure 2-15). Moreover, addition of cement caused linear stress-strain behaviour while mixes with natural filler behaved non-linearly

[50]. However, high demand for water by mixes with cement addition was reported by Khweir (2007) [44]. Treatment of FBMs with cement also showed improvement in performance characteristics [43, 51]. FBMs showed better resistance to permanent deformation [43, 52]. However, fatigue behaviour of mixes with cement addition is still questionable as mixes become brittle and prone to cracking, further studies would appear to be needed.

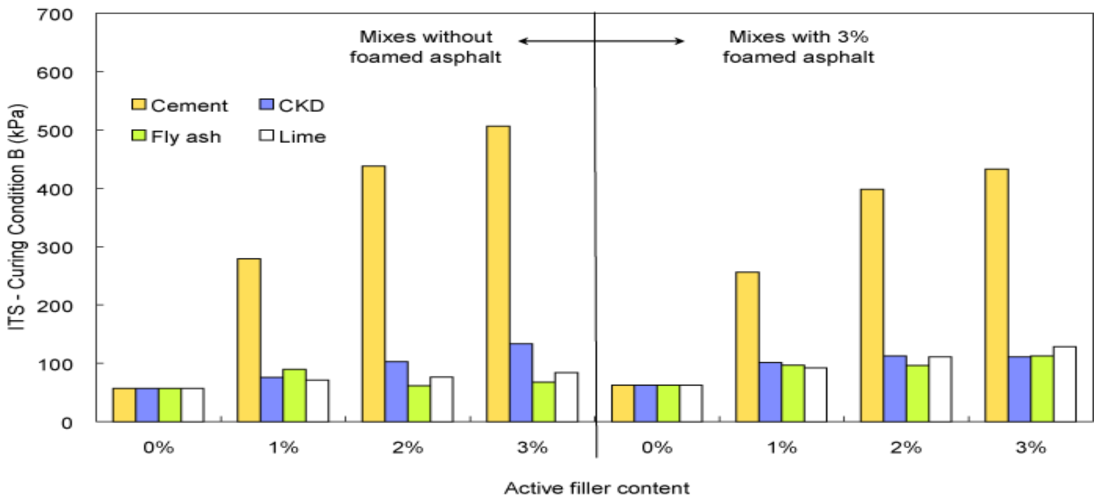


Figure 2-14 Effect of active filler on mix strength [43]

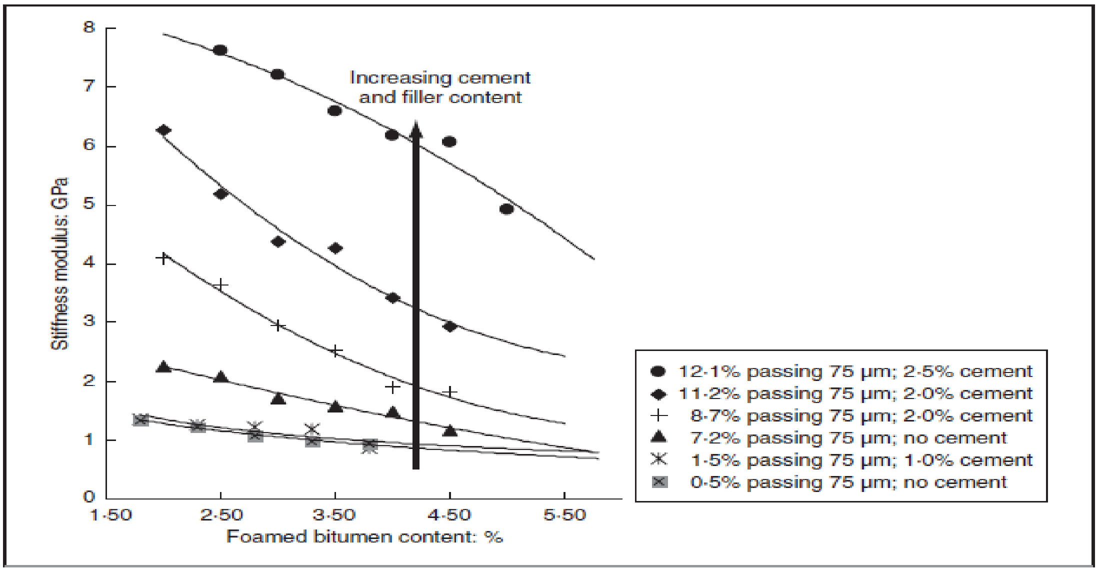


Figure 2-15 Influence of cement on FBMs [44]

Although active filler addition improves performance of FBMs, the amount of cementitious additives should preferably be a maximum of 2% by mass of the mix to minimise the potential of shrinkage cracks [53]. Moreover, care should be taken when pre-treating with cement since the hydration process commences as soon as the dry powder comes into contact with water, binding the fines and effectively reducing the fines fraction. The quality of the mix when FB is subsequently added may then be poor due to insufficient fines being available to disperse the bitumen particles. Cement should therefore always be added in conjunction with the foamed bitumen.

2.3.6 Mixing water content

The water content during mixing and compaction is considered as the most important mix design criterion for FBMs. Both Professor Csanyi's original work [4, 54] on FB stabilisation and studies in Australia [55, 56] showed the need for mixing water in the aggregate before addition of FB. Mixing water helps in dispersion of FB in the mix and to coat the aggregate during mixing [15]. It also acts as a lubricant during compaction to achieve maximum density [56]. Insufficient water reduces workability of the mix and causes inadequate compaction. Nonetheless, too much causes delayed curing and reduced early strength and also density of the compacted mix. However, there has been no unique recommendation for the quantity of added of water such that FB would coat well and adhere to aggregates.

In Csanyi's experiment the mixing water content ranged from about 6% to 10% of total aggregate weight. Csanyi did not suggest methods that could be used to determine sufficient water other than visual examination of the trial mixes, nor did he relate this water content to Optimum Water Content (OWC). Mobil Oil Australia had suggested the ''fluff point'' of aggregate, the water content that gave the material its maximum loose volume. This is approximately 70% to 80% of OWC as determined by the Modified Proctor Density test [57, 58]. However, the fluff point concept of mixing water content is not completely adequate [59]. Bowering, observed that where inadequate foam dispersion occurred as a result of insufficient mixing water, the compacted densities were low and little benefit was gained from the foamed bitumen treatment. Fluff point water content is far below the OWC for compaction.

Lee (1981) [17] investigated the effect of mixing water content on Marshall stability. In this study it was found that the optimum MWC varies with type of material (% of fines), and ranges from 65% to 85% of OWC determined by the Modified Proctor Density test. The same results were also found in an investigation by Bissada (1987) [36].

The concept of optimum fluid content was later borrowed from emulsion mix design in which the sum of the water and bitumen content should be close to OWC [11, 48]. This concept considers the lubricating action of the binder in addition to that of water. Thus the actual water content of the mix for optimum compaction is reduced in proportion to the amount of binder incorporated.

Sakr and Manke (1985) [41] developed a relation to calculate the water content for maximum density of FBMs (Eq. 2-1). The equation considered the modified AASHTO OWC, percentage of fines (PF) of the aggregate, and bitumen content (BC).

$$MWC\ (\%) = 8.92 + 1.48OWC + 0.4PF - 0.39BC \qquad \text{Equation 2-1}$$

The relation given by Eq. 2-1 suggests that, for a given aggregate used in mixing, the higher the bitumen content the lower the required water content for compaction (MWC). The optimum water content for mixing is approximately 10% to 20% higher than the MWC, as predicted by Eq. 2-1 [11]. In order to prevent the time consuming task of drying the mix after mixing, it was suggested that the MWC can be used for both mixing and compaction as no significant differences were observed in mix properties when this procedure was adopted [41].

2.3.7 Mixing

Foamed bitumen begins to collapse rapidly once it comes into contact with relatively cold aggregates. Therefore, the mixing process should be a dynamic one and consequently is most often applied directly from the laboratory foaming plant to the aggregate as it is being agitated in the mixer.

As different mixers can produce up to 25% difference in strength [10], selection of an appropriate mixer is very important in the production of FBM. It is always recommended to utilise a mixer that simulates site mixing. From the literature it was found that most of the research was carried out using a Hobart type mixer (blender type). Pug mill drum mixers and milling-drum mixers are the most commonly used mixers on site for the production of FBMs. These mixers provide sufficient volumes in the mixing chamber and energy of agitation to ensure better mixing [7]. Hobart mixers which are blender types do not simulate this [60]. Moreover, they are not ideal for restricting particle segregation. Imperatively therefore a pug mill type mixer is recommended for production of FBM that is representative of the field [61].

Mixing time should be in accordance with the time required by bitumen foam to collapse. Therefore, half-life is an important factor to be considered in the mixing process. The more the half -life the better the mix that is predicted. A half-life of more than 60 seconds can be achieved with addition of foamant [32], but this approach is seldom used. In the laboratory a mixing time of 60 seconds is recommended [36] which is longer than in situ mixing but simulates the difference in the energy of the laboratory mixer and field plant. Ruckel et al., (1982) [18] stated that if the mix darkens appreciably in comparison with the moist aggregate colour, the mixing time has to be shortened. Over mixing was noted to cause drying of the mixture and formation of balls of fines in the mix.

It was noted that FB coats finer aggregate fully and coarse aggregate partially. Ruckel et al., (1982) [18] suggested division of the aggregates into a coarser fraction(> 4.75mm) and a finer fraction(< 4.75mm). The finer fraction was first mixed in the mixer with FB, followed by the coarser fraction. This work was taken a step further by Maccarrone et al., (1994) [19] in which the finer and coarser fractions were treated separately. The finer fraction was treated with FB while the coarser was treated with emulsion, to provide a composite mix that resembles HMA with complete coating.

2.3.8 Compaction

As density achieved is crucial to the ultimate performance of the mix, special attention needs to be paid to compaction. Because of the presence of the water phase in FBMs the compaction mechanism differs from that of HMA. In FBMs compaction promotes adhesion of the bitumen mastic to the stone, improves particle contacts and reduces voids. A possible compaction mechanism is illustrated in Figure 2-16. With compaction, two adjacent aggregate particles come closer and mastic between the two particles is forced to extend along the gap between them resulting in less voids and high density [14]. According to this mechanism, higher densities can be achieved by more compaction effort and thus higher strength (Figure 2-17).

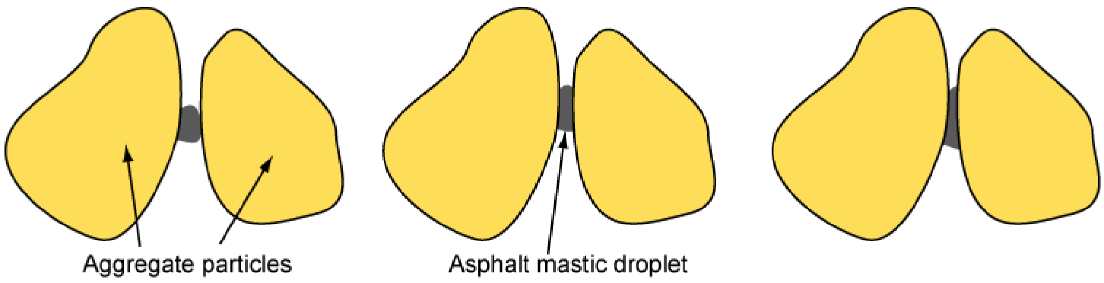


Figure 2-16 Compaction mechanism[14]

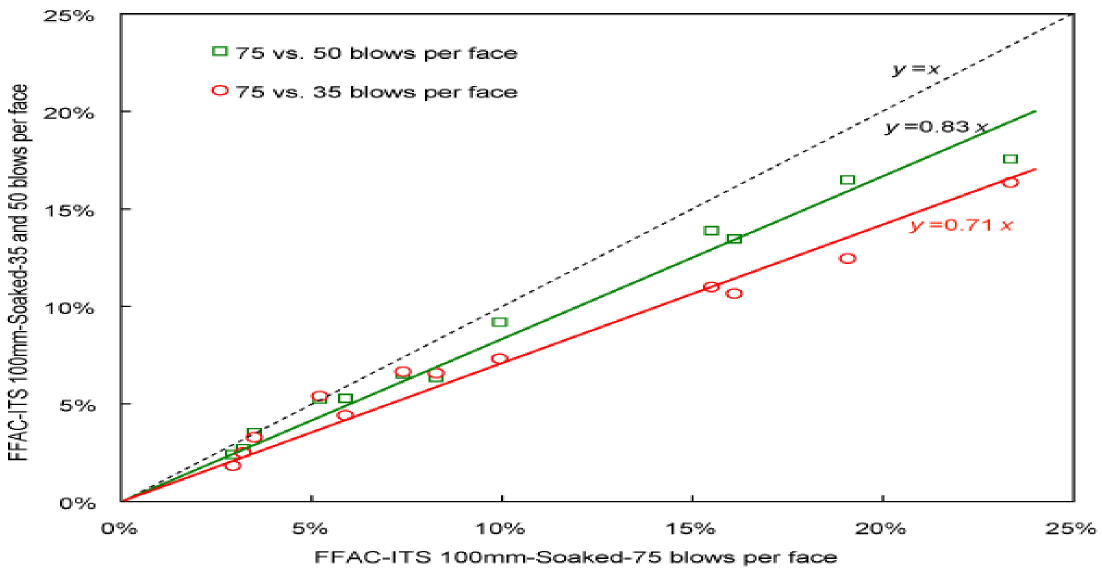
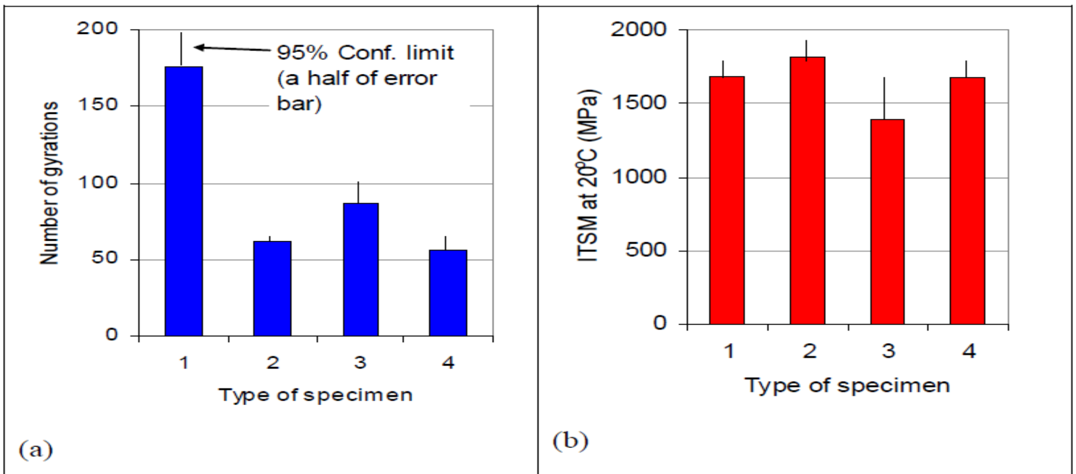


Figure 2-17 Effect of compaction level on mix strength [14]

Various laboratory compaction methods have been used such as Marshall compaction [11, 15], vibratory compaction [7, 16, 62], gyratory compaction [3, 15, 19, 46]. Brennan (1983) [15] compared performance of FBMs compacted using the gyratory and Marshall compaction methods. The Marshall Stability values obtained by gyratory compaction were twice the values obtained by Marshall Compaction that indicated that gyratory compaction is more suitable for compacting mixes than the Marshall Method of compaction. The research also found that 75 blows of Marshall Compaction were insufficient to simulate field compaction of FBMs. In addition, Brennan showed that the maximum stability occurred at the least flow for the gyratory compacted mixes while maximum stability and least flow did not coincide for the Marshall compacted mixes. Maccarrone et al., (1994) [19] recommended compaction conditions for gyratory compaction (Table 2-3) which were said to be simulative of compaction levels achieved in the field. The values cited in the table are consistent with the Australian approach to gyratory compaction. These were given earlier by SHRP work on HMA. The compaction conditions seems to require major adjustments [7]. Sunarjono (2008)[3] evaluated the compactibility of FBMs with different configurations (Figure 2-18). Results indicated that the compaction mode did not significantly affect the stiffness values as long as the final densities were comparable (Figure 2-18). However, this was not the case for Permanent Deformation for which the number of gyrations applied in the compaction process might be expected to affect the particle arrangement.

Table 2-3 Gyratory Compaction recommended conditions [19]

Mould Diameter(mm)	Gyratory compaction condition
100	150 cycles, 2° and 240 kPa
150	150 cycles 1, 3° and 540 kPa



1 – Compacted at 600 kPa, angle 1.25°	3 – Compacted at 800 kPa, angle 1.25°
2 – Compacted at 600 kPa, angle 2.0°	4 – Compacted at 800 kPa, angle 2.0°

Each result is an average of two specimens. Specimens were produced using 20 mm graded limestone aggregates and 4% bitumen (% of aggregate mass). Foamed bitumen was generated at FWC 2% and temperature of 150°C. Materials were mixed using dough hook agitator. All specimens were compacted to obtain a target density of 2242 kg/m^3.

Figure 2-18 Compaction configurations and their effect on ITSM [3]

In recent years the gyratory compactor has gained popularity for the preparation of samples. The standard Superpave protocol can be used in which compaction is set to 30-gyrations, ram pressure 600kPa and compaction angle 1.25 degrees. Jenkins (2002) found that applying 150 gyrations using the Superpave protocol will produce a specimen density approaching 100% modified Proctor compaction. It is recommended to observe the number of gyrations before compaction because this is affected by material type.

2.3.9 Curing conditions

Curing is the process in which FBMs lose their water content at elevated temperatures. Bowering (1970) [59] found that FBMs gain their full strength only if they expel a large amount of their mixing water content. Lee (1981) [17] from his limited experimental data stated that the gain in Marshall stability was accompanied by loss of water and water loss occurred more rapidly at higher temperature. Thus one can ascertain that during the curing process, the FBM gradually gains strength over time which is accompanied by a reduction in water content. From the literature [16, 39] it was found that pavements with FB treated layers exhibited premature distress in days rather than in weeks or months after construction indicating the need for expulsion of water to ensure satisfactory performance of pavements with FBM.

Ruckel et al., (1982) [18], concluded that the sample water content was the most important parameter affecting mix strength. Therefore, a laboratory mix design procedure needs to simulate the field curing process in order to correlate the properties of laboratory prepared mixes with those of field mixes. An accelerated laboratory curing procedure which means curing at elevated temperature is the best available option. This characterisation is especially important in structural capacity analysis of pavement design which is based on laboratory measured strength values [11].

From the literature it was found that temperatures of 40°C or 60°C have usually been used to accelerate the laboratory curing process. Most of the previous researchers [11, 32, 39, 63] adopted 60°C curing temperature, which had been proposed by Bowering (1970) [59]. However, Ruckel et al., (1982) [18] expressed his concern over a curing temperature of 60°C which is above the softening point of many bitumen grades. This high temperature may cause a change in mix properties, which is not desirable. Ruckel et al., (1982) [18] recommended curing at 40°C for 1 day which simulates intermediate curing (7 to 14 days) and at 40°C for 3 days which is for long term curing. Jenkins et al., (2004), and Marquis (2003) [64, 65] adopted this method of laboratory curing at 40°C for 3 days. The research in this thesis will follow this approach.

In addition to time and temperature, Maccarrone et al., (1994) [19] stated that active filler addition also influence the curing. From his research it was found that 2% cement addition helped in gaining 80% of full strength which usually takes 30 days without cement. Bowering and Martin (1976) [16] found that an increase in conditioning temperature from 40°C to 60°C had no effect on mix stability. However, Roberts et al., (1984) [40] found increases in ITS values when the conditioning temperature was increased from 23°C (75°F) to 60°C (140°F) (Figure 2-19). Kim et al., (2007) [66] reported that FBMs cured at 60°C for two days showed significantly higher ITS values than those cured at 40°C for three days.

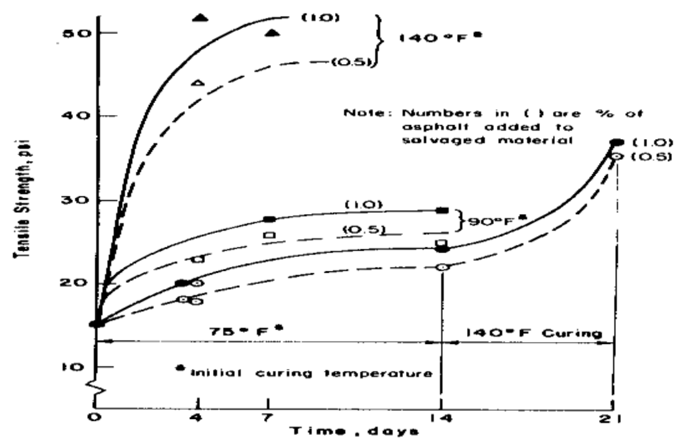


Figure 2-19 Effect of curing time and temperature on mix strength [40].

2.3.10 Engineering properties for optimum binder content

It is common practice to determine optimum binder content by optimising strength (or) stiffness parameters such as Marshall Stability, CBR, resilient modulus or unconfined compressive strength (UCS). As FBMs contain relatively low binder content and high voids, their strength characteristics are highly water dependent. Therefore, a parametric test is usually accompanied by a water susceptibility test of the parameter. An overview of the types of tests utilised for FB research and the performance properties that were identified by Jenkins (2000) [7] is presented in Table 2-4.

Table 2-4 Test methods on FBM [7]

Performance Property	**Engineering Property**	**Test**
Workability	Cohesion	Vane Shear
Fracture resistance, Fatigue resistance	Tensile strength, fracture energy, cohesion, tensile stiffness	Indirect Tensile Strength(ITS), Hveem Cohesiometer,
Permanent deformation resistance	Plastic deformation, shear strength, stability	Static Creep, Dynamic creep, Tri-axial, Hveem Cohesiometer, Vane Shear, Marshall stability, Hveem resistance
Load spreading and Stress distribution	Resilient modulus (or) Stiffness	Indirect Tensile Stiffness Modulus (ITSM), Dynamic (or) Static tri-axial
Water susceptibility	Retained strength, stability, stability (or) stiffness after water exposure	Marshall Stability, ITS, ITSM, Tri-axial
Crushing resistance	Compressive strength	Unconfined Compressive Strength (UCS)

The Marshall method was the most commonly used mix design method till the early 1990's [15, 17, 38]. Later this shifted to methods involving strength and stiffness characteristics. Some researchers [11, 19, 31, 32] recommended that optimum binder content (OBC) should correspond to the highest ITS value, whereas others [16, 67] recommended UCS. Selection of OBC based on the relation between resilient modulus and foamed bitumen content has also been proposed [19, 33, 53, 68]. However, 100ms loading rise time which is common in determination of resilient modulus of HMA was felt to be too harsh on FBMs and a rise time of 50ms was recommended [19, 53]. Researchers proposed minimum acceptable criteria for the design engineering properties. Minimum acceptable values proposed for different tests are tabulated (

Table **2-5**). More complex material characterisation tests such as dynamic modulus test, dynamic creep test have also been recommended if needed [11, 31]. Muthen (1998) [11] recommended a minimum creep modulus of 20MPa at OBC to evaluate permanent deformation characteristics.

Table 2-5-Minimum acceptable criteria for different properties

Author	Property	Conditions	Minimum acceptable value
Bowering (1970) [59]	UCS	4 day soaked (wet) 3 day cured at 60°C (dry)	500kPa 700kPa
Akeroyd and Hicks (1988) [38]	Marshall Stability Marshall Quotient	24 hours soaking at 60°C, test at 60°C	3.5 kN 1.5 kN/mm
Maccarrone et al., (1994) [19]	ITS	3 days at 60°C (dry) 24 hours soaking at 60°C (wet) Test at 25°C and 50ms rise time	200kPa 100kPa
Lancaster et al., (1994) [53]	ITSM	3 days at 60°C (dry) 24 hours soaking at 60°C (wet) Test at 25°C	6000MPa 1500MPa

2.4 Structural design considerations

2.4.1 Mechanical behaviour of FBMs

Milton and Earland (1999) [69] of TRL, UK considered cement (hydraulic binder) stabilised materials as one family and foamed bitumen treated materials as another family of in situ stabilised materials. On the same lines Merrill et.al.,(2004) [70] proposed a diagram, based on three distinct material types. The apexes of this diagram correspond to fully hydraulically bound, fully visco-elastically bound and unbound material. Materials bound with Portland cement are expected to cure more quickly than materials with foamed bitumen. The materials treated with foamed bitumen are expected to be less prone to shrinkage cracking than hydraulically bound materials. The four material types discussed fall into three material families as follows (Figure 2-20):

- Quick hydraulic (QH) with hydraulic only binder(s) including cement;
- Slow hydraulic (SH) with hydraulic only binder(s) excluding cement;
- Quick viscoelastic (QVE) with bituminous and hydraulic binder(s) including cement;
- Slow viscoelastic (SVE) with bituminous only or, bituminous and hydraulic binder(s) excluding cement.

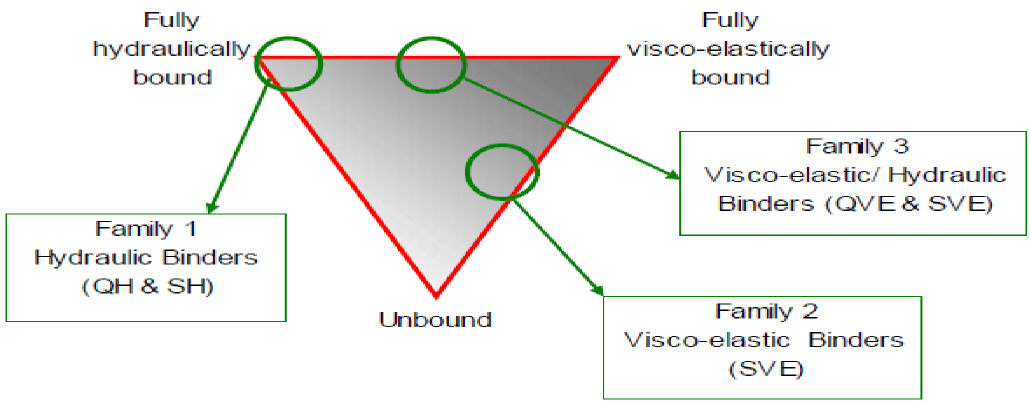


Figure 2-20 Classification of stabilised materials by TRL [70]

Based on this new classification FBMs are expected to fall into Family 3 or Family 2 depending on the amount of cementitious binder in the mix. As with any other pavement material, laboratory testing needs to be conducted on FBMs. The mechanical characterisation of the FBMs has been carried out by different testing modes by different researchers. The following section overviews the tests that have most commonly been used to determine strength and stiffness behaviour of the FBMs. It also discusses the factors that influence the mechanical behaviour of FBMs.

2.4.1.1 Strength behaviour

Strength can be referred to as the limiting stress or the combination of stresses (such as principal stress components in a tri-axial test) that a material can withstand without failure. There are different types of strength depending on mode of load (stress) applied such as tensile strength, compressive strength and shear strength. It is very unfair to state that one strength is more important than another, as pavement structures experience all corresponding stresses depending on relative location and load application.

From a micromechanical point of view, Fu (2009) [14] explained the role of each phase of FBM (2.1.3) in terms of the strength behaviour of the mix under different modes of stress. The aggregate skeleton (Figure 2-3) whose main load transfer and bearing mechanism is through particle interlocking and friction between adjacent aggregate particles takes most compressive and shear stresses. This mechanism is more effective if confinement is provided. Thus, the aggregate skeleton plays an important role in compressive strength. The presence of the mastic phase makes a difference in FBMs. This bitumen mastic is expected to provide tensile strength to the mix. The mineral filler phase is not only brittle, but also susceptible to water. So, the strength offered by the mineral filler phase is less water resistant. However, the presence of cementitious filler in the mineral phase improves tensile strength of the phase. From the literature it is understood that bitumen content, percentage of fines in the mix, presence of active filler, and curing time are the important factors that affect strength characteristics.

According to Jenkins (2000) [7] crushing is the representative failure mechanism in FBMs and so the UCS test has to be considered more appropriate for FB treated materials than conventional HMA. Bowering and Martin (1976) [16] from their limited laboratory data mentioned that in practice UCS of FBMs lies in the range of 1790kPa to 5400kPa depending on the water in the mix. They also found that above 1.5% residual bitumen FBMs showed superior strength characteristics to emulsion treated mixes. The influence of curing on compressive strength was also found to be significant [71]. The percentage of fines (<0.075mm) also influenced UCS of FBM. It was shown to be a positive influence by Semmelink (1991) [72] in which he treated sand with 5% FB. The effect of the presence of active filler in a mix on UCS has also been studied [73]. Most active fillers contribute to compressive strength improvement (Figure 2-21). Hydrated lime and quick lime appear to be less advantageous in comparison to cement. Loizos et al., (2004) [47] investigated the effect of Reclaimed Asphalt Pavement (RAP) addition on UCS. The laboratory data indicated that with an increase in RAP content UCS decreased.

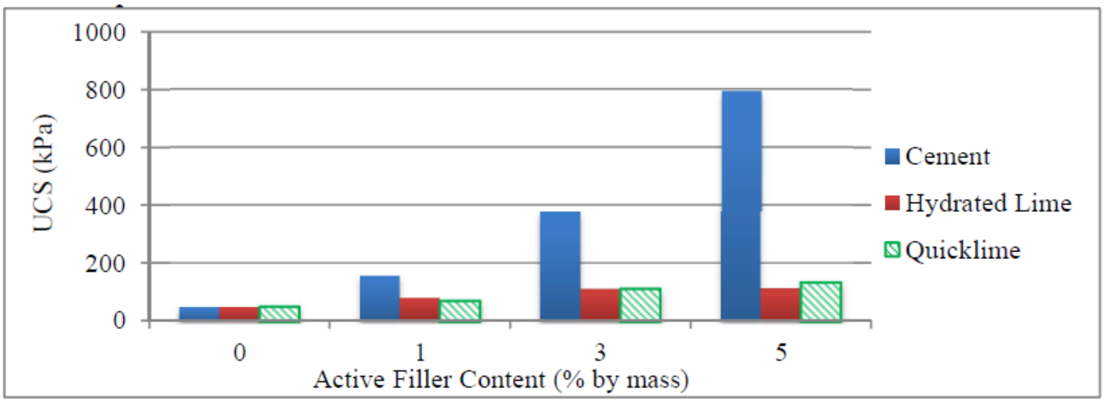


Figure 2-21 Effect active filler content on UCS[73]

From the laboratory data, Bowering and Martin (1976) [16] concluded that tensile strength (ITS) of FB treated mixes was superior to that of emulsion treated mixes and dry ITS values ranged between 200kPa and 550kPa. Sunarjono (2008) [3] evaluated the effect of curing time and thus available water content in the mix on ITS of FBMs (Figure 2-22 and Figure 2-23). ITS values showed a clear decreasing trend with available water content. The positive effect of the presence of active fillers was shown by many researchers [22, 73](Figure 2-24). The research also found that the presence of active filler does not affect the OBC of the mix [22]

Many researchers reported studies of FBMs with RAP material [22, 42, 47, 74]. A recent work [74] investigated the strength behaviour FBMs with RAP material and cement (Figure 2-25). The tensile strength achieved by FBMs with RAP was comparable with those with virgin sub-base material (SB), while cement showed a clear effect on ITS. Similar results were also observed by Halles and Thenoux (2009) [43]. Al-Abdul Wahhab et al., (2012) [74] also observed that the presence of RAP influenced the OBC.

From the literature it seems that the presence of RAP has a positive influence on ITS and a negitive effect on UCS. Moreover, OBC is independent of active filler presence, but this is not the case if RAP is present.

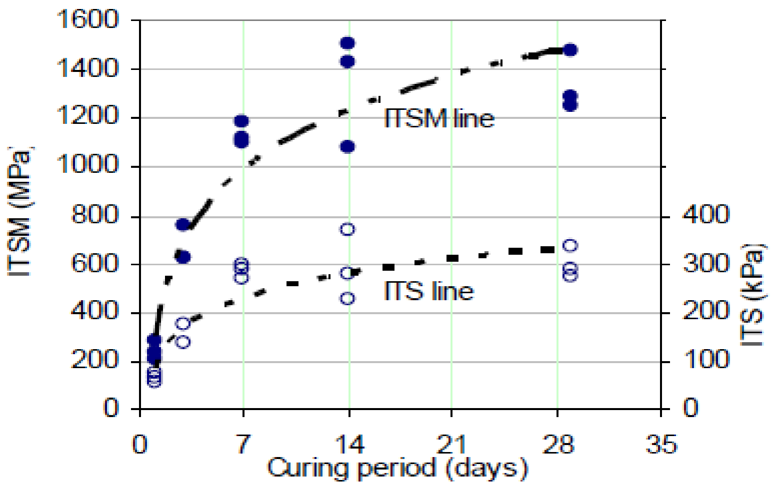


Figure 2-22 Effect of curing time on ITS [3]

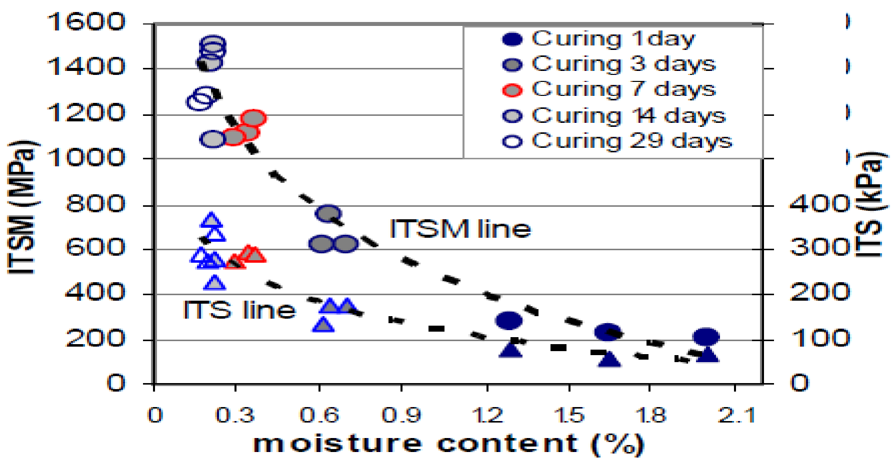


Figure 2-23 Effect of water content on ITS [3]

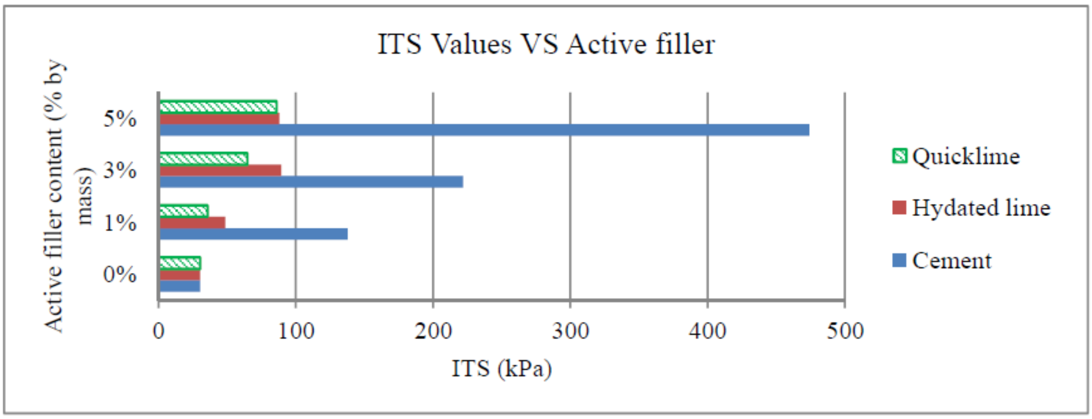


Figure 2-24 Effect of different active fillers and their content on ITS [73]

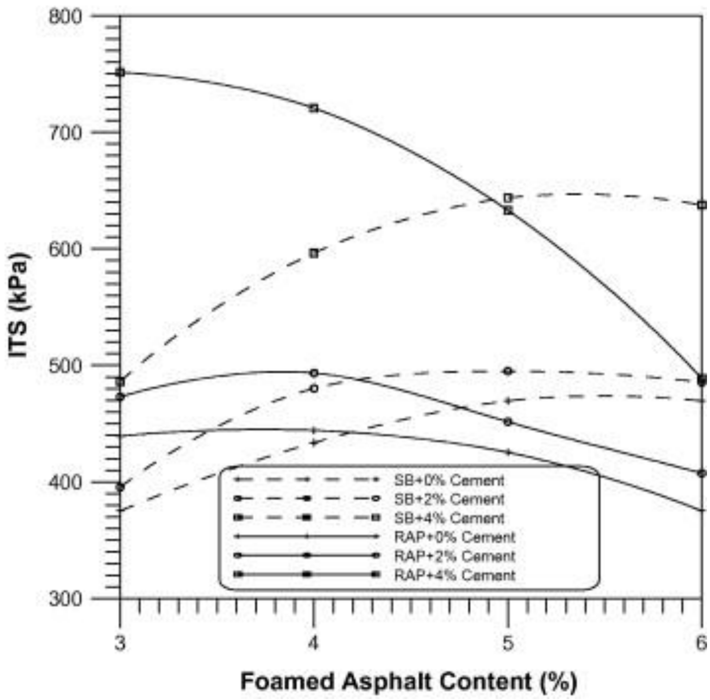


Figure 2-25 Effect of RAP addition on ITS [74]

2.4.1.2 Stress – Strain behaviour

It has become necessary to pay more attention to pavement material behaviour, particularly in terms of dynamic stress-strain response. Stiffness generally refers to material behaviour under applied load (stress). The slope of a stress-strain curve for a pavement material is generally referred to as stiffness. It is important to be clear what stress state is referred to when talking about stiffness values and carefully differentiate between tensile stiffness and compressive stiffness. The mechanisms through which the individual phases of FBMs contribute to stiffness are almost identical to those for strength behaviour [14].

Many laboratory and field test methods have been used to determine stiffness of FBMs. Each method applies a different state of stress, has different specimen geometry and loading pattern. The measured values in the laboratory for stiffness determination are certainly representative of the stress state applied, but are not necessarily relevant to the stress states and conditions in the field. Rate of loading, available water content, and the temperature at which load is applied also affect the measured stiffness.

***Test methods for stiffness*:**

Stiffness is a material property for numerical analysis of both primary response and distress of pavements. In the literature stiffness values have been derived from resilient modulus tests and performance tests. In the laboratory, tests can be conducted on cores extracted from the field or specimens compacted in the laboratory. A variety of loading patterns are used to measure the stiffness of pavement materials under a variety of loading conditions. These include monotonic loads in stress or strain control, frequency sweep tests, impulse and wave propagation tests, repeated load tests, creep, relaxation and creep and recovery tests. The laboratory tests on FBMs include the indirect tensile tests, tri-axial test, and flexural beam test. In the field, the stiffness of pavement materials may be measured either destructively or non-destructively. Destructive tests in the field use small scale testing instruments such as cone penetration tests or large scale methods such as an accelerated loading test facility. Non-destructive tests include impulse or static instruments such as Benklemann Beam and Falling Weight Deflectometer (FWD) tests. In general test data from non-destructive or destructive testing must be analysed by some form of inverse analysis to obtain stiffness. Stiffness can also be determined from data from repeated load permanent deformation tests and cyclic flexural beam fatigue tests.

Testing in indirect tensile stress mode is the most commonly used method for stiffness measurement of FBMs [33, 44, 65, 75, 76]. The loading is applied across the vertical diameter of a cylindrical specimen (Figure 2-26). This vertical loading produces both compressive stress and horizontal tensile stress along the diameter of the specimen. The magnitude of stress varies along the diameter, but is at a maximum in the centre of specimen. In uni-axial and tri-axial tests (Figure 2-27) cylindrical specimens are used and loading is applied along the axis of the specimen. However, in tri-axial loading lateral confinement also needs to be applied. Confinement may be an important factor in FBM evaluation, as aggregate is not fully bound as in HMA. Flexural beam type tests include monotonic loading and repetitive loading (fatigue) teats (Figure 2-28). The beam is subjected to both tension (at the bottom) and compression (at top). In the literature, back calculated resilient modulus based on FWD has been reported between 700MPa and 5000MPa [63, 76, 77].

From the literature it was found that, of the above discussed tests, the ITSM test was the most widely used test on FBMs for stiffness determination. However, unrealistically high modulus values of higher than 5000MPa have been reported [77]. On the other hand, stiffness measured using other tests such as tri-axial [43, 78-80] and flexural beam [80-82] were consistent with back calculation results from FWD tests. A potential difference between the indirect tensile test and other modes of test is that the Poisson’s ratio is used in calculating the stress in the ITSM test while no material specific constant is involved in the stress calculation of the other two tests.

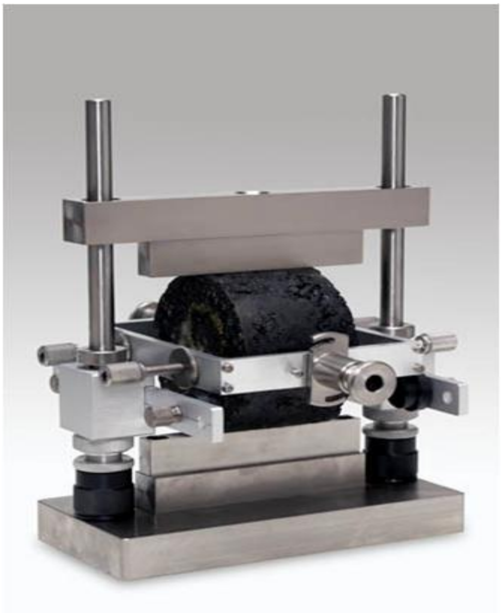


Figure 2-26 Indirect tensile mode of loading

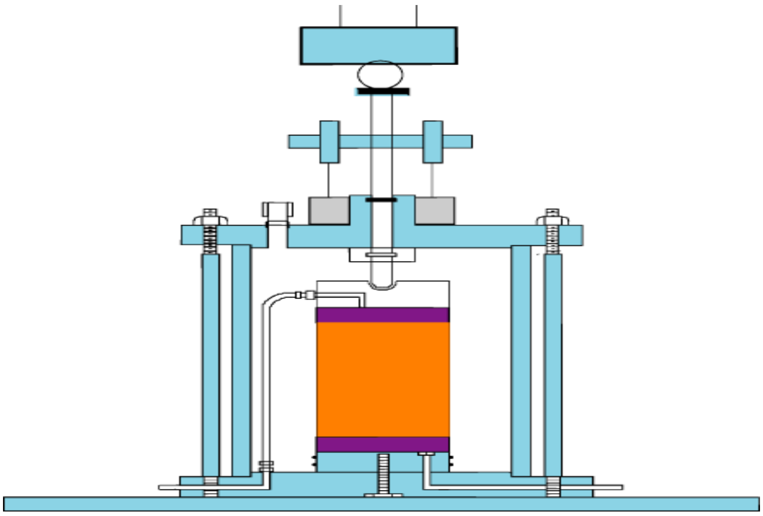


Figure 2-27 Uni-axial and tri-axial mode of loading

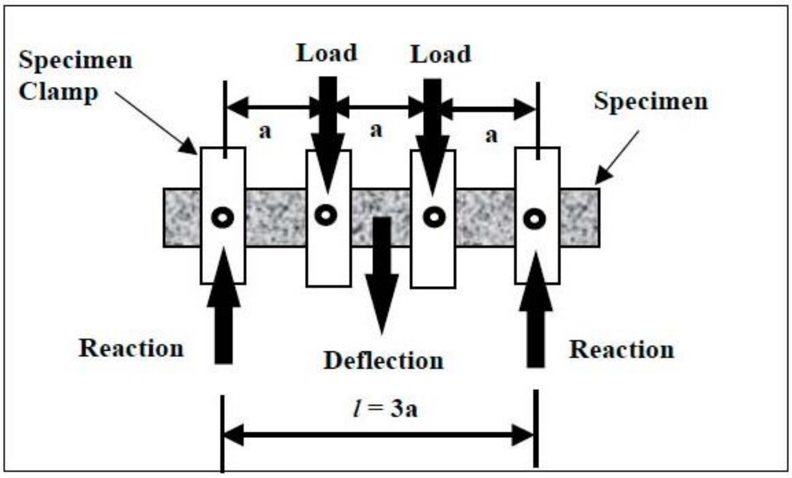


Figure 2-28 Flexural beam test

Temperature dependency:

In general, the stiffness of HMA decreases as the temperature increases. The temperature sensitivity of HMA and FBM stiffness can be assumed to be somewhat similar as they are dependent on the rheology (although to differing degrees) of bitumen in the mix. However, the role of bituminous binder in FBM is different from that in HMA as FBMs have partial coating of large aggregates and 'spot welding' of the mix with mastic. Fu and Harvey (2006) [79] stated that a knowledge of the temperature sensitivity of FBM properties, especially of stiffness, is an important issue in both project level pavement mixes, structural design and advanced research for the following three reasons. First, stiffness is an important input parameter in mechanistic-empirical pavement design and values at different temperatures are needed. Second, it is impossible to control the pavement temperature during field testing, such as deflection testing with the falling weight deflectometer (FWD). This makes interpreting the test results very difficult since comparison of moduli at the same temperature is essential. Finally, studying the temperature sensitivity of FBMs can provide useful insight into the stabilization mechanism of FB. The temperature sensitivity of the mix properties is primarily due to the rheological characteristics of the bitumen which is the stabilizing agent.

It has been reported that the stiffness of FBMs was as high as that of HMA mixes at intermediate temperature, but that it was much less sensitive to temperature [83]. It has also been reported that the stiffness of FBM reduced by 30% to 44% when the temperature increased from 10°C to 40°C. Saleh (2006) [46] studied temperature susceptibility of FBMs. Results showed a decrease in resilient modulus values with temperature (Figure 2-29). Results also showed that FBMs that were produced with binder that was of low temperature susceptibility also had low temperature susceptibility. HMA mixes were more sensitive to temperature variation than FBMs.

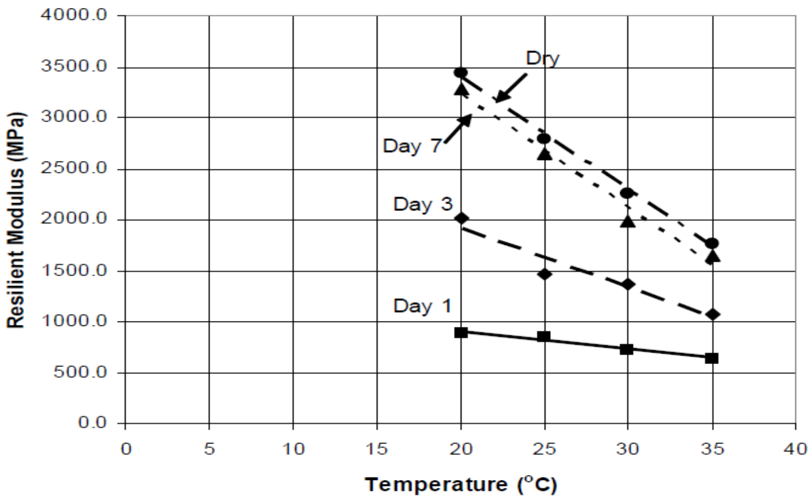


Figure 2-29 Temperature dependent behaviour of FBMs[46]

Stress –state dependency

Jenkins (2000) [7] has examined the stress dependent behaviour of FBMs with cement and without cement. He carried out tri-axial tests on FBMs at different stress ratios. Based on his test results he stated that stress dependent behaviour is only valid for FBMs without the addition of cement. Fu and Harvey (2007) [79] made some observations with regard to the effect of confining stress and deviator stress. Their results showed that as bulk stress (θ) increases, the resilient modulus also increases (Figure 2-30).

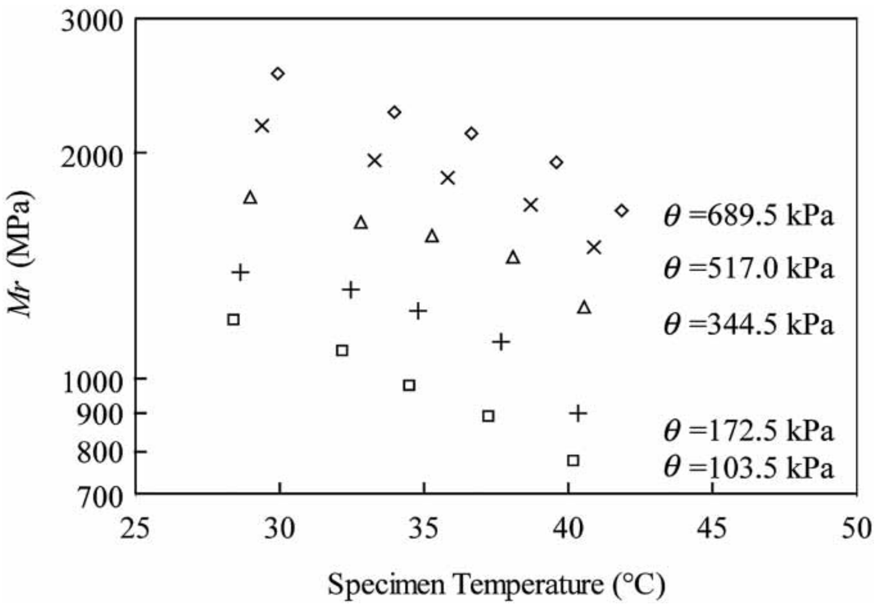


Figure 2-30 Stress dependent behaviour of FBMs[79]

***Water susceptibility*:**

Because of micro-structure of FBMs in which aggregates are partially coated and there are significant voids, water sensitivity is an important issue to be considered in mix design and structural design of FBMs. Some researchers [33, 76] have reported the water susceptibility of stiffness. Fu et al., (2008) [34] reported that for field performance the conditions under which FBM properties are tested are critical to field performance.

2.4.2 Performance characteristics of FBMs

The Asphalt Academy (2002) [84] has also published a schematic including FBM types as shown in Figure 2.31. As shown in the scheme, foamed asphalt materials sit approximately in the middle relative to unbound/cement bound (left side) and HMA (Hot Mix Asphalt, right side). Foamed asphalt properties seem to vary from weak to moderately strong, depending on the granular type and active filler content. At the same time, these properties also vary from low to high resistance to permanent deformation. It is supposed that a mixture using large stone (nominal size more than 25mm) and high coarse particle content will potentially result in a lightly bound material and hence exhibit low strength and low resistance to permanent deformation. When active filler e.g. cement is added to this mixture, the strength will increase moderately. In addition, when binder content is increased, the mixture exhibits more flexibility and hence greater resistance to fatigue. HMA and Half-warm mixtures appear at the bottom right; this does not mean that these mixtures have low resistance to permanent deformation, but that they normally use high binder content without active filler.

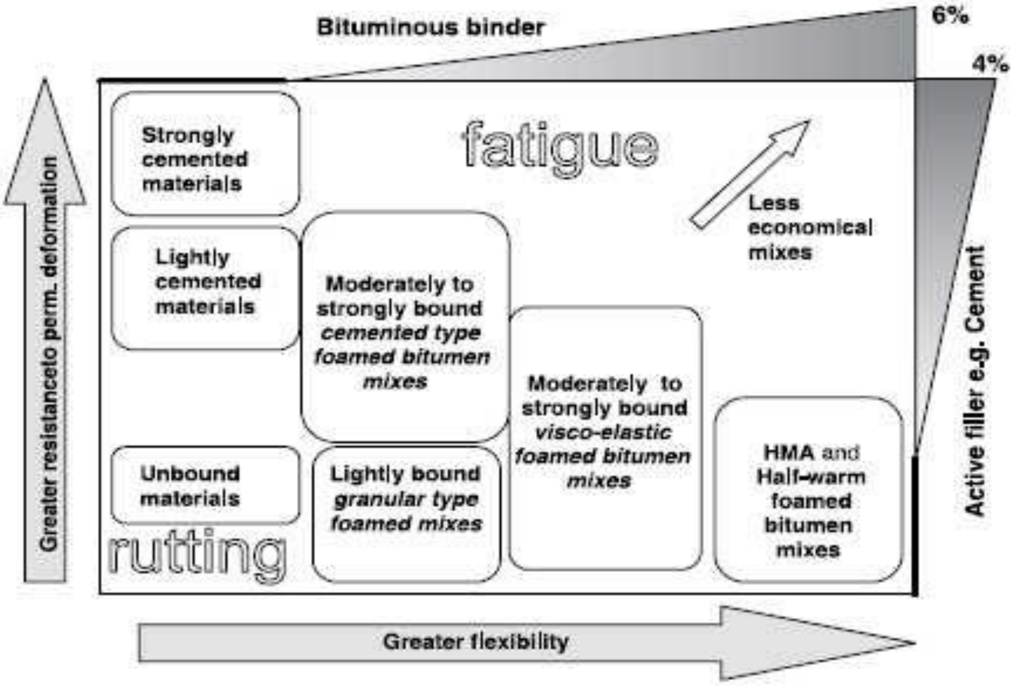


Figure 2-31 Expected performance of foamed bitumen mixes [84]

2.4.2.1 Permanent deformation behaviour

In pavement layers with bituminous mixes subjected to traffic loads, stresses are induced which are usually very small when compared to their strength. These stress pulses cause strain most of which is recoverable. The irrecoverable strain is very small for one load application. However, for repeated load which is the actual case in pavements these small irrecoverable strains accumulate. This permanent strain induced by traffic loads manifests as permanent deformation in pavement layers. A number of researchers [43, 50, 52, 85, 86] have studied the permanent deformation behaviour of FBMs and the factors that affect it.

Jenkins (2000) [7] stated that strength ratio which incorporates factors such as compaction and water content could be an indicator for permanent deformation behaviour of FBMs. The stress ratio employed was the ratio of applied deviatoric stress to deviatoric stress at failure which can be obtained by a monotonic tri-axial test. These authors performed repeated load tri-axial tests over a range of stress ratios to establish stress dependent behaviour of FBMs. The test results describe a critical boundary in terms of stress ratio between stable permanent axial strain and accelerated permanent strain. This boundary was at a stress ratio of 55% for mixes without cement and 52% for mixes with cement (Figure 2-32). These results were later verified by Long and Ventura (2004) [81] who found that as the ratio increased the load repetitions that could be sustained to a certain level of plastic strain decreased, with this decrease becoming more rapid at higher stress ratios. In both works dominance of neither density not saturation was apparent in the permanent deformation behaviour of FBMs.

He and Wong (2007) [85] conducted repeated uni-axial load tests on specimens compacted by the Marshall method. The main aim was to understand the effect of addition of RAP on permanent deformation behaviour. The results implied that an increase of RAP improved resistance of FBMs to permanent deformation. Their continued research work [87] studied the effect of binder grade and RAP age on the water susceptibility of permanent deformation behaviour. However, it has been noted that a higher percentage of RAP may result in high permanent deformation (Figure 2-33) [52].

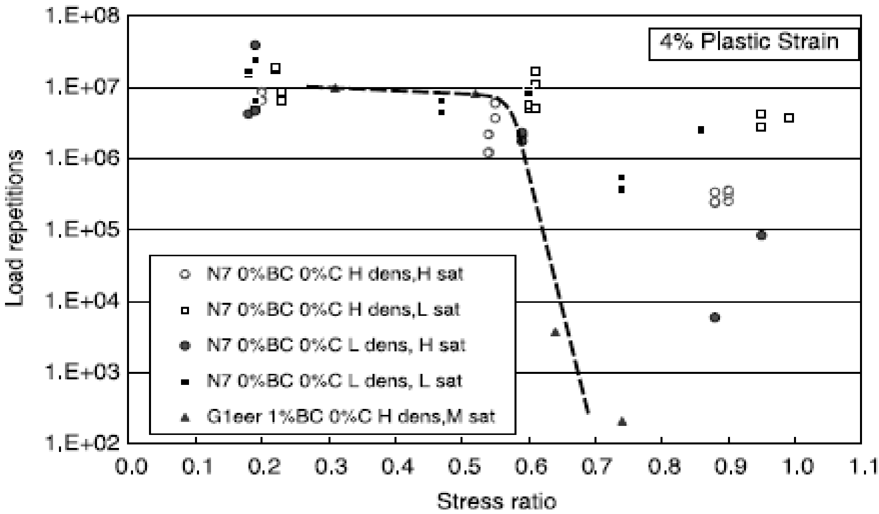


Figure 2-32 Effect of stress ratio on permanent deformation [50]

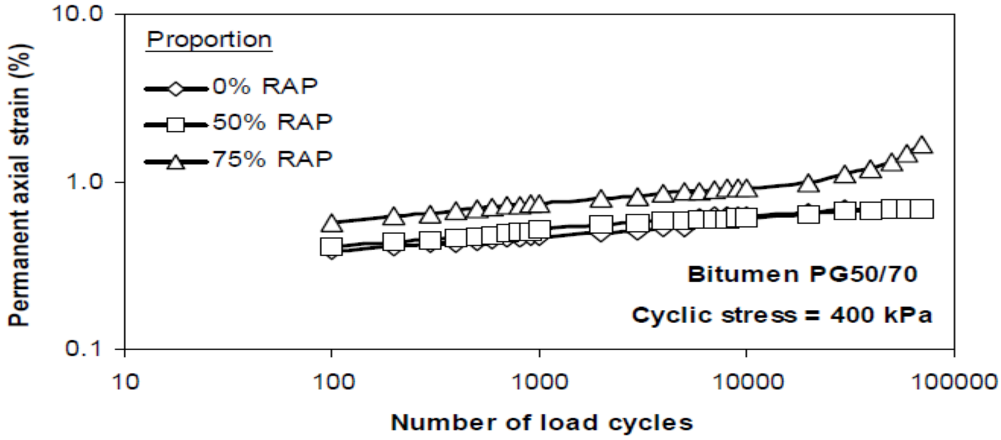


Figure 2-33 Effect of RAP content on permanent deformation [52]

The effect of binder grade on permanent deformation was studied and results showed that mixes with softer binder tend to deform more rapidly than those with harder binder (Figure 2-34). Studies on stress dependent permanent deformation has also been undertaken (Figure 2-35). Kim et al., (2008) [88] studied the permanent deformation behaviour of mixes with FB contents of 1%, 2% and 3% by tri-axial repeated load tests. They used flow number as an indicator of behaviour. The results showed that the higher the FB the higher the flow numbers i.e. the better the resistance to permanent deformation (Figure 2-36). Similar results were also shown by repeated load tests as part of laboratory testing of full scale accelerated experiments on FB pavements by Gonzalez (2009) [89]. Halles and Thenoux (2009) [43] studied the influence of active filler such as cement, lime and fly ash on the performance of mixes. Tri-axial permanent deformation tests showed that mixes with fly ash had the poorest resistance to permanent deformation, while mixes with cement and lime had better resistance and improved mix performance.

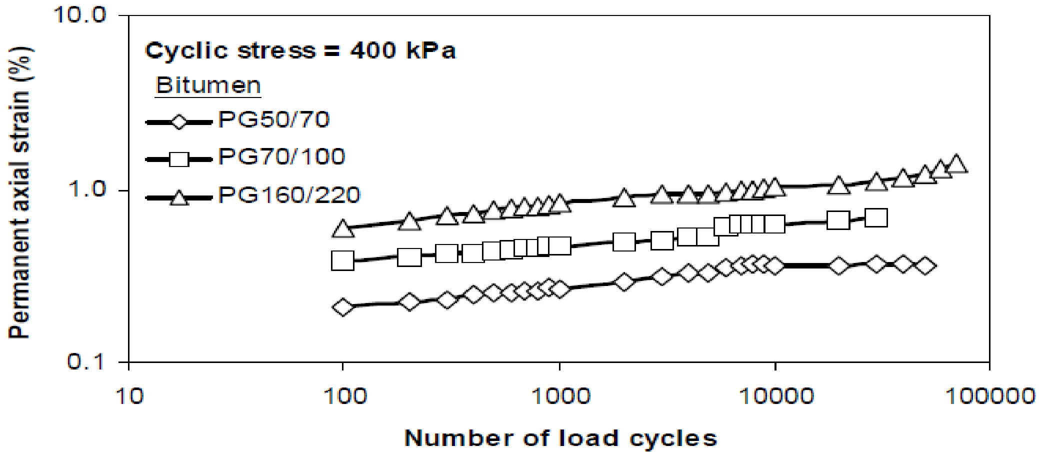


Figure 2-34 Effect of binder grade on permanent deformation[52]

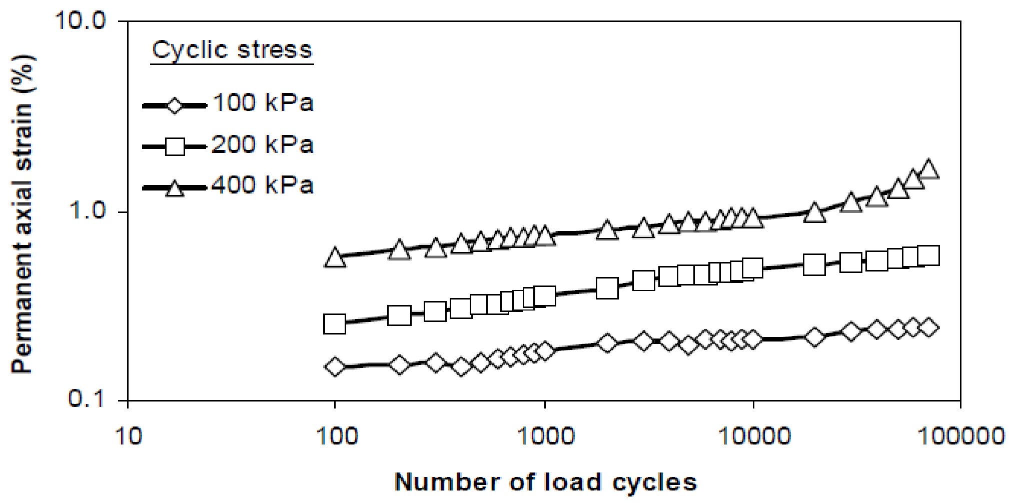


Figure 2-35 Stress dependency of permanent deformation[52]

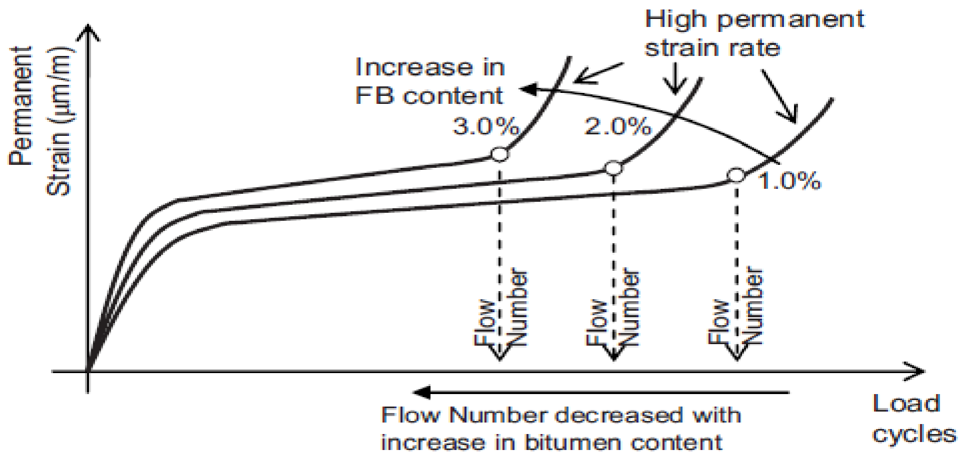


Figure 2-36 Effect of bitumen content on permanent deformation [88]

2.4.2.2 Fatigue behaviour

Fatigue behaviour of mixes with bitumen under repeated loading is an important factor in pavement design. A variety of methods have been developed for the fatigue testing of bituminous mixtures. Among the most popular methods are bending of beams and the indirect tensile test. In beam tests, a simple beam with third-point or centre point loading or a cantilever beam with rotating bending have been used. Repeated load indirect tensile tests have also been employed (Figure 2-37). Two types of controlled loading namely controlled stress or controlled strain can be applied. In the controlled stress test, the applied stress remains unchanged but the strain increases with the number of repetitions. In the controlled strain test, the applied strain is kept unchanged and the load decreases with the number of repetitions. Long (2001) [51] tested beams for fatigue with a range of cement and foamed bitumen contents. He considered strain-at-break values as an indication of flexibility and therefore of the fatigue resistance of the mix. The results showed that addition of FB contributed significantly to the fatigue resistance of the mix. In assessment of fatigue properties of foamed bitumen, he carried out repeated load indirect tensile fatigue tests on Marshall samples at optimum binder content. The results showed improvement in fatigue life with cement addition. Moreover, this research also claimed that the fatigue life of FBMs is the same as those of HMA and superior to those of cement stabilised materials. This in turn is contradicted by Saleh (2006b) [46] in his investigation of fatigue behaviour of foam-stabilised mixes (M20FA1C) in comparison with dense graded HMA (AC10 HMA) and open graded porous asphalt (OGPA). Foam-stabilised mixes had a lower fatigue life compared to HMA and OGPA (Figure 2-40). Similar results were also found in the investigation undertaken by Sunarjono (2008) [3].

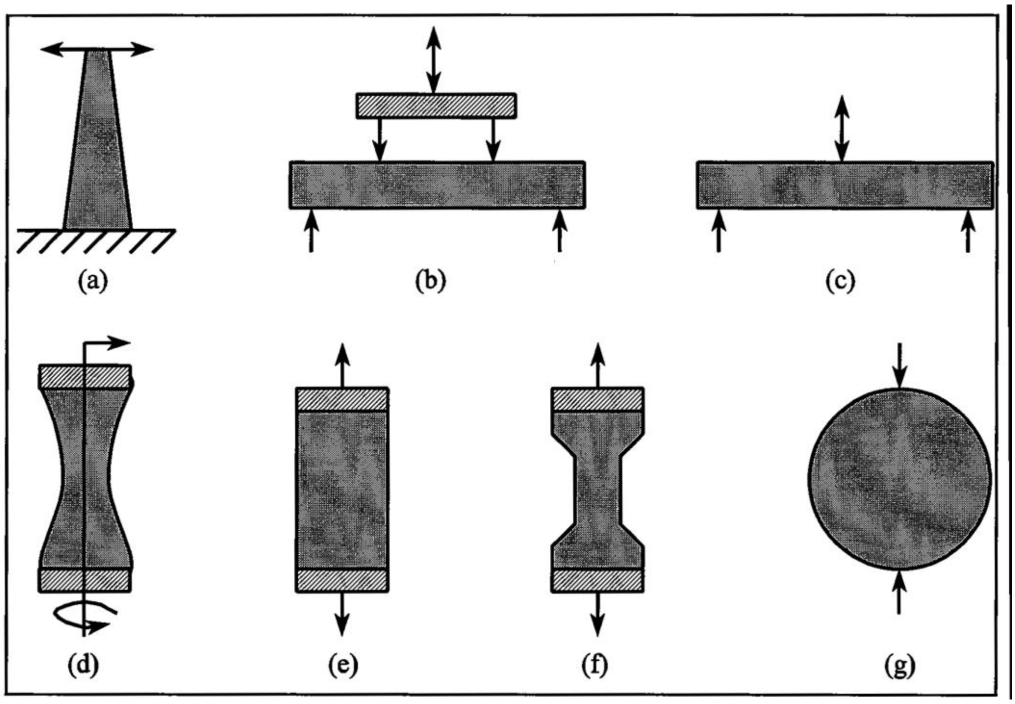


Figure 2-37 Fatigue tests that can be employed [90]

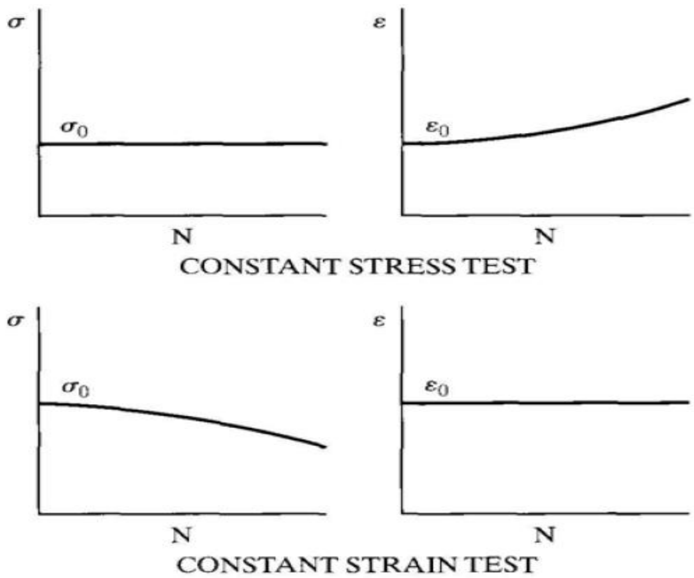


Figure 2-38 Constant stress or constant strain modes of loading

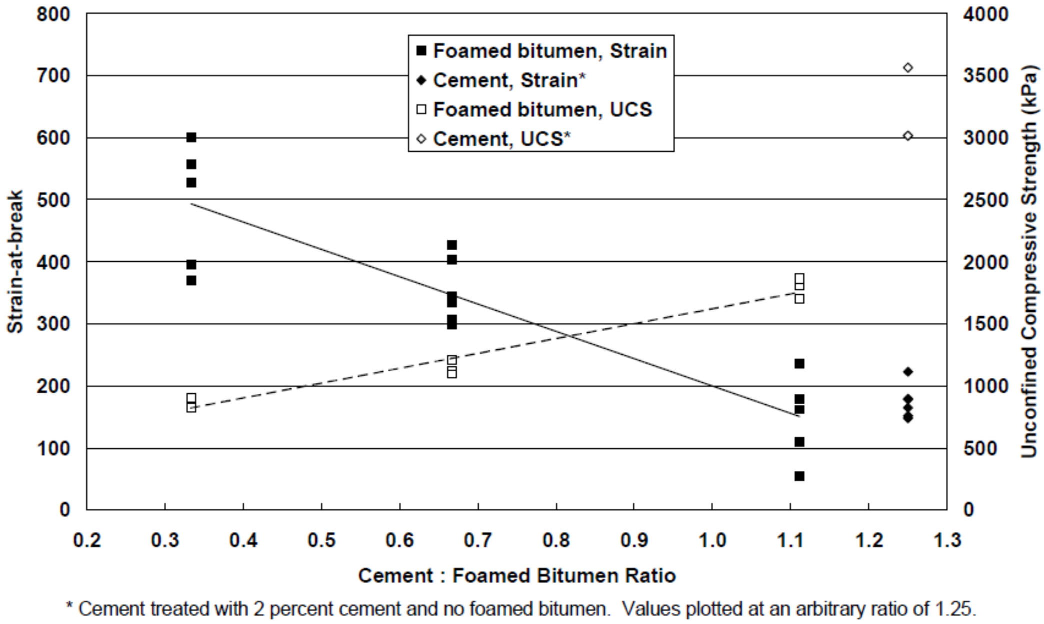


Figure 2-39 Effect of FB content on strain-at-break [51]

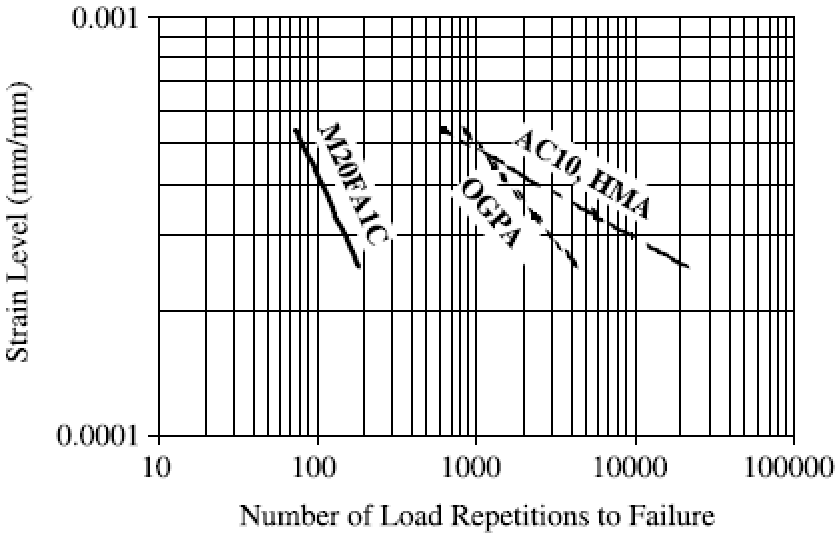


Figure 2-40 Fatigue life for three different mixes [46]

2.4.3 Failure mechanism

In development of structural design models (using transfer functions relating laboratory performance to the field) for FB treated pavement layers, heavy Vehicle Simulator (HVS) testing has been carried out [51]. A two phase failure mechanism of effective fatigue and permanent deformation was observed. During the effective fatigue phase the layer stiffness reached a constant value. This constant stiffness value was independent of traffic loads. After this stage the layer was assumed to behave as a granular material and this was termed the 'equivalent granular state'. During this granular state the layer distress is in terms of permanent deformation. In the granular state, though stiffness remained the same, permanent deformation increased rapidly in the beginning while the pavement was bedding in and then continued at a more gradual rate. In contradiction of this view, Long Term Pavement Performance (LTPP) data on FB stabilised pavements indicated that most pavements did not exhibit any cracking or phase change in any layer [75, 91, 92].

Theyse et al., 2006 [93] reported the expected behaviour of FB treated RAP under HVS testing. The results indicated that the mode of distress of the pavements differed between the favourable conditions in summer and autumn and unfavourable conditions in winter. The mode of distress before the onset of winter consisted of gradual deformation of the pavement resulting in a surface rut with limited fatigue cracking. After the winter, the mode changed to a more rapid rate of rutting.

Chen et al., (2006) [94] investigated the cause of structural distress of FB treated layers. Extensive field tests including FWD, seismic, GPR and DCP were conducted. The study concluded that rutting and alligator cracking were the primary distress modes of FBM base. Based on laboratory observations, they hypothesised a failure mechanism for FB treated layers (Figure 2-41). The layer effectively became two layers in terms of stiffness. The wetting front which was considered as the interface between the two layers was assumed to deteriorate from one end.

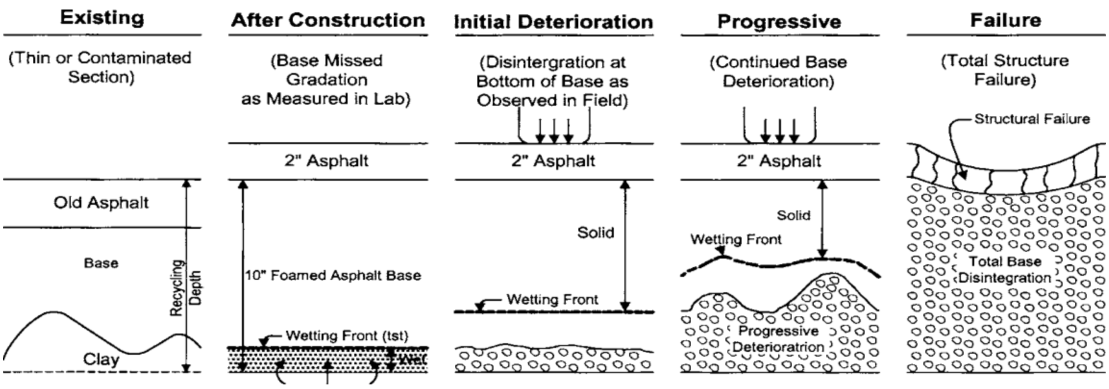


Figure 2-41 Failure mechanism in FB treated layers [94].

2.4.4 Failure evaluation

The use of foamed bitumen technology in pavement rehabilitation or new construction was given a boost with the availability of cold recyclers and other equipment in the mid 1990's. Mix design considerations and mechanical behaviour were well documented and mostly standardised. However, not much focus was given to field performance and the evaluation of pavements with FB treated layers. The reason may be availability of pavement sections. Nevertheless, field evaluations have been carried out on trial sections in Australia, South Africa, Greece and USA. There are two sources of field data; full scale accelerated pavement tests (APT) and field evaluation data. The advantage of APT over in-service pavement evaluation is rapid application of loads and data collection which allows the pavement to be tested in a short time and with less resources.

The first noted performance evaluation on pavements with FB treated layers was carried out in Australia [67] where FB technology was first focused. Non-destructive testing was carried out immediately after compaction, three weeks after compaction and after four months. The results showed 50% and 300% increase in modulus after three weeks and four months respectively. However, modulus values immediately after compaction were the same as for untreated material. Lancaster et al., (1994) [53] also found similar results in which deflections were unchanged even after an FB treated layer was laid over subgrade. FWD tests were carried out on some completed pavements in Australia [95] and results indicated an increase in stiffness with time.

Lane and Kazmierowski, (2005) [96] compared FB stabilisation with emulsified bitumen stabilisation. FWD tests and evaluation of pavement roughness and rutting were carried out shortly after placing the HMA surface course. The results showed that both pavements were performing similarly. Ride comfort rating was good for both pavement sections. The study found little difference between foamed bitumen and emulsion treated pavement sections in terms of short term performance.

2.4.4.1 South African experience:

Foamed bitumen treatment has been used in South Africa on a variety of materials including RAP. Long et al., (2002) [97] developed mechanistic-empirical structural design models (transfer functions) for pavements with FB treated layers. The models were developed using data collected from pavements loaded with the Heavy Vehicle Simulator (HVS), a full scale accelerated testing facility. The sections (Figure 2-42) used for developing the transfer functions were

- P 243/1, Vereeniging, 2000 which was rehabilitated using deep in-situ recycling and named as Section 411A4. The reclaimed material was a cement treated base with some surface and sub-base, treated with 1.8% FB and 2% cement [98].
- N7 (TR 11/1), Cape Town, 2002 was constructed using deep in situ recycling (DISR), recycling a crushed stone with 2.3% of FB and 1% cement. The FB treated sections were named as 415A5 and 416A5 [91].

The loading of the pavement included a test using a 40kN wheel load (E80 axle load) and another test at a higher load. During the load application, the pavement condition was assessed at regular intervals of time including measurements of the surface deformation, water conditions and visual inspection. Water was added at the end of the test to induce additional damage in the pavement to simulate a worst possible condition. The elastic and plastic deformations of the layers were measured using a Multi Depth Deflectometer (MDD) which was installed at different depths in the pavement to measure strains (Figure 2-43). The resilient moduli were back calculated using data from the MDD. The accumulation of permanent deformation of the layer was also derived from MDD data.

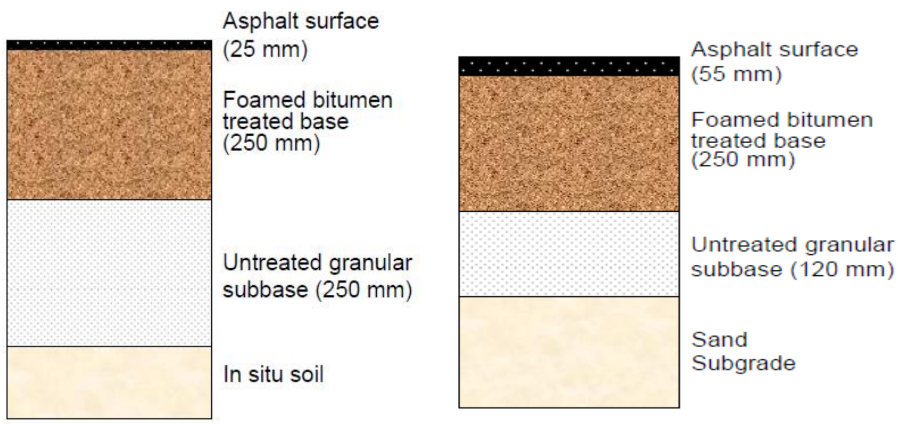


Figure 2-42 Section 411A4 and 415A5 [91, 98]

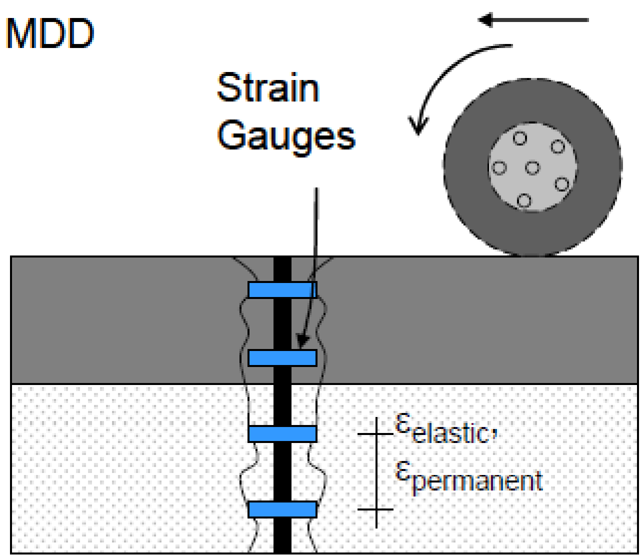


Figure 2-43 Strain gauges (MDD) embedded in pavement layer [98]

The results indicated that the initial resilient moduli of FBMs are expected to be at least 1000MPa and even higher depending on the quality of the parent material. However, this high value of resilient modulus reduced under trafficking, the rate of decrease being dependent on a range of factors including environment, traffic loading and flexibility of the material (Figure 2-44). The results also showed that permanent deformation (PD) increased at a faster rate at the beginning of the test, and then at a slower rate for the remainder of test (Figure 2-45).

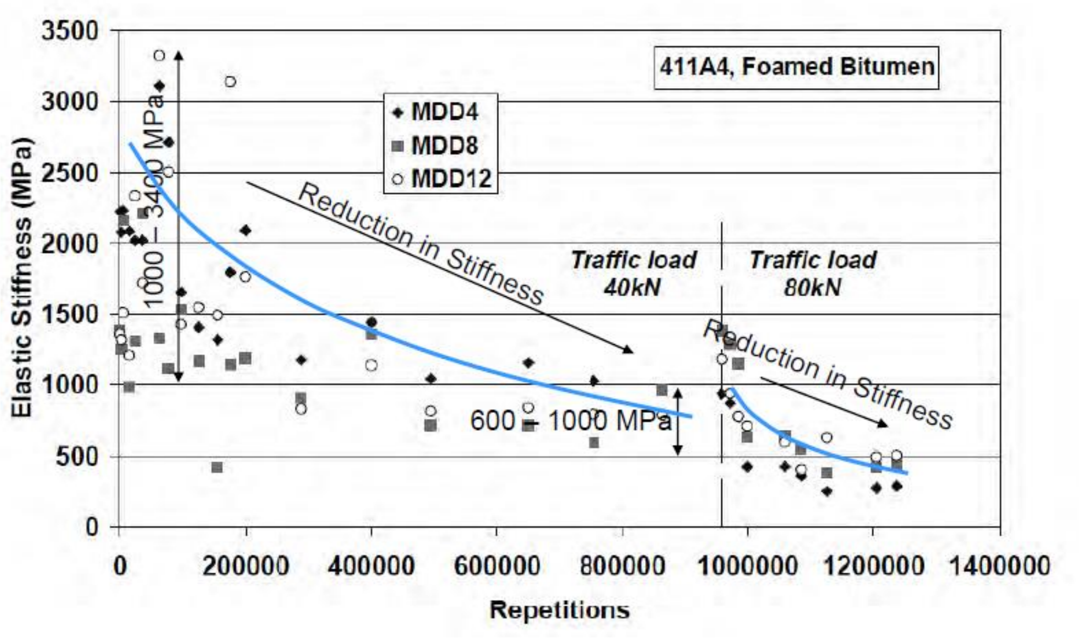


Figure 2-44 Back-calculated resilient modulus of a tested section [98]

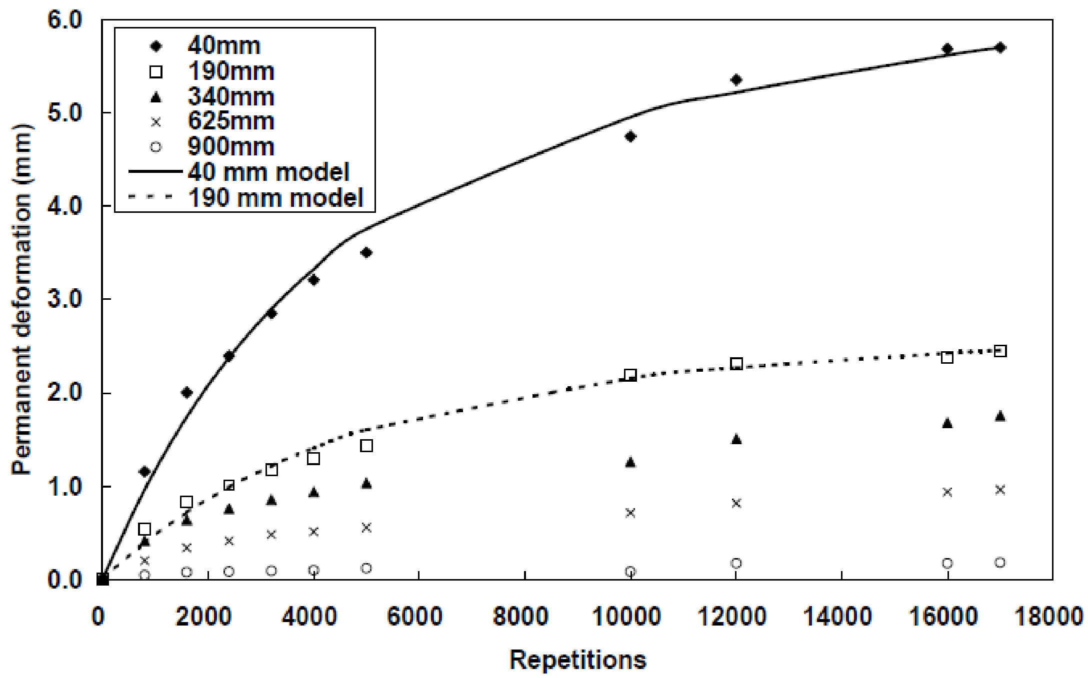


Figure 2-45 Permanent deformation accumulation in a FB treated layer [91]

2.4.4.2 United States experience:

An accelerated full scale experiment on a FB treated pavement was conducted at the University of Kansas [99]. Four pavement sections were constructed in which three had foamed bitumen treated layers with different dimensions and the other had a control unbound granular layer (Figure 2-46). The FB treated materials had 50% RAP and were stabilised with 3% FB and 1% cement. Pressure cells and strain gauges were embedded in the sections to monitor response during loading. FWD tests were conducted before accelerated loading and during loading of the sections. All four pavement sections were loaded with 500000 axle loads repetitions (dual wheel single axle with a total load of 75.7kN for the first 100000 load cycles, followed by 400000 load cycles of a dual wheel tandem axle with a total load of 142.4kN) at room temperature and moderate water levels in the subgrade soil [100]. The layer moduli were back calculated from FWD deflection data (Figure 2-47). All pavements showed a decrease in modulus including the unbound granular layer. As no other distress was observed, comparison was made using permanent deformation measurements only. From the results it was concluded that 225mm of recycled FB base course showed performance equivalent to that of a 225mm Kansas granular base layer [100].

HVS testing of a full-scale experiment was also carried out on a FB treated pavement in northern California [93]. The recycled layer was treated with 2.5% of FB and 1% of cement. Out of four tests, one test was carried out with controlled water flow over the surface. The back-calculated results from FWD values showed a decrease in moduli with trafficking. The test results also showed that the FBM layer elastic and plastic responses were sensitive to high water content.

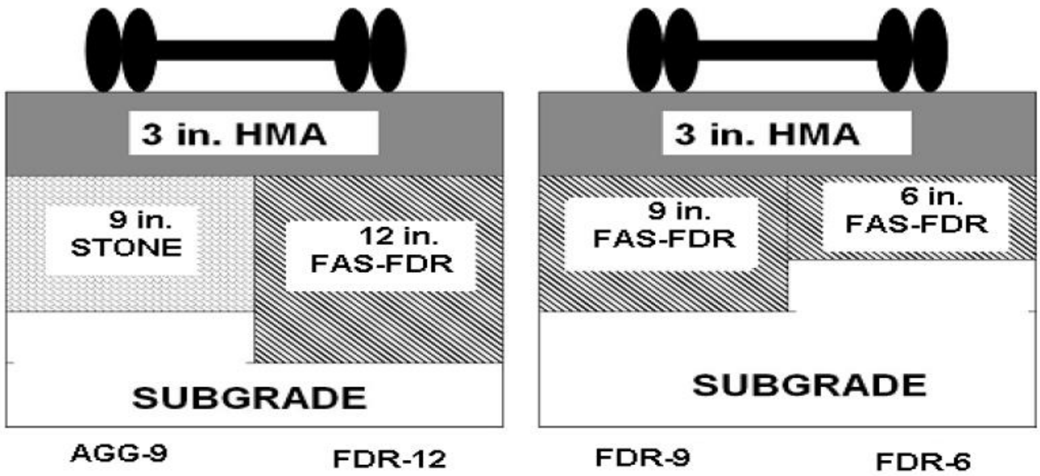


Figure 2-47 Cross sections of trial pavement sections [98]

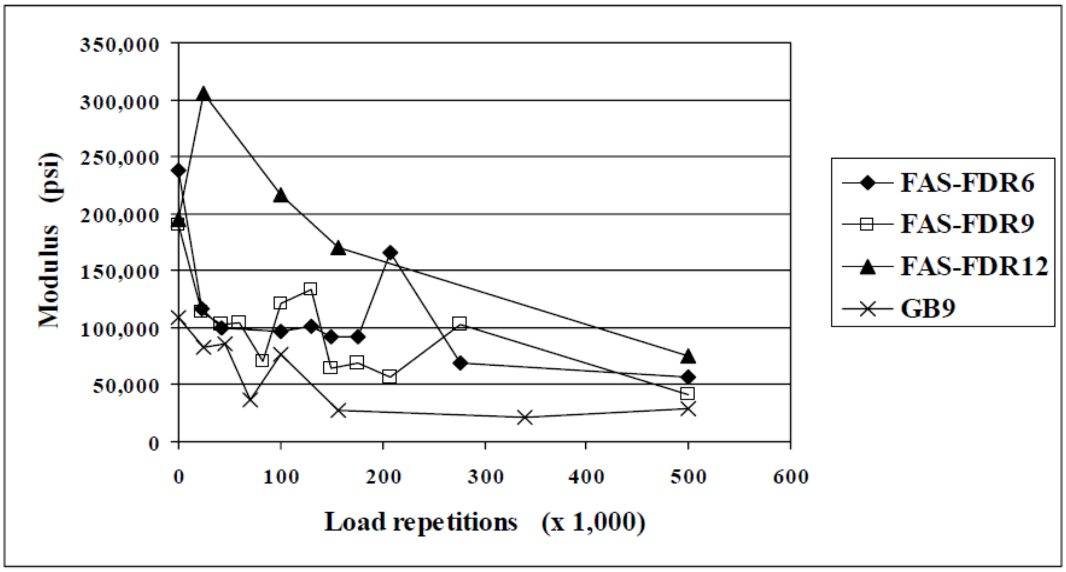


Figure 2-48 **Back-calculated moduli for different sections [100]**

2.4.4.3 Greece experience:

A field evaluation was undertaken by the National Technical University of Athens (NTUA) on a major Greek heavily-trafficked highway which had been recycled using FB treatment [101]. The main focus of monitoring and data analysis was the applicability of the non-destructive (FWD) technique (NDT) for structural assessment of early life performance of recycled pavements. The recycled layer was treated with 3.2% FB and 1% cement (Figure 2-49); 30% sand was also added to achieve the desired gradation. FWD surveys were undertaken during and after completion of the recycling work which comprised several monitoring levels in chronological order in order to include the curing phenomenon of the FBM layer. The measured deflection (corrected to 20°C) showed a decreasing trend with time (Figure 2-50). Composite moduli (combined bituminous and FBM modulus) were back-calculated from FWD data (Figure 2-51). In most of the cases there was an increase in the FBM layer modulus over time even after sections were opened to traffic. This improvement in modulus was attributed to curing of the FBM and compaction under traffic of the bituminous overlay material. The modulus increment was also confirmed by ITSM tests using the Nottingham Asphalt Tester (NAT) on extracted cores.

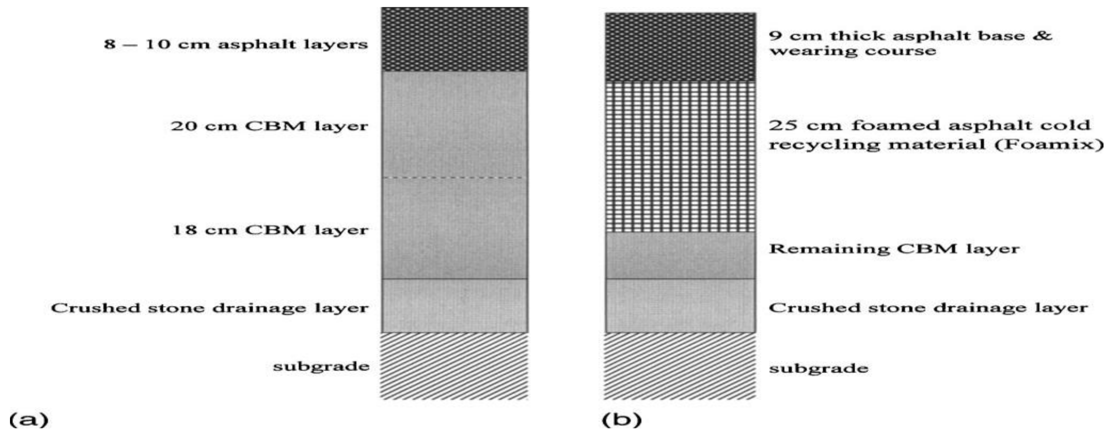


Figure 2-49 (a) Semi-rigid pavement before rehabilitation; (b) recycled pavement structure [101]

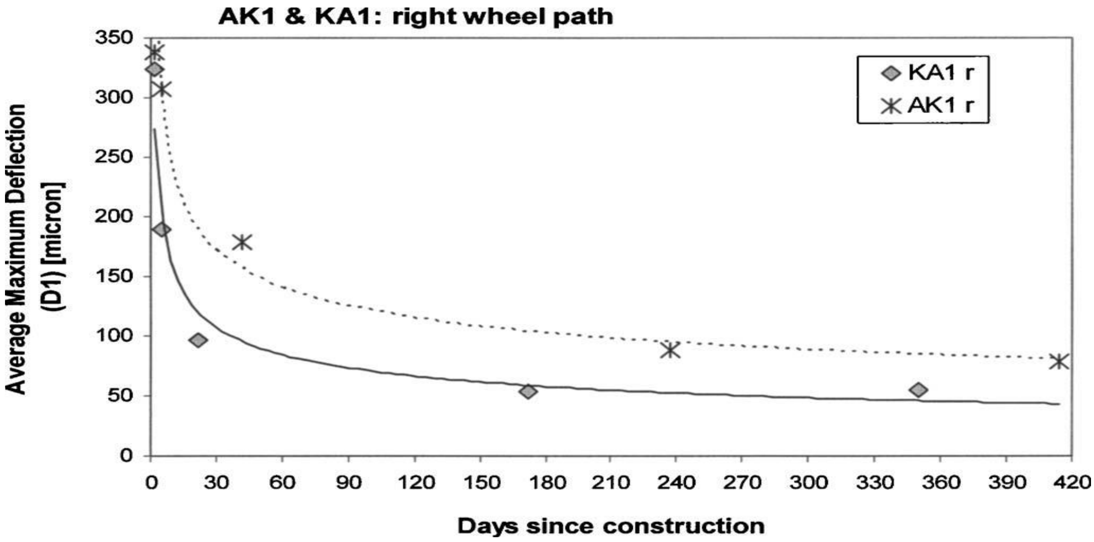


Figure 2-50 Decrease of average maximum deflection with time [101]

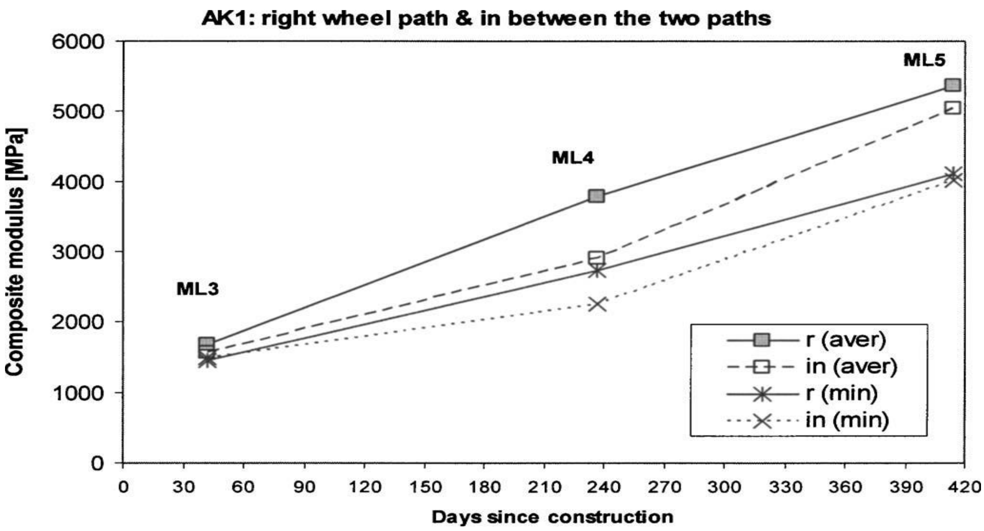


Figure 2-51 Increase of composite modulus with time [101]

2.4.4.4 University of Nottingham experience:

A trial section of FBM was constructed in the Nottingham University Pavement Test Facility (NPTF) [3, 52]. Four combinations of crushed limestone and RAP with selected binder were used to simulate recycled construction (Figure 2-52). Strain gauges were embedded in the test section to collect strain data during loading. Performance of the trial pavement under traffic was evaluated by repeatedly applying wheel loads. Trafficking was carried out as in Figure 2-53 at an approximate velocity of 3 km/hr. The results for each section are as in Figure 2-54. The strains at the bottom of the FB layer were recorded at intervals of 1000 load applications. The profile of the pavement was measured using a straight edge. After completion of all trafficking, cores were extracted from each section for laboratory testing of the modulus. The surface rutting of each section was measured at seven points at equal intervals along the wheel path. The rutting was developed to varying degrees in different sections [52]. The results indicated that the deformation resistance potential of FBMs depends on the binder type, mixture proportion and the presence of cement.

Section	Mix 2	Mix 1
Mixture type	Foamed asphalt	Foamed asphalt
Composition	75% RAP+25% VCL	75% RAP+25% VCL
Binder	Pen 70/100, FBC 2.5%	Pen 50/70, FBC 2.5%
Storage time	15 days	11 days
Water content at compaction	3.90%	4.20%
Target density	2020 kg/m3 (total mass= 311 kg)	2020 kg/m3 (total mass= 311 kg)
Age at start trafficking	12 days	13 days
Section	**Mix 4**	**Mix 3**
Mixture type	Foamed asphalt + 1.5% cement	Foamed asphalt
Composition	50% RAP+50% VCL	50% RAP+50% VCL
Binder	Pen 70/100, FBC 2.7%	Pen 70/100, FBC 2.7%
Storage time	14 days	11 days
Water content at compaction	4.30%	4.30%
Target density	2200 kg/m3 (total mass= 338 kg)	2200 kg/m3 (total mass= 338 kg)
Age at start trafficking	8 days	13 days

Figure 2-52 Summary of mixes used [3]

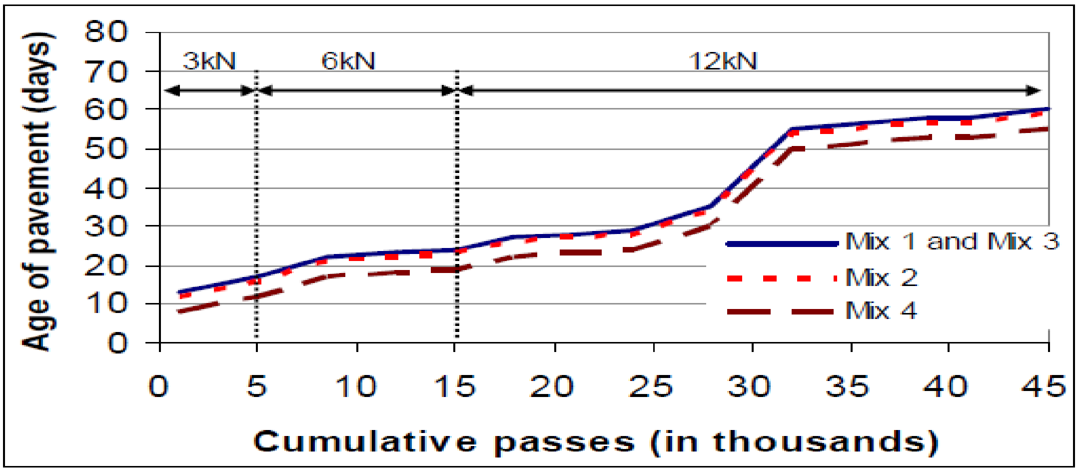


Figure 2-53 Trafficking schedule of test sections [3]

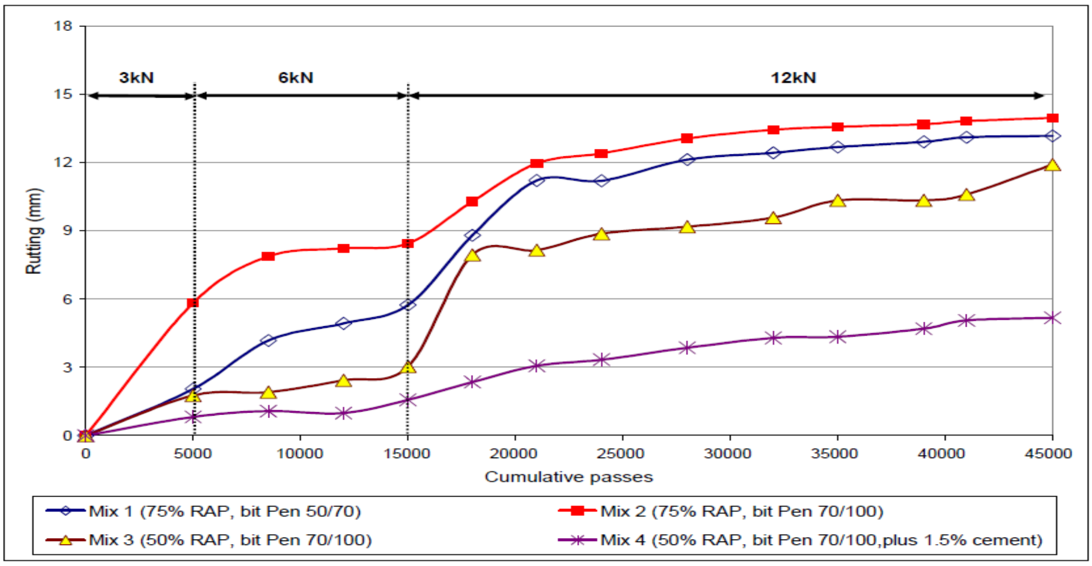


Figure 2-54 Permanent deformation behaviour of different mixes [3]

2.5 Summary

The process of bitumen foaming was originally invented by Dr. Ladis Csanyi of Iowa State University in the 1950s. In the 1960s Mobil Australia acquired the patents of the process and modified Csanyi's process of foaming (injecting steam into hot bitumen). Mobil Australia's modified process was injecting water into hot bitumen. The patent expired in the 1990s from which time FB treatment has gained popularity. The most widely accepted process of foaming bitumen is Wirtgen's process of injecting a small quantity of water (1% to 5% of the mass of bitumen) and compressed air into hot bitumen (140°C to 180°C). During the mixing process, bitumen coats a fraction of fine aggregate particles and forms mastic. In compacted FBM there exists a mineral filler phase in addition to aggregate skeleton and the bitumen mastic phase, whereas HMA mixes consists of the two latter phases only.

ERm and HL are the most used qualitative characteristics of FB. However, a few researchers felt that these two parameters were not adequate for complete characterisation of FB and introduced the concepts of foam decay and Foam Index (FI). Jenkins et al., (1999) [12], suggest FI as a bitumen selection criterion which seems not always to be possible. Moreover, Saleh (2004) [25], argued that FI is also an empirical parameter, as the FI value is a function of expansion of and the HL, which are empirical. He described a new approach to characterising bitumen foam quality by measuring viscosity of bitumen foam.

In HMA mix design, selecting bitumen grade and its properties are important steps. However, in FBM mix design bitumen properties are of lesser significance than bitumen foaming potential. Bitumen with better foaming potential is expected to disperse bitumen mastic uniformly all over the mix and improves mix performance. So identifying bitumen with good foaming potential is an important task. Bitumens with penetration of 80 to 150 are generally used. Harder bitumen is generally avoided as mixes with these binders are expected to have poor dispersion. It was identified that selection of optimal foaming conditions is also an important task in mix design. Of all foaming conditions, foaming water content (FWC) and temperature are found to be important. Usually trial water contents between 1% and 5% by weight of bitumen are applied to find optimum foam characteristics. Bitumen temperature is varied between 140°C and 180°C.

The presence of bitumen mastic plays an important role in strength and stiffness behaviour of FBMs. The presence of fines enhances the ability of the foam to produce better mix by foaming mastic (bitumen + filler). As the fines (<75 micron) content defines the amount of mastic formation, the presence of a high percentage of fines is recommended. However, the soaked strength of the mix may also have to be considered. As it is assumed that aggregate interlocking is major factor in FBMs which are not fully bonded, angularity of the aggregate needs to be considered in the mix design. It is a very common practice in FBM mix design to add cement which gives early strength and allows early traffic. Care should be taken that cement addition should not negate the benefits (flexibility) that FB gives.

During mixing of aggregate with FB and compaction, the available water acts as a lubricant and improves workability and compactability of the mix. Therefore, mixing water content and compacting water content are considered as significant mix design parameters. The compacted mixes have to be cured to gain strength and these curing conditions need to simulate those conditions which are expected in the field. As the ITS test and UCS test represent two different properties of the mix, a mix design has to accommodate these two tests in dry and soaked conditions.

Bitumen content, amount on a fines in the mix, curing conditions and presence of active filler are important factors that affect strength characteristics. The presence of active filler does not affect the optimum binder content (OBC). However, RAP addition to the mix changes the OBC. For stiffness measurements on a FBM various test modes such as indirect tensile, compressive and flexural modes have been used. Of all these test methods, ITSM is the method most commonly used. However, stiffness values were found to be unrealistically high. The reason for these higher stiffness values may be the use of unrealistic Poisson's ratio in the stress determination. In addition to test methods, rate of loading, active filler, and temperature all affect the stiffness values. Hydrated lime and quick lime appear to have less influence on stiffness.

The permanent deformation behaviour of FBMs has been studied by many researchers and the same factors that influence stiffness also affects permanent deformation (PD). Addition of cementitious additives clearly improves permanent deformation resistance. RAP addition also improves resistance to permanent deformation, up to a certain amount of addition. Although softer binder shows on effect on stiffness, mixes with harder binder show better resistance to PD. Fatigue behaviour of FBMs with cementitious additives has to be studied.

It seems there is not enough evidence for the two phase failure mechanism of effective fatigue and PD proposed by Chen et al., (2006) [94], although rutting and cracking are the primary distress modes of FBM layers. However, the rate of distress depends on many factors. Most of the test pavements with FBM layers used accelerated pavement testing (APT). All APT tests showed a decrease in modulus and an increase in deflection with time.

However, these APT tests commonly do not give enough time to cure the materials fully. Those tests that were conducted on in-service pavements showed decrease in deflection of the pavements and an increase in back calculated modulus with time.

3 REVIEW OF MIX DESIGN METHODS

3.1 Introduction

Increasing use of FB technology demands a standardised mix design methodology. A standardised method not only guides engineers, but also helps in comparison of mixes laid in different conditions. Extensive research and attempts to standardise FBM design were done in South Africa and Australia. This section aims to report FBM design methods that are followed internationally to identify areas that can be improved.

3.2 Mix design methods

Although other sources were also consulted and referenced, the following mix design procedures (or) documents were reviewed in detail.

1. California Department of Transportation (CalTrans) Guidelines [102]
2. Wirtgen Cold Recycling Manual [8]
3. Asphalt Academy Technical Guidelines [10]
4. Queensland Department of Transport and Main Roads method [103]
5. Austroads Guide to Pavement Technology Part 4D: Stabilised Materials [104]

3.2.1 Bitumen properties

In general Foamed Bitumen Mixtures (FBMs) contain less bitumen than HMA. Moreover, in FBMs the selection criteria are mostly based on foamed bitumen characteristics rather than bitumen properties and there is also no guarantee that the same grade of bitumen from different sources yields same foaming characteristics. Therefore as long as stability of the mixture is not compromised, in FBMs the bitumen properties are not as crucial as in HMA mix design. For this reason some agencies like Caltrans [102] do not specify the required properties of the virgin bitumen; bitumen selection is made solely based on foam characteristics. However, harder grades of bitumen are usually recommended to be avoided as they tend to give poor foam characteristics and only modest dispersion in the mixture. Wirtgen [8] and Asphalt Academy [10] recommend a minimum penetration value of 80 dmm to be used for foaming while corresponding maximum values were limited to 100 dmm and 150 dmm respectively to avoid poor stability of the mixtures. Queensland department [103] recommends the slightly harder Queensland class 170 bitumen (penetration around 70 dmm) to use for foaming, however with the addition of 0.5% of foaming agent to obtain the required foaming properties. The Austroads [104] requirement for bitumen is that FBMs should have a minimum of 3% residual Class 170 bitumen in them.

3.2.2 Foaming conditions and Foam Characteristics

Bitumen temperature and Foaming Water Content (FWC) (by ratio of water injection rate into bitumen to bitumen discharge rate) are the foaming variables that are recommended by all agencies to optimise the foaming characteristics (expansion ratio and half-life).

Though each agency proposes to consider a range of temperature or FWC based on experience, they give suggestions regarding these ranges to assist inexperienced users. Wirtgen and Asphalt Academy recommends three bitumen temperatures (160°C, 170°C and 180°C) and three FWCs (2%, 3% and 4%) a total of nine combinations where foaming characteristics are to be determined. However, the Caltrans method requires considering sixteen combinations (bitumen temperature: 150°C – 180°C in steps of 10°C; FWC: 2% - 5%, steps of 1%).

As a general trend, expansion ratio increases with increase in bitumen temperature and FWC. Half-life has exactly the opposite trend to expansion ratio. The bitumen dispersion in the aggregate improves with increasing expansion ratio and half-life, and better binder dispersion leads to better material properties. Accordingly, the objective of the foaming study should be to identify the combination of water addition and bitumen temperature at which the optimal foam characteristics are achieved. However, as the expansion ratio and the half-life have opposite trends with respect to bitumen temperature and FWC there are no upper limits to foaming characteristics and the aim should always be to produce the best quality foam for stabilisation. Therefore the agencies specify minimum values of expansion ratio and half-life.

Wirtgen recommends minimum values of 10 for expansion ratio and 6 seconds for half-life. The optimum foaming water content is to be determined by plotting a graph of the expansion ratio versus half-life at the different FWCs on the same set of axes. The FWC is chosen as an average of the two water contents required to meet these minimum criteria. A similar approach is also followed by the Asphalt Academy to determine optimum FWC. However, it recommend different minimum criteria based on the temperature of the aggregate that is used for mixing. The method recommends a minimum expansion ratio of 8 if aggregate temperature is greater than 25°C and 10 if aggregate temperature is between 10 and 25°C. A minimum half-life of 6 seconds is recommended irrespective of aggregate temperature. Queensland Department suggest the use of foamed bitumen with a minimum expansion ratio of 10 and half-life of 20 seconds. It also recommend a FWC of 2.5%, if the minimum expansion ratio or half-life requirements cannot be achieved while all other agencies insist to reject the bitumen if it doesn't satisfy the minimum requirements.

The minimum requirements for Caltrans are an expansion ratio of 10 and half-life of 12. The optimum FWC for a specific temperature is the FWC corresponding to the point of intersection of the expansion ratio trend and half-life trend. When two bitumen types are compared, selection should be based on the product of maximum expansion ratio and maximum half-life. The bitumen with the highest value of the product of the two parameters should be selected. The Austroads recommendation for minimum expansion ratio and half-life are 15 times and 30 seconds respectively. The optimum FWC is selected by a similar approach to Caltrans.

3.2.3 Aggregate properties

Filler content (passing 0.075mm) is an important parameter that describes the suitability of aggregate for use in Foamed Bitumen recycling. Materials with a relatively large percentage of filler have been considered better because the bitumen tends to coat the filler fully and only partly coat the larger aggregate. The presence of filler enhances the ability of the foam to produce a better mix by forming mastic (bitumen + filler) and mastic has a significantly higher viscosity than raw bitumen; it acts as a mortar between the coarse aggregate particles and hence increases mix strength. Of course there exists a maximum limit for filler content beyond which the mechanical properties deteriorate. Consequently most of the agencies specify minimum and maximum limits for the amount of filler. Other aggregate properties that affect the mechanical properties and performance of FBMs are plasticity and angularity.

Wirtgen recommends the range of particles passing through the 75 micron sieve as 5 to 20 % of the total weight of aggregate. It does not mention about plasticity, angularity or durability. The Asphalt Academy gives detailed information about the aggregates to be used in the mix design. The amount of filler to be included depends on the maximum size of the aggregate in the mix. If the maximum size is 37.5mm, it limits the fine aggregate to between 4% and 10%. If the maximum size is 19.5mm, it recommends use of higher percentage (10% to 20%) of filler in the mix. It also recommends the use of material with a plasticity index less than 10. It recommends the use of hydrated lime to reduce the plasticity of the aggregates to a maximum amount of 1.5% by weight of coarse aggregate if the aggregate exceeds the recommended plasticity limits. It even proposes durability limits for the untreated aggregates.

Queensland Department recommends grading envelopes that are slightly finer than those recommended by the Asphalt Academy. It also recommends a maximum Plasticity Index (PI) of 10, but states that lime may be used. It commonly uses 1.5 and 2% of lime for the treatment of aggregates. The gradation range allowed by Austroads is similar to the Wirtgen gradation. It also allows treatment with lime of materials whose PI is greater than 10. The Caltrans requirement for the minimum amount of filler is 5% and it allows up to 20%. However if filler content is between 15% and 20% it suggests to conduct soaked indirect tensile strength test (ITS-wet) to assess the mixture. The guidelines also require the use of aggregates whose plasticity index is less than 12. Though it did not suggest any measures to control the plasticity of the granular material it requires the mix designer to address the issue.

It is understood from the review that except for the Asphalt Academy little regard is paid to durability of aggregates. Attempts have been made in Oklahoma and Western Australia to investigate FBM [41, 105], which showed that mechanical properties of foamed asphalt mixes depend mostly on aggregate interlocking rather than binder viscosity. It is also commonly stated that aggregate angularity is an excellent indicator of aggregate suitability. Though the Asphalt Academy mentioned aggregate angularity as an important factor it does not provide any recommendations.

3.2.4 RAP characteristics

In hot recycling mix design, the bitumen in the RAP is usually considered as active and it affects the additional virgin bitumen content since the recycling temperatures are significantly above the temperature required to soften the aged bitumen. The aged bitumen is usually factored in with the help of blending charts. However this consideration of aged bitumen RAP material as active is not always the case with cold recycling mix design. The reason is that the mixing and compaction temperatures in cold recycling are well below the temperature required for the aged bitumen to be active or in a fluid state. Citing this as the reason, most of the agencies (Asphalt Academy, Austroads, Queensland Department and Caltrans) consider bitumen in the RAP as inactive and suggest treating the RAP as 'black rock' which means having properties similar to virgin aggregate. However it is not always wise to consider the RAP as 'black rock'. The new Wirtgen manual [9] gives some recommendations as to whether to consider the aged bitumen as active or inactive. RAP is classified as active or inactive depending on the penetration value of extracted bitumen at 25°C. The RAP material has to be considered as active in cold recycling only if the penetration at 25°C on recovered bitumen is more than 25 dmm. This is due to the hypothesis that foamed bitumen has sufficient heat energy to warm and adhere (as spots) to the low viscosity aged bitumen in the RAP. Therefore if the RAP material falls into the active class, then some adherence to the old bitumen coating individual RAP particles is expected, which leads to a more continuously bound material.

As cold recycling involves addition of water during mixing, the presence of water in RAP material obtained from the field is not always such a significant parameter as in hot recycling in which complete drying of RAP material is essential. Nevertheless the amount of water present in the material should be carefully evaluated to determine the additional water to be added to reach optimum Mixing Water Content (MWC). In view of this, all the mix design methods considered except the Caltrans method advise not to dry the aggregate (RAP + virgin aggregate if any) before foaming. The Caltrans method requires the RAP material from the field to be dried in an oven at 60°C until the water content reaches a constant value.

3.2.5 Active filler content

It is common practice to recommend addition of active filler such as cement or hydrated lime for a number of reasons such as to improve resistance to water, to achieve the required amount of filler or to increase early strength so as to accommodate early traffic. Wirtgen recommends the addition of cement or lime, which is 100 % passing the 0.075 mm sieve to material that is deficient in filler. It also proposes to add cement to achieve early strength to the stabilised layer. In that case the filler has to be replaced with cement. However, the use of cement in excess of 1.5 % by mass should be avoided due to the negative effect on the flexibility of the stabilised layer. The Asphalt Academy suggests use of cement and hydrated lime as active fillers. But, when cement is added as the active filler the amount needs be limited to a maximum of 1% by mass of dry aggregate. This limit is pushed to 1.5% if hydrated lime is added and even more can be added if aggregate plasticity needs to be modified. Queensland Department recommends hydrated lime between 1% and 2%, with 1.5% used where PI < 6% and 2% where PI > 6%. No recommendations are made in terms of cement addition. The Austroads method allows adding both cement and hydrated lime up 2% depending on the design traffic. In the Caltrans mix design method cement is allowed up to a maximum limit of 2% while hydrated lime is allowed as high as 3% depending on different factors such as plasticity of the materials and design traffic on the pavement etc.

3.2.6 Optimum and mixing water content

The water content of the mineral aggregate at the time of mixing with foamed bitumen is termed as mixing water content (MWC). No mix design method recommends optimum MWC as it is generally dependent on the foaming unit and mixer used. Therefore these methods suggest determining MWC by carrying out a parametric sensitivity analysis with reference to densities. The guidelines however recommend a trial water content, usually in terms of Optimum Water Content (OWC) obtained by Proctor compaction, for determining MWC. The Wirtgen and Asphalt Academy mix design methods consider different optimum water contents during mixing and compaction. However, other mix design methods consider optimum compaction water content (CWC) is the same as optimum MWC.

The Asphalt Academy mix design assumes that the optimum MWC lies between 65% and 85% of OWC as determined by modified Proctor compaction (AASHTO:T-180-10) of the untreated material. The optimum CWC needs to be determined in the laboratory using vibratory hammer compaction. The guidelines to use vibratory hammer are provided by the Asphalt Academy. The Wirtgen method follows a similar approach to the Asphalt Academy in determining optimum MWC and CWC with a few exceptions. The major exception is that the Asphalt Academy assumes that active filler doesn’t affect the optimum water content and thus compaction tests can be carried out on untreated aggregate without any addition of active filler. The same water content- density relationships can be used if the actual mixtures contain active filler in them. However, in the Wirtgen method the water content – density relationships are to be obtained separately for materials with cement and without cement. The second difference in the Wirtgen method when compared to the Asphalt Academy method is that it requires optimum CWC to be determined by modified Proctor test instead of vibratory hammer test.

In the Caltrans mix design method the OWC of the untreated aggregate is determined using the modified Proctor test. If active filler has to be added to the final mixture the OWC is to be determined on the material with active filler. The OWC obtained is the trial water content for MWC determination. It has to be noted that CWC for the Caltrans method is the same as MWC unlike the Wirtgen and Asphalt Academy methods in which CWC and MWC are different. Austroads and Queensland Department MWC requirements are based on plasticity index of the material used. If plasticity index is less than 6% then mixing and compaction has to be carried at 70% of OWC of the untreated material as determined by the standard Proctor compaction test (AASHTO T 99). If plasticity index is more than 6% the water content should also be increased accordingly, however less than OWC.

3.2.7 Mixing and Compaction

The mixing process for FBM should be swift and effective as little time is available before the foam completely collapses once it comes into contact with relatively cold aggregates. In general, the foamed bitumen is applied directly from the foam plant to the aggregate as it is being agitated in the mixer. The key characteristics of laboratory mixes are to restrict aggregate segregation during the mixing and to produce a similar mixture to that produced in field construction. As there are different laboratory mixers available (Hobart type mixer, pug-mill type mixer etc.) and each mixer type produces different mixtures, it is vital to specify a suitable and consistent mixer to be used in any mix design method.

Mixing can be carried out either by a Hobart type mixer or pug-mill mixer if the designer follows Wirtgen guidelines. However the method requires factoring the bitumen losses up to 10% if a Hobart type mixer is used. The aggregates and active filler (if any) are mixed with water (MWC) for 10 seconds before foamed bitumen is discharged onto the aggregates and mixed for 30 seconds. The Asphalt Academy also follows a similar approach, but it doesn't recommend a Hobart type mixer stating aggregate segregation as a possible drawback with this mixer. While both Austroads and Queensland department advise to use a Hobart type mixer, the Caltrans method recommends a pug-mill, type mixer considering this mixer type to replicate the mixing in a recycling plant.

Compaction of FBM not only reduces voids and increases density of the mixture but also promotes adhesion of the bitumen mastic to the coarser aggregates. Most of the available mix design methods recommend one of three compaction methods on foamed bitumen treated mixtures; Marshall compaction (impulsive), gyratory compaction (kneading) and vibratory compaction. The Wirtgen method requires making 100 mm diameter Marshall samples with 75 blows on each face. The Asphalt Academy recommends the use of a vibratory compactor for making the foamed bitumen treated specimens. The total compaction effort (surcharge, time of compaction and number of layers) is dependent on the level of mix design that is being carried out. The method also allows use of Marshall compaction in the absence of a vibratory compactor in Level 1 mix designs where 100 mm diameter specimens are to be made. Austroads and Queensland Department suggest making FBM specimens employing Marshall compaction with 50 blows per face on 100 mm diameter samples. Austroads also permits to use gyratory compaction (80 gyrations).

3.2.8 Laboratory curing regime

Arguably curing is the most important phase of FBM mixture design. The main aim of all laboratory curing protocols is to accelerate the curing phenomenon. This acceleration of curing is generally influenced by time and temperature of curing and available humidity. Therefore a curing regime specifies the temperature and humidity (sealed (or) unsealed) condition alongside time of curing. The Wirtgen manual recommends the extracted and unsealed Marshall specimens to be cured in a draft oven at 40°C for 72 hours (3 days). The Asphalt Academy curing procedure depends on the level of mix design (Level 1, 2 or 3). Level 1 mix design requires the extracted samples to be cured at 40°C for 3 days which is the same as the Wirtgen recommendation. If the design level is either 2 or 3, a two stage curing protocol has to be followed. In the first stage the extracted and unsealed specimens are cured in a draft oven at 30°C. Thereafter in the second stage each specimen is sealed in loose plastic bags and cured for a further 48 hours at 40 °C. The wet plastic bag has to be replaced with a dry bag every twenty four hours. Austroads and Caltrans recommend oven curing of unsealed specimens at 60°C for 3 days while the Queensland Department recommends curing at 40°C for 3 days (unsealed).

3.2.9 Mechanical tests for optimising bitumen content

The cured specimens are to be tested for their mechanical properties and the results are to be plotted against the respective bitumen content that was added. The added bitumen content that best meets the desired properties is regarded as the optimum bitumen content. Each mix design method has its own recommendations. Wirtgen recommends the

Indirect Tensile Strength (ITS) test in both dry and wet condition (ITS-dry and ITS-wet). Dry condition testing requires bringing the specimen temperature from curing temperature to ambient temperature before testing. The cured samples are soaked in water at ambient temperature for 24 hours before testing to determine ITS-wet. The Asphalt Academy recommendations are very detailed and requires ITS and tri-axial testing depending on the mix design levels. Level 1 and Level 2 mix design require testing the cured specimens for ITS-dry and ITS-wet. The bitumen content is optimised using the test results. Along with optimisation of the mechanical properties, it has to be verified whether they satisfy the minimum requirements specified in the guidelines. Mix design Level 3 of the Asphalt Academy advises to use tri-axial testing to assess the shear strength of the FBM and the water resistance using purpose-built Moisture Induced Sensitivity Test apparatus. Values of cohesion, friction angle, and retained cohesion obtained from the testing need to be within the minimum limits documented.

Both the Austroads and Queensland Department methods optimise the bitumen content with reference to dry and wet resilient modulus test results. After obtaining optimum bitumen content the Queensland Department method requires performing wheel tracking tests on specimens prepared at optimum binder content. A Wheel tracking test on the cured FBM slab is performed at 25°C with a wheel load of 0.7kN. The rut depth measured after 2000 passes should be less than 5 mm to authorise the designed mixture.

The Caltrans method not only optimises the bitumen content based on mechanical tests but also consider the benefits that addition of bitumen brings to the mechanical properties and thus economic benefits. The method requires testing specimens that are with bitumen and without bitumen. The parameter that is used for optimising is $ITS_{imp@n\%}$ which is the difference between ITS-wet with n% foamed bitumen and the ITS of the untreated control mixture. For example: $ITS_{imp@3\%}$ = ITS-wet @ 3% FB – ITS of untreated controlled mixture. The mix design method requires testing of samples with 2%, 3% and 4% bitumen content. The 3% foamed bitumen mixture requires having an $ITS_{imp@3\%}$ value of at least 100kPa and is selected unless the difference is more than 20% from the corresponding values of the other mixtures. The flow chart that explains the selection of optimum binder content is presented in Figure 3-1.

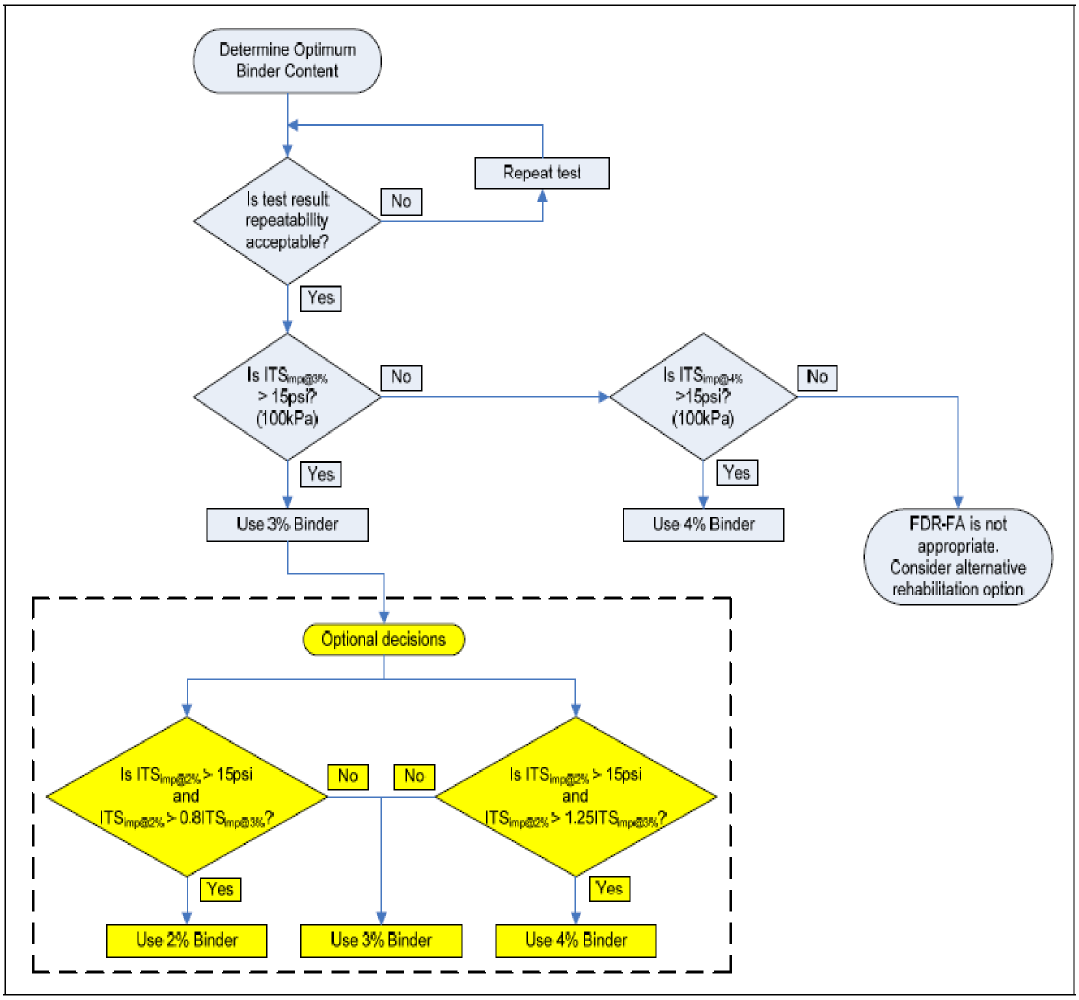


Figure 3-1 Flow chart to determine the optimum bitumen content using Caltrans method [102]

3.2.10 Summary

A comparison of the mix design procedures considered in the present study is tabulated in Table 2-1. From the reviewed mix design methods it was understood that all the methods specified relatively softer bitumen to be used for foaming since soft binders give better foaming properties than hard bitumen at a given temperature and FWC. However mixtures with harder grade bitumen have been shown to give higher resistance to permanent deformation [3]. Moreover the mix design methods also assume that the better the foam characteristics the better the mixture perform. But the research at University of Nottingham in which the effect of foam characteristics on FBM performance was studied showed that FB properties (which depend on FWC) have only a moderate effect on FBM performance while mixing protocol and binder type were found to have a more dominant effect. Furthermore, except for the Asphalt Academy and Wirtgen methods, other mix design procedures propose high limits for HL which seems to be difficult to achieve as half-life is very sensitive to FWC. A half-life of 6 to 10 appears reasonable.

In terms of filler content all mix design methods recommend minimum of 5% filler in the FBMs in view of the fact that dispersed foamed bitumen droplets only partially coat the larger aggregate. The mastic (filler, bitumen and water) spot welds the coarser aggregate fractions together in FBM. While the purpose of natural filler is primarily to supplement the filler needed for bitumen dispersion, there are many benefits with active filler addition to FBM such as early strength. In current practice there are two active fillers that are recommended, cement or hydrated lime. The selection of active filler depends on the specific role of the active filler in the FBM. For example if the aim is to limit plasticity index of the material hydrated lime is prescribed while for early strength cement is usually added. As can be seen from the Table 3-1 the maximum application rate of cement is usually less than that of hydrated lime. This is in view of the fact that though the cement addition increases the initial stiffness, in the long term high cement addition reduces the flexibility of the FBM layers and thus the benefit of bitumen addition is rarely seen.

Regarding the compaction method employed, mixtures without added bitumen are recommended to be compacted using the Proctor (standard or modified) method whereas mixtures with added bitumen are recommended to be compacted with Marshall, gyratory or vibratory compaction. Though a pug-mill mixer is assumed to better simulate the mixing carried out in the field, Austroads and Queensland Department recommend the use of a Hobart type mixer which works on a rotary mixing motion of the blenders. This motion is not ideal for restricting aggregate segregation. Most of the methods recommend a similar curing regime which is the conditioning of unsealed specimens at 40°C for 3 days and this was adopted for the curing described in Section 5.3.2. In regard to mechanical testing to select the optimum bitumen content, ITS (wet and dry) and resilient modulus tests are widely used. The Asphalt Academy and Queensland Department methods recommend performance tests alongside mechanical tests for design bitumen content selection.

Table 3-1 Comparison of the mix design procedures

Mix design method	Bitumen	Foam Characteristics		Active filler limits	Compaction for fluid consideration	Mixer	Compaction for specimen preparation		Curing regime	Mechanical tests
		Expansion ratio	Half-life				Method	Compaction effort		
Wirtgen	80 - 100 dmm	10	6	1.5%	modified Proctor	Pug-mill	Marshall	75 blows per face	3 days @ 40°C, Unsealed	ITS-dry and ITS-wet
Asphalt Academy	80 - 150 dmm	8	6	1% for cement/1.5% for lime	modified Proctor	Pug-mill	Vibratory	depends on level of mix design	depends on level of mix design	depends on level of mix design
Queensland	Class 170	10	20	2%	standard Proctor	Hobart type	Marshall	50 blows per face	3 days @ 40°C, Unsealed	resilient modulus
Austroads	Class 170	10	30	2%	standard Proctor	Hobart type	Marshall/ Gyratory	50 blows per face/ 80 gyrations	3 days @ 60°C, Unsealed	resilient modulus and wheel tracking test
Caltrans	N/A	10	12	2% for cement/3% for lime	modified Proctor	Pug-mill	Marshall	75 blows per face	3 days @ 40°C, Unsealed	ITS-dry and ITS-wet

4 MATERIALS

4.1 Introduction

The physical properties of the materials used within this study are discussed in this chapter. Alongside the bitumen and virgin aggregate, particular attention will be given to reclaimed asphalt pavement (RAP) characterization. This is important as RAP characteristics have considerable effect on the mix design of cold bitumen mixtures (CBM) because of the amount of variability associated with RAP in terms of source, production, storage and usage. However, it has to be noted that studies have found that RAP is less variable that virgin aggregate if RAP storage or stockpiling of RAP is well managed and that bituminous mixtures produced with high RAP content are actually less variable [106].

It is a known fact that in mix design of HMA containing RAP, the aged bitumen in the RAP is often considered as an active component during the mixing and the bitumen in the new bituminous mixture is adjusted using blending charts. This approach is rational as the mixing of HMA is usually carried out at temperatures above 140°C where the aged bitumen in the RAP is less viscous. However, this is not always the case in CBMs containing RAP, in which mixing and compaction is carried out at ambient temperatures which are much lower than the temperature required for softening the aged bitumen. Hence, each of the mix design method treats the RAP differently. Some agencies as discussed in Chapter 2, factor the contribution of the aged bitumen present in RAP while others do not. This conflicting consideration is due to the unknown effect of the properties of aged bitumen in the RAP on the properties of the added fresh bitumen and on the amount of bitumen to be added. To address these issues research is ongoing under the initiative of the CR (Cold Recycling) task group (TG6) of RILEM (TC-237 SIB). Most of the tests that are performed on RAP were part of the inter laboratory round robin testing programme on RAP characterization as a part of TG6.

4.2 Bitumen

In HMA mix design, the expected traffic and the regional climate influence the selection of the bitumen type. However in FBM mix design, foamability (foaming potential) of the bitumen and the mixture compactability also need to be considered during selection of the bitumen. In the present study a 70/100 penetration grade bitumen (90dmm penetration at 25°C and softening point of 45°C) supplied by Shell was used. The properties of the bitumen used in this study are tabulated in Table 4-1.

Table 4-1 Properties of 70/100 bitumen used in the study

70/100 grade bitumen properties	
Specific gravity (BS 2000-549:2007)	1.03
Penetration Index at 25°C (BS-EN1426:2000)	90
Softening Point (°C) (BS 2000-58:2007)	45
Viscosity at 135°C(mPa-s) (BS EN 13302: 2003)	321

4.3 Virgin aggregates

The virgin mineral aggregate used in this study was carboniferous limestone from Dene quarry, Derbyshire, UK. The aggregates were stored separately in stockpiles of size fractions of 20mm, 14mm, 10mm, 6mm, dust (0.063mm < dust > 6mm) and filler (<0.063mm). The stocks were batched to attain the design gradation for each of the mixes. Particle size distribution was determined according to BS EN 933-1:1997. The individual gradations are shown graphically in Figure 4-1. The particle density and water absorption were determined in accordance with BS EN 1097-6:2000 and BS EN 1097-7:1999 for filler particles and tabulated in Table 4-2.

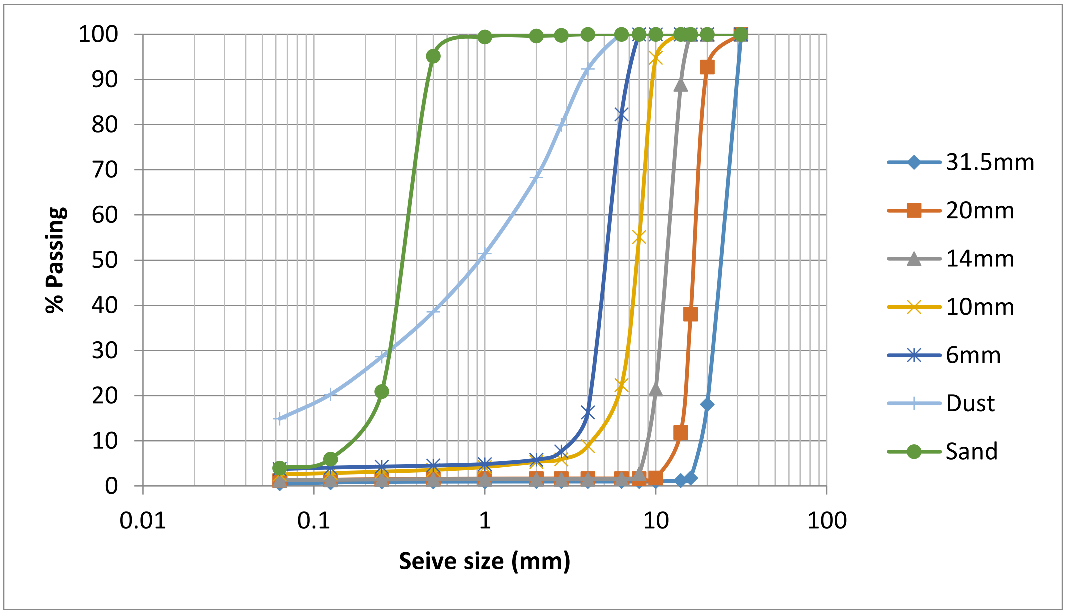


Figure 4-1 Individual gradation of each fraction of the virgin aggregate used in the study

Table 4-2 Physical properties of virgin aggregate used in the study

Particle Density (kg/m^3) and Water Absorption (%) (BS EN1097-6:2000)						
Size (mm)	20	14	10	6	dust	filler
Oven dried density	2633	2607	2608	2427	2668	2623
Surface saturated dry density	2653	2634	2640	2526	2674	N/A
Apparent density	2685	2679	2693	2696	2686	N/A
Water Absorption	0.74	1.03	1.2	4.1	0.26	N/A

4.4 Reclaimed Asphalt Pavement

The basic properties that are recommended to be measured on RAP for use in HMA mix design are aggregate gradation before and after bitumen recovery, bitumen content, bulk specific gravity of recovered aggregates and recovered binder properties. Obtaining these properties is particularly important in the mix design of CBMs as they often contain high amounts of RAP. In addition to the above mentioned tests, fragmentation and cohesion tests were recommended by CR task group (TG6). These two tests are discussed in the following sections.

4.4.1 Source and storage

The RAP material used was supplied by Lafarge Aggregates Limited obtained from Elstow Asphalt Plant in Bedfordshire, UK. The RAP was from a single source and from a well-managed stockpile before being delivered to the laboratory. The RAP aggregate material from the quarry was initially air dried at room temperature in the laboratory at 20±2°C for 24 hours and then placed in a thermostatically controlled oven at a temperature of 40°C for 24 hours and thereafter sieved into different sizes to improve the consistency of the material and to reduce variability in the RAP. These separated fractions were stored in sealed containers for further use.

4.4.2 Analysis on RAP constituents

To determine mass/volume parameters such as VMA (Voids in Mineral Aggregate), the aggregate volume properties have to be known. When RAP materials are included in the mixtures, the determination process becomes more complicated as it is necessary to calculate the bulk specific gravity of each aggregate component (virgin and RAP aggregate). Measuring specific gravity of the RAP aggregate requires extracting the aggregate, recovering the bitumen, sieving the RAP aggregate into coarse and fine fractions, and determining the specific gravity of each fraction. Extraction is the process of separating the binder off the aggregate by dissolving in suitable solvent, while recovery is a process of separating the binder from the solvent by distillation.

Before bitumen recovery, the initial gradation, which is a basic characteristic of RAP, was ascertained in accordance with BS EN 933-48 2:2012. To evaluate constituents of the RAP, a composition analysis was conducted in accordance with BS 598-102:2003. The aggregates from the RAP were extracted by centrifuge using Dichloromethane (DCM) as recommended by the standard. After extracting bitumen from the RAP, sieve analysis was carried out on the extracted aggregates. The gradation of the RAP including that of the recovered aggregate is shown in Figure 4-2. The physical properties of the extracted aggregates are presented in Table 4-3.

Once the binder was extracted and recovered from the RAP materials, its properties such as penetration and softening point were determined in the same way as for virgin bitumen. To determine the chemical composition of the recovered bitumen BS 2000 Part 143:2004 was followed 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 fully in toluene while the inorganic and waxy components come out as precipitates. The precipitates were filtered using a filter paper. After that 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. The results of asphaltene content and physical properties of recovered bitumen are presented in Table 4-3.

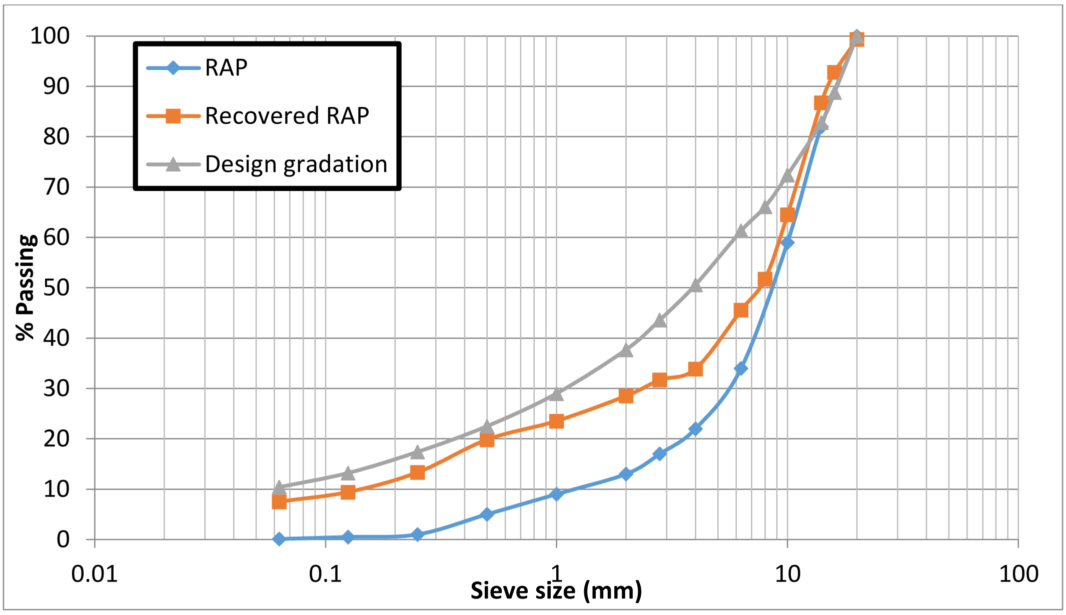


Figure 4-2 Gradation of RAP and recovered aggregate

Table 4-3 Physical properties of virgin aggregate

Particle Density and Water Absorption (BS EN1097-6:2000)		
Size (mm)	31.5mm-4mm	4mm-0.063mm
Oven dried density(kg/m^3)	2690	2720
Saturated surface-dry density (kg/m^3)	2710	2740
Apparent density (kg/m^3)	2760	2750
Water Absorption (% of dry mass)	1.06	0.44

Table 4-4 Properties of recovered bitumen from 3 samples of RAP

Recovered bitumen properties	RAP1	RAP2	RAP3	Average	Std. Dev.
Binder Content (%) (BS 598-102:2003, BS 598-101:2004)	4.5	4.7	4.4	4.5	0.1
Penetration (dmm) at 25°C (ASTM D5-05A)	20	16	17	17.7	1.7
Softening Point (°C) (ASTM D36-95(2000))	64.2	67.3	67.8	66.4	1.6
Viscosity at 135°C(mPa-s) (BS EN 13302:2003)	1077	1154	1189	1140	46.8
Asphaltene content (%) (BS 2000-143:2004)	35	N/A	N/A	N/A	N/A

4.4.3 Homogeneity of RAP

Verifying the homogeneity of RAP properties is an important step in quality control when designing bituminous mixtures with RAP. This is particularly true in cold recycling in which high amounts of RAP are often incorporated. Moreover, the mean values of the RAP properties are used to adjust the required grading curve and to select the virgin bitumen. Therefore, homogeneity of RAP in terms of gradation, bitumen content and the properties of recovered bitumen such as penetration, softening point and viscosity was evaluated. Figure 4-3 shows the gradation of different samples of the RAP before and after aggregate extraction. The figure also shows the standard deviation for each particle size for both RAP and extracted aggregates. As can be seen from the figure the standard deviation at all sieve sizes are reasonably low (maximum standard deviation is found to be 2.2%). However, it has to be noted that the extracted aggregates from the RAP were found to be less variable than the RAP before bitumen recovery as seen in Figure 4-3.

Homogeneity of RAP was also evaluated with reference to the limits suggested by NCHRP report 752 [107] and guidelines for the use of RAP in Lithuania [108]. The standard deviation of recovered bitumen properties and extracted aggregate properties along with homogeneity limits specified by the above mentioned references are presented in Table 4-5. As can be seen from the table the standard deviations are well below the specified maximum limits which suggests the homogeneity of the RAP used in the study was acceptable. It has to be noted that both the references suggest testing at least 10 samples. However in the present study only 3 samples were tested for homogeneity as recommended by RILEM TG6 technical committee.

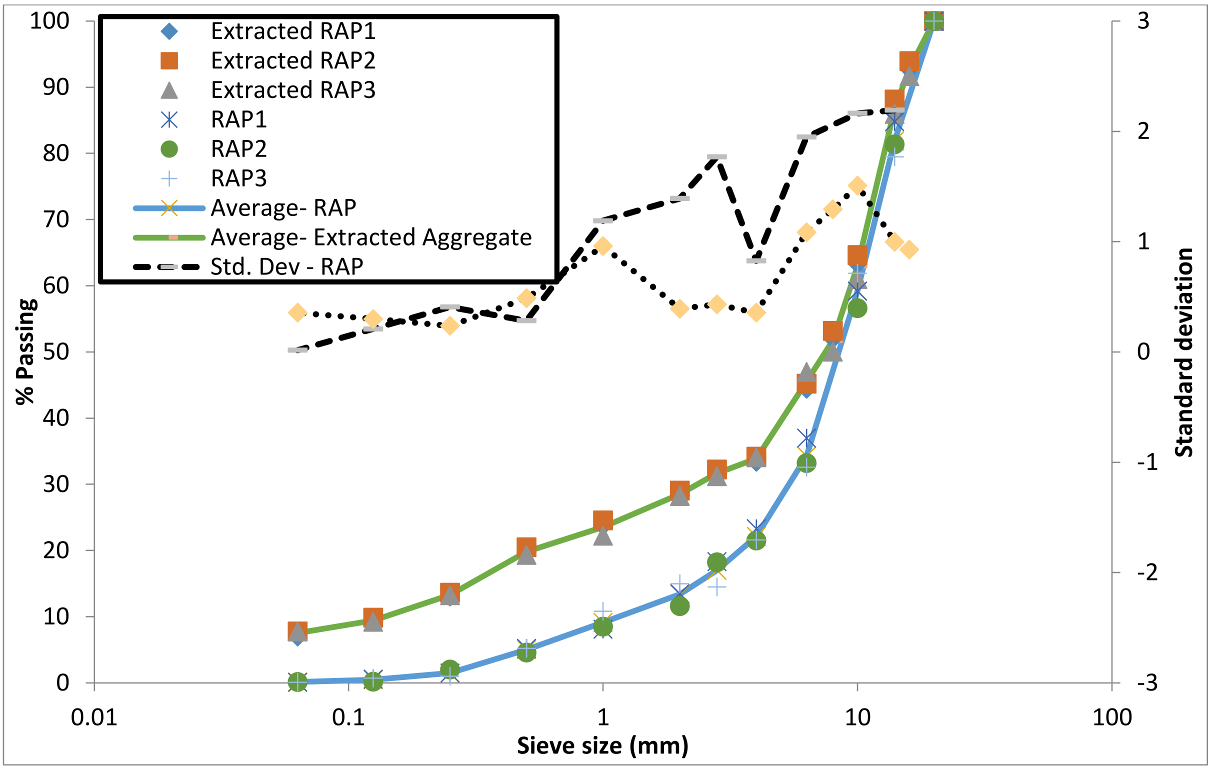


Figure 4-3 Homogeneity evaluation of RAP in terms of gradation

Table 4-5 Homogeneity limits for RAP stockpile

Properties of RAP constituents after bitumen recovery	Standard Deviation	Maximum Standard deviation	Reference
Binder Content (%)	0.1	0.5	NCHRP-752[107]
Penetration (dmm) at 25°C	1.7	4	Lithuania[108]
Softening Point (°C)	1.6	2	Lithuania
Aggregate gradation-all sieves (max)	1.5	5	NCHRP-752
Aggregate gradation-0.063mm sieve	0.35	1.5	NCHRP-752

4.4.4 Fragmentation test on RAP

The fragmentation test is an impact test which involves a normalised mass falling from a height for a fixed number of times onto the surface of the RAP and thereafter evaluating the amount of material passing the 1.6mm sieve. The coefficient of fragmentation is the ratio of the weight of the material before impact and the weight of the material passing the 1.6mm sieve after impact. The available guidelines for this test are from *French standard P 18-574: Granulats – Essai de fragmentation dynamique.* The standard requires the test to be carried out at different temperatures on the different sizes of the aggregate. As RAP includes bitumen, different results are expected at different temperatures (temperature sensitive material). The standard recommends using a 14 kg mass, lifted mechanically and allowed to fall under gravity on to the top surface of a RAP sample placed in a steel mould of 100mm diameter and 50mm height. The number of blows depends on the size of the

RAP in the mould. A similar impact test is also recommended in BS EN 1097-2:2010, which requires material to be placed in a steel cylinder and subjected to ten impacts from a hammer of mass 50 kg freely falling from 400mm height. The amount of fragmentation caused is measured by sieving the tested material using five specified test sieves. However in the present case modified Proctor compaction (BS EN 13286-2: 2004) which is also an impact test was employed as recommended by RILEM TG6 technical committee.

The modified Proctor compaction involves 56 blows with a standard rammer on each of 5 layers. The rammer and mould specification are as mentioned in BS EN 13286-2: 2004. The RAP was tested in different size fractions, 14mm/20mm, 10mm/14mm and 4.5mm/10mm (the first number is the size of the sieve on which 100% of material is retained; the second number is the size of the sieve on which 100% of material is passing) and at different temperatures, 5°C, 20°C and 40°C. The test was performed after conditioning the material for 4 hours at the test temperature. The results of the tests are presented in Figure 4-4. As can be seen from the figure, the coefficient of fragmentation has not followed any trend, which indicates that the test results are not temperature dependent as expected.

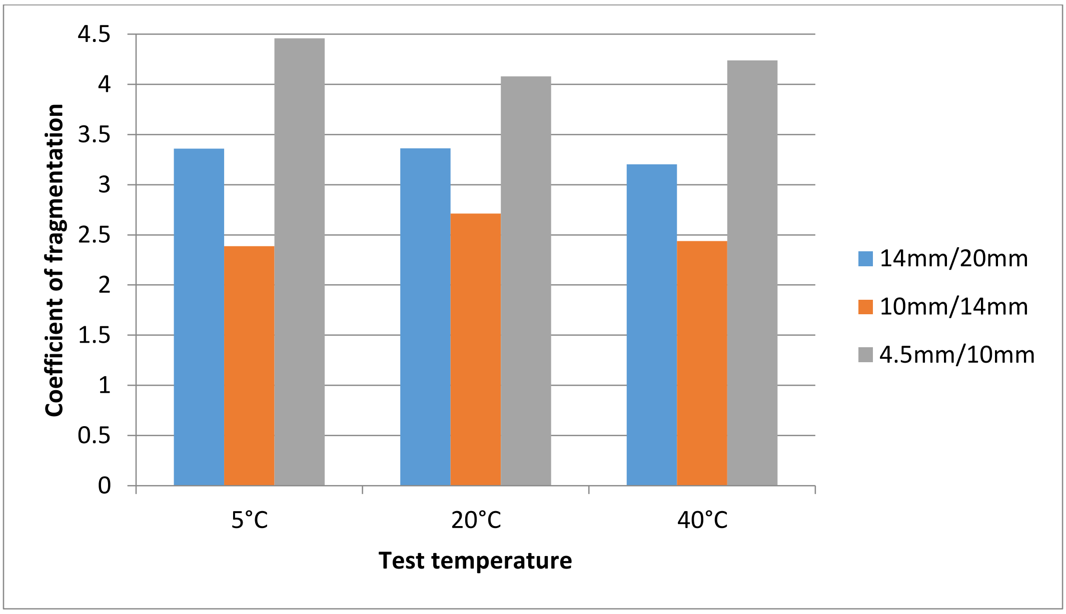


Figure 4-4 Fragmentation test results on RAP

4.4.5 Cohesion test on RAP

Further to the above tests, to ascertain if the bitumen in the RAP could be classified as "active" or "inactive", an indicative test was conducted currently under investigation by the RILEM committee. This involved conditioning a sample of RAP for 4 hours at 70°C followed by the manufacture of three 100mm diameter by 63.5mm high specimens using Marshall compaction with 50 blows per face. After compaction, Indirect Tensile Strength (ITS) tests in accordance with BS EN 12697-23 were carried out at 20°C and then in wet conditions, soaked at 20°C for 24 hours. If the soaked ITS ≤ 100kPa or the specimens do not hold together after compacting at 70°C, the RAP is considered to be inactive. For comparison, the test was also conducted with RAP conditioned at 140°C.

In all cases, the values exceeded 100kPa indicating that the binder in the RAP used in the study can be classified as active. The results are presented below in Figure 4-5.

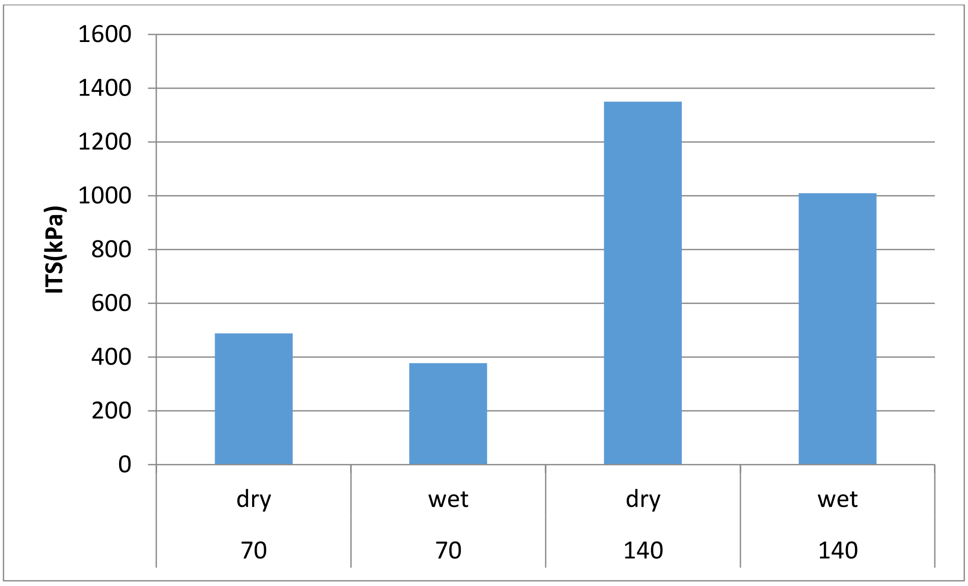


Figure 4-5 Cohesion test results on RAP

4.5 Summary

The physical properties of the materials used within this study have been presented in this chapter. Alongside the bitumen and virgin aggregate, particular attention has been given to reclaimed asphalt pavement (RAP) characterization. The tests on recovered aggregate and bitumen revealed that the RAP is well within the homogeneity limits recommended by different agencies. A cohesion test revealed that the RAP used in this study can be classified as active.

5 MIX DESIGN CONSIDERATIONS

5.1 Introduction

Unlike HMA (Hot Mix Asphalt), there is no universally accepted mix design method for FBMs. Most of the agencies [8, 10] which use FBMs have their own mix design procedures which are the result of numerous efforts over decades [7, 11, 16, 31, 39, 76, 109]. In spite of all these efforts, Foamed Bitumen application in cold recycling in the United Kingdom suffers from the lack of a standardised mix design procedure specifically using the gyratory compactor. As a result, the mix design parameters such as Foam characteristics, mixing, compaction, curing and testing that are being adopted are far from being standardised. To overcome this, research was undertaken at the University of Nottingham by Sunarjono (2008) [3] to develop a mix design procedure by identifying critical mix design parameters.

The research by Sunarjono was focussed on the influence of the bitumen type, the foaming conditions, foam characteristics and mixer type on the mechanical properties of FBM. The major outcomes of the work were recommendations for producing an optimised FBM in terms of mixer type and usage, selection of binder type, bitumen temperature, and foam characteristics. Therefore this present study focussed on other mix design parameters such as Foamed Bitumen (FB) content, mixing water content (MWC), and compaction effort. Thus, the primary objective of the present study is to propose a practical and consistent mix design procedure with emphasis on the use of the gyratory compactor.

The amount of water during mixing and compaction is considered as one of the most important parameters in FBM mix design [56, 110]. The MWC of FBM is defined as the water content in the aggregate when the FB is injected [14]. The MWC helps in dispersion of the mastic in the mix [7, 15]. However, too much water causes granular agglomerations which do not yield optimum dispersion of the mastic in the mix [18, 111]. In view of this fact many studies have been focussed on the optimisation of MWC. Lee (1981) [17] and Bissada (1987) [36] optimised MWC with reference to Marshall stability and found that the optimum MWC was very much dependent on other mix design variables such as the amount of fines and bitumen content. Sakr and Mank (1985).[41] related the MWC to other mix design variables and recommended a relationship among them to obtain optimum MWC. However, this work was performed on a FB stabilised sand mixture which did not have any coarser fractions of aggregate. Moreover, the work was based on optimising the density, without considering any mechanical properties. The concept of optimum fluid content was later borrowed from emulsion mix design in which the sum of the water and bitumen content should be close to OWC [11, 48] obtained by the modified Proctor test. This concept considers the lubricating action of the binder in addition to that of water. Thus the actual water content of the mix for optimum compaction is reduced in equal measure to the amount of binder incorporated. However, the work of Kim and Lee (2006).[31] and Xu et al., (2012) [110], who optimised MWC based on both density criteria and fundamental tests (indirect tensile strength (ITS) and tri-axial tests) on FBM Marshall specimens, calls into question the lubricating action of bitumen in the mix. Although the above discussed works are very informative, they have their limitations and little attention has been paid to optimising MWC using gyratory compaction. Therefore, the present work was aimed at obtaining a rational range of MWC for mix design with the help of fundamental tests such as ITS (BS EN 12697-23:2003) and indirect tensile stiffness modulus (ITSM) (DD 213: 1993) on FBM specimens.

Because of the presence of the water phase, the compaction mechanism of FBMs is very different from that of HMA. Various laboratory compaction methods such as Marshall compaction [11, 15, 31, 110], vibratory compaction [7, 16, 62], gyratory compaction [15, 19, 46, 64] have been used in the past. There are very well established guidelines for Marshall compaction [8] and vibratory compaction [9, 112]. However, there are no established guidelines for a gyratory compaction method for FBMs in terms of compaction effort (number of gyrations, gyratory angle and gyratory pressure). Past studies have evaluated the feasibility of using laboratory gyratory compaction on FBM (Table 5-1). In these studies efforts were made to obtain the design compaction effort in terms of compaction pressure, compaction angle and number of gyrations. The compaction pressures recommended by Australian guidelines (0.24MPa and 1.38MPa from Table 5-1) were taken forward in SHRP (Strategic Highway Research Program) work on HMA, resulting in recommendations of 0.6MPa and 1.25° angle of gyration. Jenkins et al., (2004) [64]'s tabulated conditions were based on a single water content and a single FB content. From preliminary trials it was found that the 30 gyrations recommended by Kim and Lee were too few to achieve modified Proctor densities. The ideal compaction effort has to produce mix densities that are achieved in the field. Therefore, modified Proctor density which is used worldwide to represent field compaction is used as a reference in the present study. It was understood from the past studies [3] that the permanent deformation behaviour of FBMs is sensitive to the number of gyrations, which might be attributed to the arrangement of the aggregate skeleton. Hence efforts were made to propose a design number of gyrations (N_{design}) and it was decided to use the SHRP recommended compaction conditions which are 600kPa compaction pressure and 1.25° angle of gyration. During the optimisation of MWC, the compactability of these mixtures during modified Proctor compaction and Gyratory compaction was also studied.

Table 5-1 Gyratory compaction effort on FBMs by different researchers

Summary of gyratory compaction effort on FBM by different researchers				
	Number of gyrations (N)	**Compaction pressure (MPa)**	**Compaction angle (degrees)**	**Reference density**
Brennan (1983) [15]	20	1.38	N/A	2.25kg/m^3
Maccarrone et al.1994 [19]	85	0.24	2	field density
Jenkins et al. (2004) [64]	150	0.6	1.25	Modified proctor density
Kim and Lee (2006) [31]	30	0.6	1.25	Marshall density (75 blows)
Saleh (2006b) [46]	80	0.24	2	Australian guidelines for HMA

5.2 Methodology

A detailed experimental design was prepared for the study and is tabulated in Table 5-2. The factors were selected by considering the findings of previous work done at the University of Nottingham. In the first phase of the present study, a mix design parametric study was performed on mixtures with 100% VA (100%VA-FBM) and recommendations were made. In the second phase, the proposed recommendations were validated on mixtures with 50% RAP (50%RAP-FBM) and 75% RAP (75%-RAP). In the final phase, FBM specimens with and without RAP were made according to the recommendations made in the first phase and the FB content was optimised.

As seen in Table 5-2 optimum parameters were achieved by optimising mechanical properties such as Indirect Tensile Stiffness Modulus (ITSM) and Indirect Tensile Strength (ITS-dry and ITS-wet).The ITSM is the most common test for determining the stiffness modulus of asphaltic samples via the NAT machine. As discussed in Chapter 3, the mix design methods usually recommends 100 mm or 150 mm diameter samples depending on the maximum size of the aggregate used in the mixture. Most of the mix design methods recommend a 100 mm diameter samples for mixtures with 20 mm maximum aggregate size which is the case in the present study. Moreover, the preliminary studies showed that for 150 mm diameter moulds are too big to uniformly distribute the foamed bitumen mastic with in the mixture. Therefore, the specimen dimensions 100±1 mm diameter and 60±2 mm thickness were selected in the present study. The ITSM test was carried out under the standard conditions of 5µm target horizontal deformation, 124 ms rise time, 20 °C test temperature. The samples were initially conditioned in a cabinet at 20 °C for at least 7 hours before testing (BS EN 12697-26, 2004). Five conditioning pulses are applied before starting the test to make any adaptation for the load needed to produce the target horizontal deformation, and to embed the loading plates correctly over the sample. The system then applies five load pulses to generate the horizontal deformation. Test data (horizontal stress and strains) were measured and stored automatically and the stiffness modulus was easily calculated by Eq. 5-1.

$$E = F * \frac{(\upsilon + 0.27)}{z * h} \qquad \text{Equation 5-1}$$

E: Stiffness modulus, MPa; F: Vertical load, N ; z: Horizontal deformation, mm; h: Thickness, mm υ: Poisson's ratio (0.35 was assumed)

Figure 5-1 ITSM test configuration

In the ITS test a cylindrical specimen is loaded diametrically across the circular cross section. The loading causes a tensile deformation perpendicular to the loading direction, which yields a tensile failure. By registering the ultimate load and by knowing the dimensions of the specimen, the indirect tensile strength of the material can be computed using Eq. 5-2. The test was conducted at 20°C using the INSTRON (hydraulic loading frame with a maximum load capacity of ±100 kN, equipped with four loading ranges of 10, 20, 50 and 100 kN) test equipment located at the Nottingham Transportation Engineering Centre (NTEC) in accordance with BS EN 12697-23.

$$ITS = 2P/(\pi * d * h) \qquad \text{Equation 5-2}$$

ITS: indirect tensile strength (kPa); *P:* peak load (N); d: diameter of the specimen (mm);

h: height of the specimen (mm)

Table 5-2 Experimental design for mix design parametric study

Mix design parameter	Factorial levels	Remarks
Bitumen type	90 pen (70/100 pen grade)	constant throughout the experiment
Target Foam Characteristics	ER = 10	Asphalt Academy (2009) and Sunarjono (2009)
	HL (seconds) = 6	
Foaming conditions	Temperature (°C):150,160,170	variable to be optimised
	FWC(%): 1,2,3,4,5	
Mixer type	Pug mill type mixer	constant throughout the experiment
Aggregate type	limestone	constant throughout the experiment
Aggregate gradation	20mm (maximum size)	Asphalt Academy (2009), constant throughout the experiment
MWC	% of OWC: 65,75,85,95	variable to be optimised
FB content	% of total mix: 2,3,4,5	variable to be optimised
Mechanical tests	ITS-dry, ITS-wet, ITSM	to obtain optimum MWC and optimum binder content

The role of water and bitumen during gyratory and modified Proctor compaction can be analysed by a weight-volume relationship. In the present study, Voids in Mineral Aggregate (VMA), which is an indicator for compactability, is used to understand the role of bitumen and water during compaction. VMA of a compacted specimen can be calculated by Eq. 5- 3.

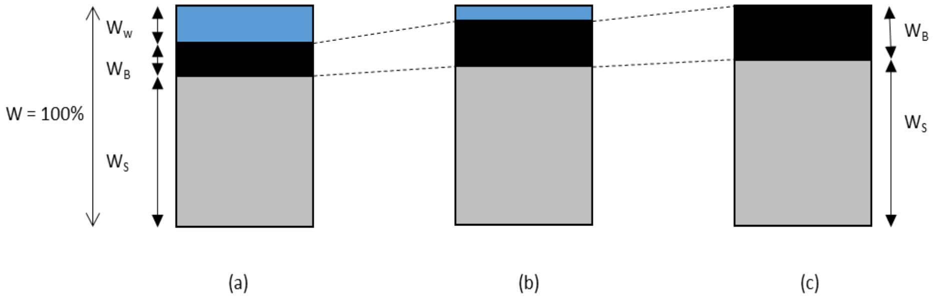
$$VMA\ (\%) = 100 - (\rho_b * P_s)/\rho_s \qquad \text{Equation 5-3}$$

Where ρ_b is the bulk density of the specimen

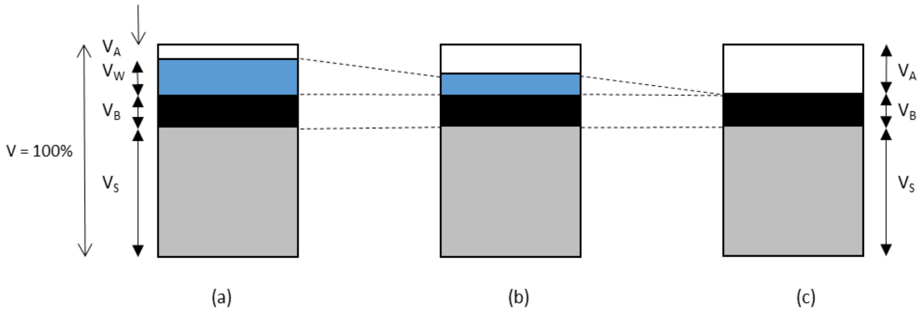
ρ_s is the bulk density of the aggregate (solids)

P_s is aggregate content by weight of mix (%)

For Hot Mix Asphalt (HMA), Eq. 5-3 can be applied as HMA has only two components, aggregate and bitumen. The weight and volume constituents remain constant throughout and volumetric relationships such as bulk density remain independent of time of test. However, for FBMs in addition to aggregate and bitumen, water also exists in the mixture. But these FBMs lose water with time as can be seen in Figure 5-2. The figure represents change in constituents (solids, bitumen, water and air) per unit weight and unit volume over time; (immediately after compaction (a), after a period of time (b) and in the dry state (c)). As can be seen in Figure 5-2 neither weight nor volume constituents remain constant with time. This is because of the presence of the water phase in these mixtures. Hence, dry density (ρ_d) was used instead of bulk density (ρ_b) in Eq. 5-1 to obtain VMA.



1. Constituents per unit weight in FBM



2. Constituents per unit volume in FBM

Constitients per unit of FBM with MWC of 85% of OWC and bitumen content of 4%						
	(a) Immediately after compaction		**(b) 48 hours at 20°C after compaction***		**(c) dry state**	
	Weight (%)	Volume (%)	Weight (%)	Volume (%)	Weight (%)	Volume (%)
Air	0	4.4	0	10.4	0	15.7
Water	5.5	11.3	2.5	5.3	0	0
Bitumen	4	8.2	4.1	8.2	4.2	8.2
Solids	90.5	76.1	93.4	76.1	95.8	76.1

Figure 5-2 Change in weight and volume constituents per unit of FBM; Note: Figure is not to the scale.

** First 24 hours in gyratory mould at 20°C*

5.3 Experimental results

5.3.1 Foamed bitumen characteristics

There have been numerous studies that have quantified the quality of FB using characteristics such as Expansion Ratio (ER), Half-Life (HL), minimum foam viscosity and foam index. In the present study it was decided to use the conventional and well documented characteristics; ER and HL. The expansion ratio is a measure of the viscosity of the foam and will determine how well the bitumen will disperse in the mix. It is calculated as the ratio of the maximum volume of foam relative to the original volume of bitumen. The half-life is a measure of the stability of the foam and provides an indication of the rate of collapse of the foam during mixing. It is calculated as the time taken in seconds for the foam to collapse to half of its maximum volume [112].

Foamed bitumen was produced using a laboratory mobile foaming plant type Wirtgen WLB 10 in which the bitumen was foamed at a water pressure of 6 bars and an air pressure of 5 bars. The characteristics of foamed bitumen (ER and HL) were obtained by applying different foaming water contents (FWC) (1% to 5% of the amount of bitumen by weight) and temperatures (150°C, 160°C and 170°C). Figure 5-3 shows the effect of FWC on expansion ratio and half-life respectively. A minimum half-life of 6 seconds and expansion ratio of 10 were adopted as selection criteria. These foaming conditions were used throughout the remainder of the study. It can be observed from the plots that as foaming temperature increased from 150°C to 170°C, the expansion ratio increased while the half-life decreased. These observations are fully in accordance with other published experience. To ensure adequate levels of both ER and HL a foaming temperature of 170°C was adopted and the procedure for obtaining optimum FWC recommended by the Asphalt Academy was used, giving 3%.

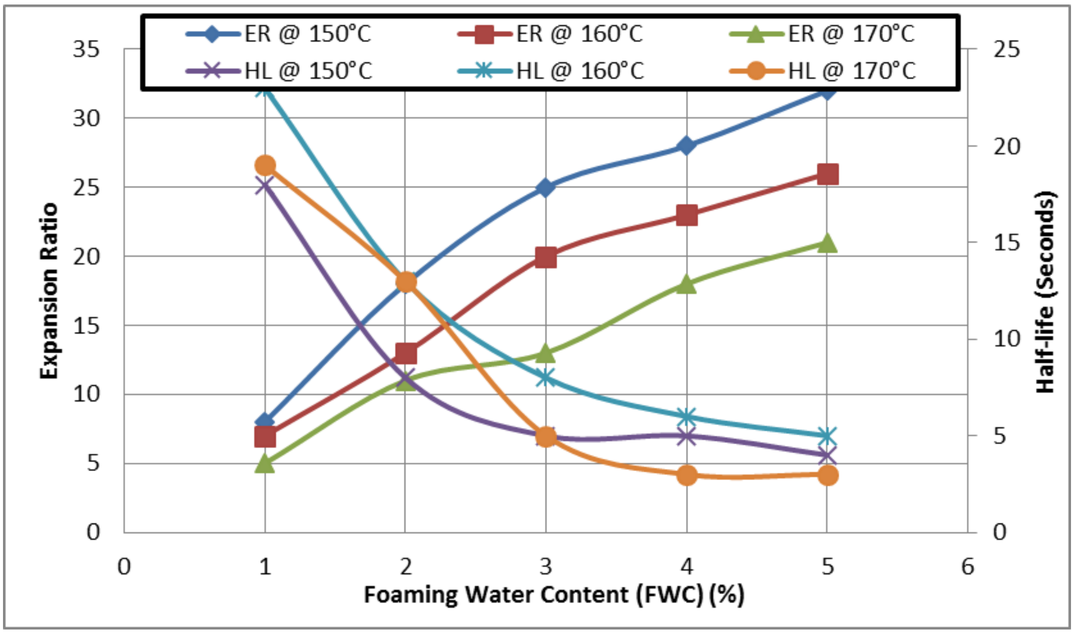


Figure 5-3 Effect of FWC and temperature on ER and HL

Foamed Bitumen begins to collapse rapidly once it comes into contact with relatively cold aggregate. Therefore, the mixing process should be a dynamic one. Consequently FB is most often applied directly from the laboratory foaming plant to the aggregate as it is being agitated in the mixer. As different mixers can produce up to a 25% difference in strength [112] selection of an appropriate mixer is very important in production of FBM. It is always recommended to utilise a mixer that simulates site mixing. From the literature it was found that most of the research was carried out using a Hobart type mixer (blender type) [17, 41]. However, pug mill drum mixers and milling-drum mixers are the most commonly used mixers on site for the production of FBM. These mixers provide sufficient volumes in the mixing chamber and energy of agitation to ensure better mixing [7]. A pug mill type mixer is therefore recommended for production of FBM representative of the field [61]. Hence, a twin shaft pug mill mixer was adopted in this work (operated at 20±2°C). Mixing time should be in accordance with the time required by the bitumen foam to collapse. In the laboratory a mixing time of 60 seconds is recommended [36] which is longer than in situ mixing but simulates the difference in the energy of the laboratory mixer and field plant and the same (60 seconds mixing time) was adopted in this study.

5.3.2 Mixing Water Content (MWC)

The optimisation of MWC was carried out on specimens compacted using the gyratory compactor to densities that were obtained by modified Proctor compaction. Targeting modified Proctor densities meant that all specimens were compacted to the same compaction effort. This approach was considered suitable as it is not appropriate to compact mixtures with different water contents to the same density as they would need very different compaction efforts. For example, mixtures with 100% of OWC (6.5% by weight of mixture) needed 200 gyrations to compact to MDD while a mixture with 65% of OWC (4.25% by weight of mixture) required around 340 gyrations. Hence, modified Proctor compaction was carried out on aggregate and water mixtures in accordance with BS EN 13286-2: 2004. The results of the modified Proctor compaction can be seen Figure 5-4, including results of modified Proctor compaction on mixtures with RAP. As can be seen from Figure 5-4, the OWC for 100% VA mixtures was found to be 6.5% and for mixtures with RAP the OWC was around 6%.

Once OWC from modified Proctor compaction had been obtained, mixing was carried out with varying water content (65%, 75%, 85% and 95% of OWC, which corresponds to 4.2%, 4.9%, 5.5% and 6.2% water content in the mixture) and varying FB content (2%, 3%, 4%, and 5%). These mixtures were compacted using modified Proctor compaction; densities were obtained and the results presented in Figure 5-5. After obtaining the densities, these possible combinations of mixtures were mixed and compacted using a gyratory compactor (angle of gyration 1.25° and compaction pressure 600kPa) using different numbers of gyrations to obtain the achieved modified Proctor densities. Gyratory compacted moulds after compaction were kept at room temperature for 24 hours and then the specimens were extracted. The extracted specimens were cured at 40°C and the water content of the specimen was monitored over time. Mechanical tests were carried out on the cured specimens after 3 to 5 days depending on the amount of water in the specimen. The tests were carried out on all specimens at approximately the same water content (between 0.6% and 0.65%) to eliminate the effect of water content on the measured mechanical properties. The effect of mixing water content on the mechanical properties can be seen in the plots in Figure 5-6.

The mechanical properties (ITSM, ITS-dry and ITS-wet) of gyratory-compacted and cured specimens are plotted against MWC in terms of % of OWC in Figure 5-6. Each ITSM value in the plot is an average of tests on 8 specimens and ITS-dry and ITS-wet are averages of 4 specimens. The properties were all measured at the same water content (0.6-0.65%). As can be seen from the figures, the approximate peak ITSM values were 85% of OWC, except for 2%FBM (FBM with 2% FB content). When ITS-dry results were considered, the optimum MWC was seen at 85% of OWC for 2%FBM and 3%FBM; and for 4% FBM and 5% FBM the peak was at 75%. For ITS-wet values the optimum was found at 85% except for 5% FBM. Overall, the optimum MWC for all mixtures was consistently found to lie between 75% and 85% of OWC.

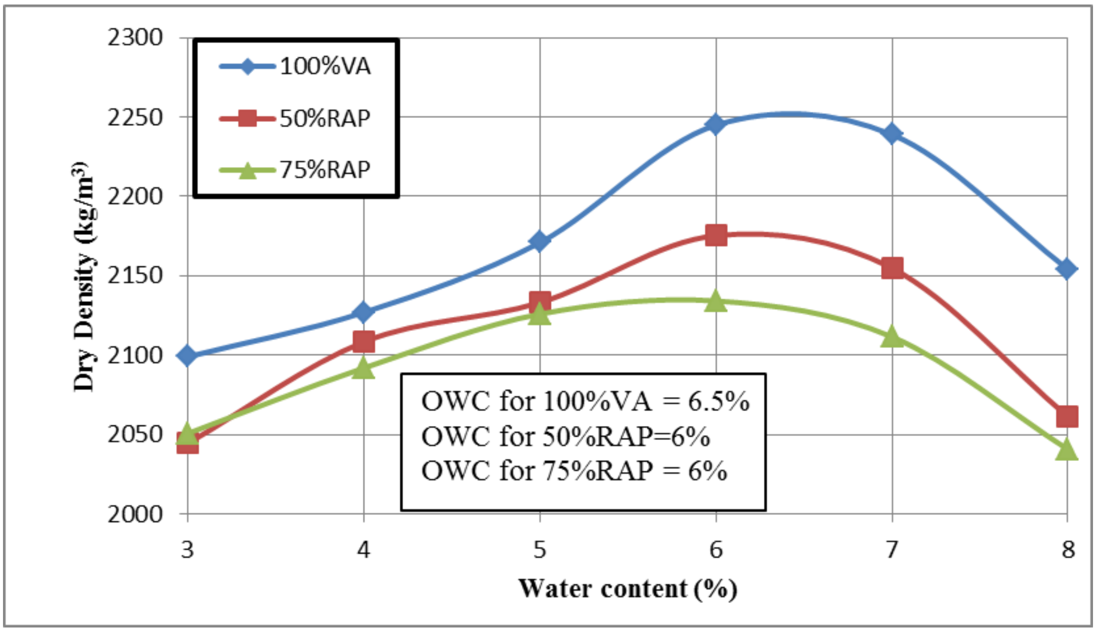


Figure 5-4 Modified Proctor test results on aggregate and water (only) mixtures

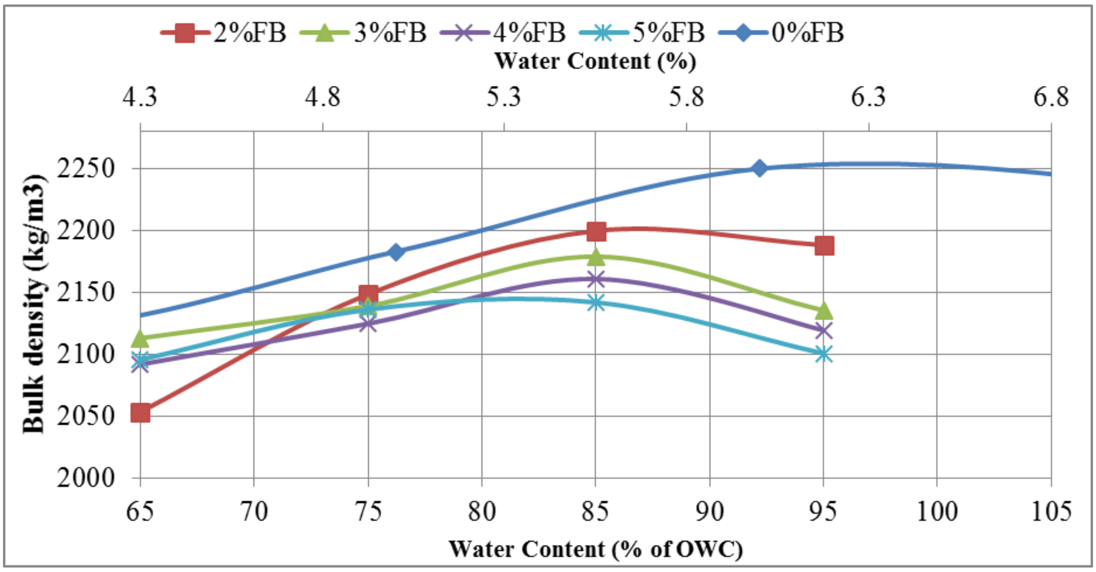


Figure 5-5 Modified Proctor compaction results on 100%VA-FBM with varying FB and water content

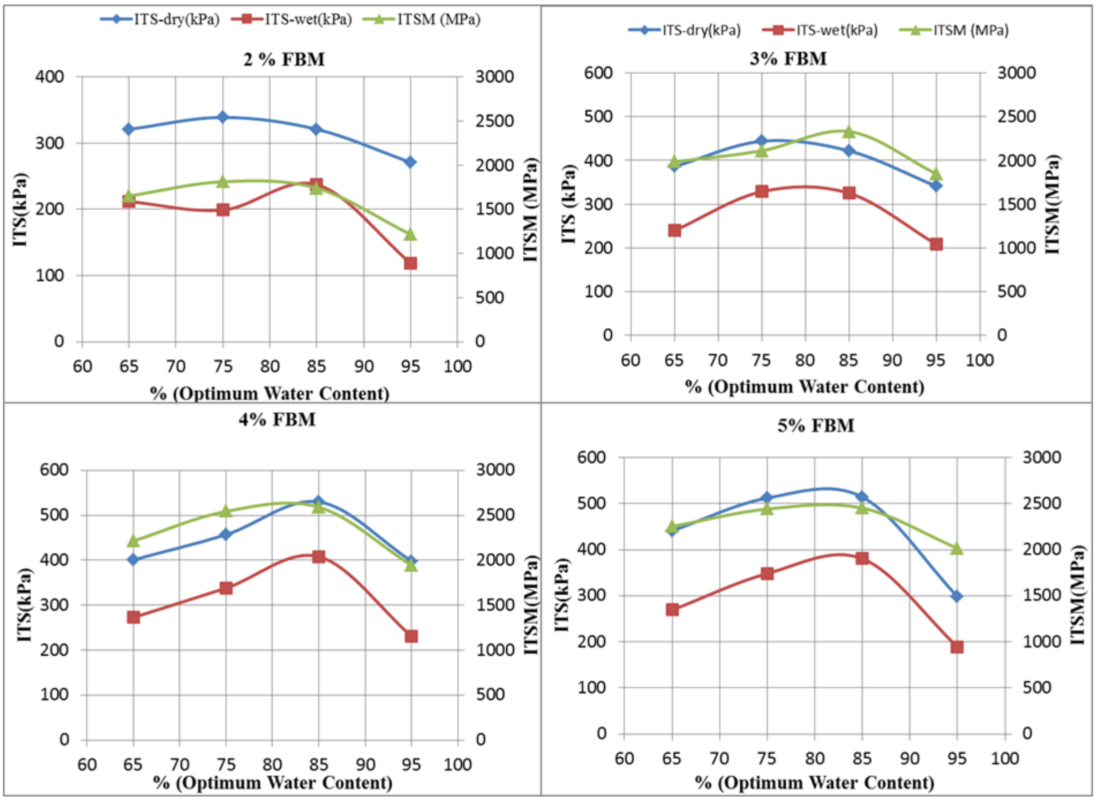


Figure 5-6 Mechanical properties of 100%VA- FBM with varying FB and water content

5.3.3 Gyratory compaction effort

As discussed in the earlier sections, one of the objectives of this study is to propose a design number of gyrations (N_{design}) for FBM mix design. For this, aggregate mixtures with 80% of OWC (based on the 75% to 85% range established above) and different FB contents were prepared. Then the mixtures were compacted to 200 gyrations and densities were plotted against number of gyrations as shown in Figure 5-7. From the data, the number of gyrations required to reach modified Proctor density was identified as can be seen in Figure 5-7. To study the optimum compaction effort and to obtain the design number of gyrations (N_{design}), the changing height was recorded from the gyratory compactor during compaction. From the height data, density was calculated and plotted against number of gyrations (Figure 5-7). The marks on the curves are the target densities that were obtained from modified Proctor data. It can be seen from the plots that, though the target densities were different, the number of gyrations required to compact to those target densities are in a similar range. That means, a design number of gyrations required to compact to modified Proctor density can be established, independent of FB content in the mixture. N_{design} for all FBMs considered was in the range of 120-160 gyrations; 140 gyrations has therefore been selected as giving an equivalence to modified Proctor.

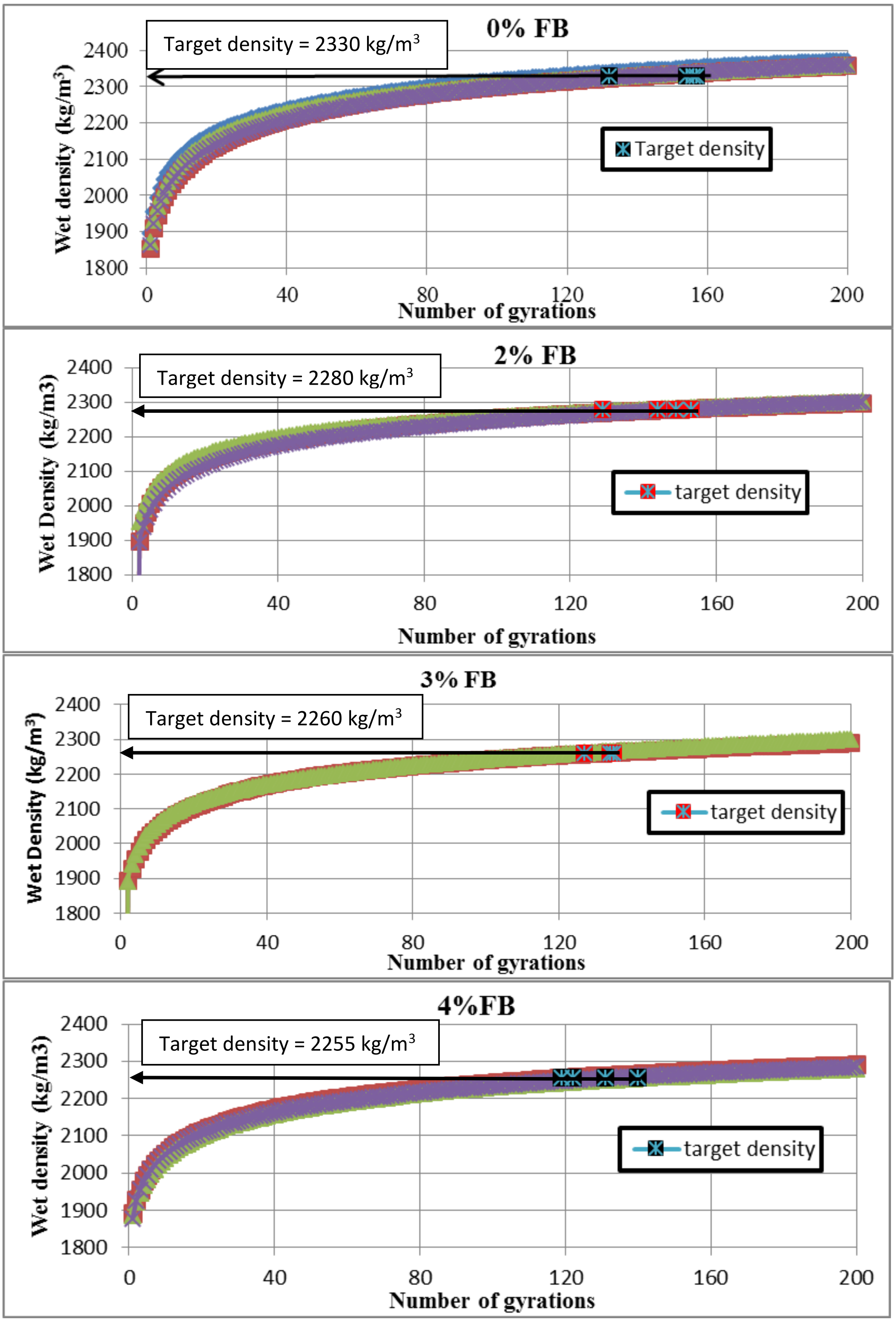


Figure 5-7 Obtaining design number of gyrations for FBM (Mixing water content of the mixture (MWC) = 80 %(OWC) = 5.2%)

5.3.4 Compactability of FBMs

The compactability of FBMs was studied on mixtures with varying amounts of bitumen and water. As discussed previously, the modified Proctor compaction and Gyratory compaction methods were considered. The study enables the role of bitumen and water with these compaction methods to be understood. As seen in Figure 5-5, from tests on modified Proctor compacted specimens, all curves give optimum water content. However, that optimum differs only slightly from one bitumen content to another, implying that the bitumen hardly contributes to the 'fluid' needed for compaction. The same effect can be seen in terms of volumetrics in Figure 5-8, where VMA is plotted against total fluid (water + bitumen). The optimum shifts to the right in steps and the shift is around 1% for the 2%, 3%, 4%, 5% FB curves, again implying negligible contribution from the bitumen.

A similar picture is obtained from the volumetrics of gyratory compacted specimens. To study the gyratory compaction, the FBMs were compacted to 140 gyrations with an angle of gyration of 1.25°, compaction pressure of 600kPa and 30 revolutions per minute. The compactability was studied using weight-volume relationships and voids in aggregate (VMA) as calculated by Eq. 5-1. VMA at 140 gyrations for mixtures with different bitumen content is plotted against MWC (dashed lines) in Figure 5-9 (each point is an average of five data points), alongside the data from modified Proctor compaction (solid lines). As can be seen from Figure 5-9, the VMA of the specimens at optimum was almost the same in the two cases, very slightly greater for modified Proctor compaction, and it increased as the foamed bitumen content increased. The optimum water content was also typically slightly higher in the case of gyratory compaction, thought to be due to the significant difference in the way the two compaction processes operate.

Overall however, the clear implication is that the bitumen gives minimal contribution during compaction and that this phenomenon is observed during both the compaction methods that were considered. Thus, the total fluid content, which has been successfully used in bitumen emulsion mix design [113-115], is not a valid parameter in FBM mix design.

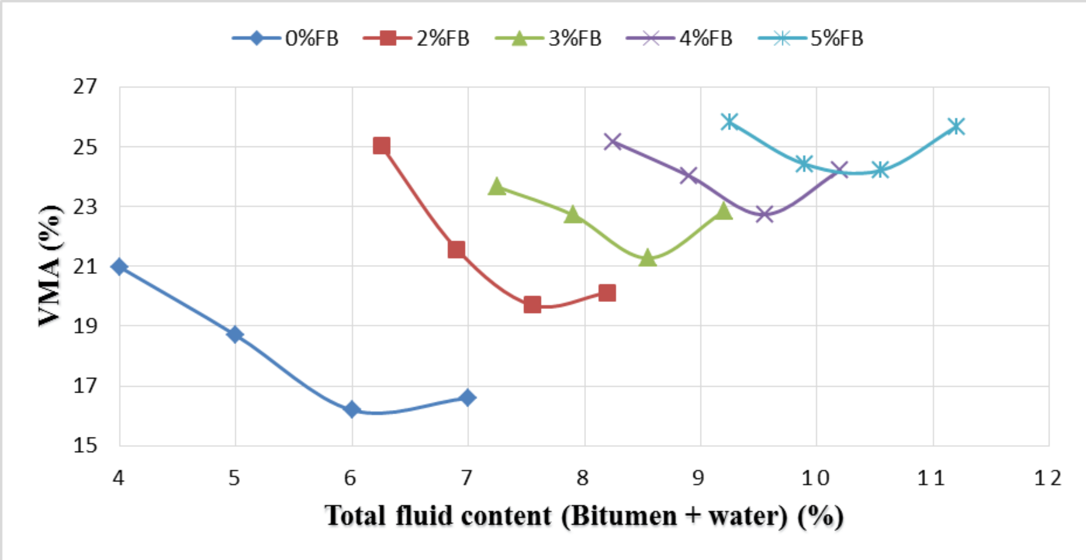


Figure 5-8 Role of combined bitumen and water during modified Proctor compaction

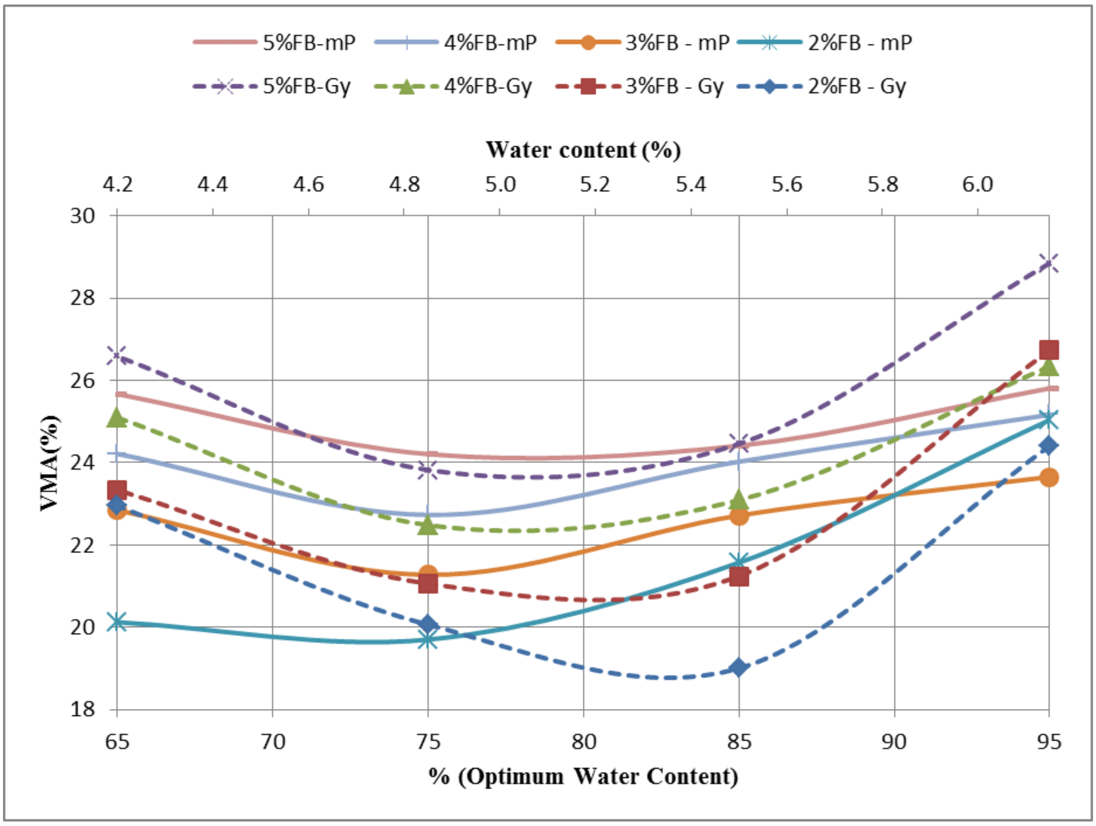


Figure 5-9 Role of bitumen and water during gyratory (Gy) and modified Proctor (mP) compaction

5.4 Validation

The mix design parametric study discussed in the previous sections was done on mixtures with 100%VA (100%VA-FBM). In this section, a study has been conducted on mixtures with 50%RAP (50%RAP-FBM) and 75%RAP (75%RAP-FBM) to validate the proposed recommendations. To validate the MWC range proposed (75% - 85% of OWC), aggregates with 50%RAP and 75%RAP and 4% FB were mixed and compacted with varying MWC (95%, 85%, 75% and 65% of OWC) to modified Proctor densities of similar mixtures. 4% FB was selected as it was the design FB content obtained for 100%VA mixes and it was assumed that the presence of RAP would not affect the design FB content (an assumption that was later shown to be incorrect). The specimens were cured as discussed for 100%VA-FBMs. The results of mechanical tests carried out on cured specimens are presented in Figure 5-10. ITSM values shown in Figure 5-10 are the average of 10 tests while ITS-dry and ITS-wet are the average of 5 tests each. As can be seen from Figure 5-10, the optima for ITSM and ITS-dry were found at 75% of OWC and 85% of OWC respectively. For 75%RAP-FBM, optimum ITS-dry and ITS-wet were found at 75% of OWC. Although ITS-wet for 50%RAP-FBM and ITSM for 75%RAP-FBM didn't showed any optimum, other properties of both the mixtures have their optimum in the proposed range (75% - 85% of OWC).

To validate the N_{design}, the aggregates with 50%RAP and 75%RAP were mixed and compacted with 0%, 3%, 4% of FB and the density data is plotted in Figure 5-11. For clarity Figure 5-11 shows only data for 75%RAP-FBM with 0%FB and 3%FB; the 4% FB lies in the same region on plot. It can be seen that the N_{design} range is the same, i.e. between 80 and 120 gyrations. The mid-point of this range which is 100 was considered as N_{design}. The study conducted on 50%RAP-FBM gave N_{design} as 110 gyrations.

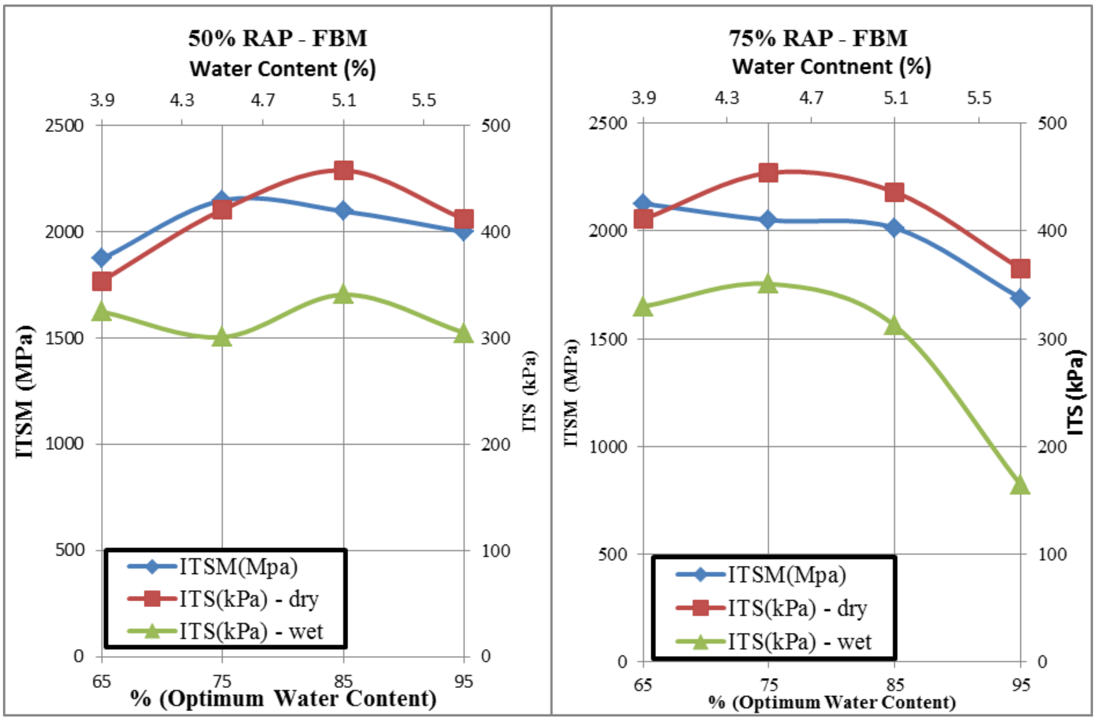


Figure 5-10 Mechanical properties on 50%RAP-FBM and 75%RAP-FBM with 4% FB content (Validation)

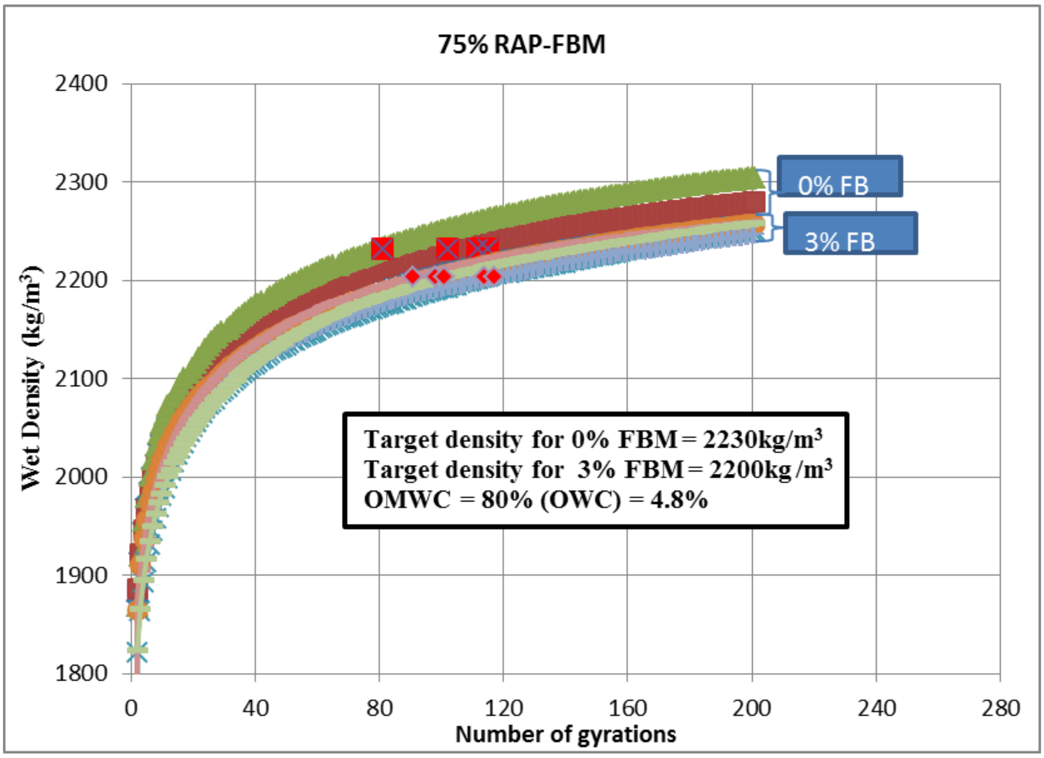


Figure 5-11 Validation of N_{design} for 75%RAP-FBM

5.5 Foamed Bitumen (FB) content optimisation

The results of mechanical tests on the mixtures that were compacted at optimum MWC (80% of OWC) and to N_{design}, and varying FB content, are plotted in Figure 5-12. As can be seen in the plots there is a clear optimum ITSM value for all mixtures. For 100%VA mixtures, the optimum was found at 4% FB content. Similarly, the optimum ITSM values for 50%RAP and 75%RAP mixtures were found at 3.5% and 3% FB content respectively. If ITS-dry values are considered, there was no optimum for 100%VA mixtures. ITS-dry values for these mixtures increase with increasing FB content without any optimum value. However, an optimum could be located for both the mixtures with RAP (50% RAP and 75% RAP). The optimum values were found at 3% FB for both mixtures. When ITS-wet results are considered, the optimum ITS-wet was found only for 75% RAP mixtures, which is at 3% FB content. There was no optimum for any mixtures if ITSR (Indirect Tensile Strength Ratio) was considered. However, it can be noted that, though ITS-dry values were higher for 100%VA mixtures than for mixtures with RAP, the ITS-wet and ITSR values were found to be superior for mixtures with RAP. This indicates that the mixtures with RAP have better resistance against water than mixtures without any RAP. This could be attributed to the presence of fully bitumen coated RAP aggregates in the mixture. Overall, from the results at 4% and 3% FB contents, optimum mechanical properties were found for 100%VA and 75%RAP mixtures respectively. However, optimum FB content was less clear for 50%RAP mixtures.

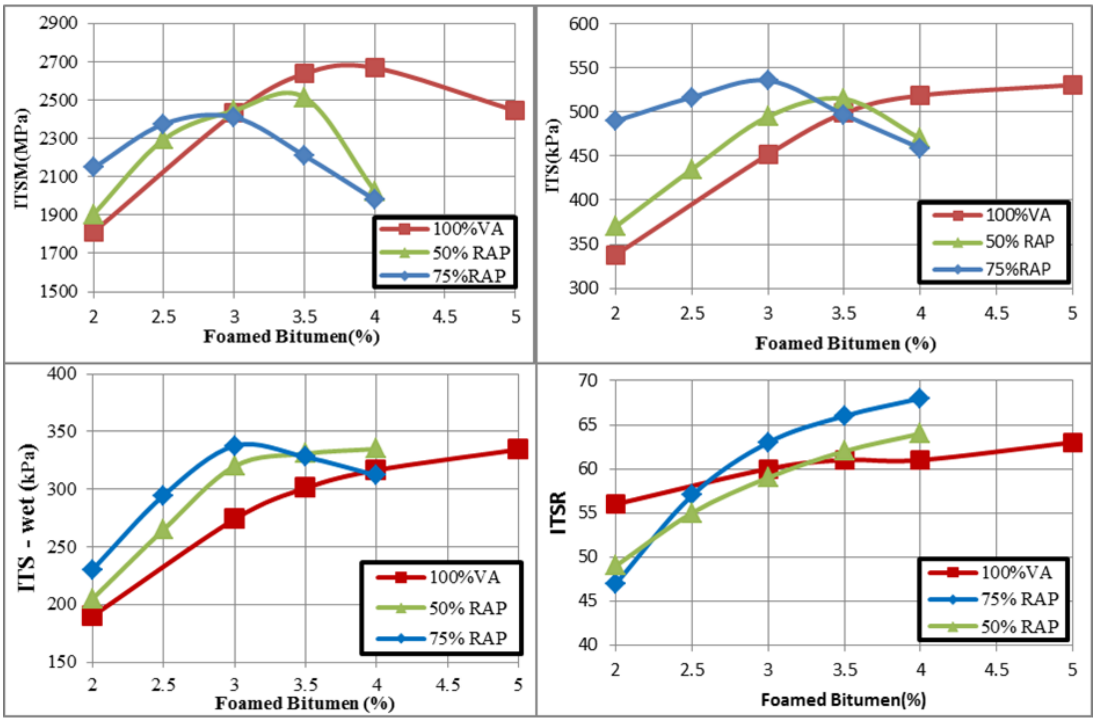


Figure 5-12 Mechanical properties of FBMs that were mixed at optimum MWC (80% of OWC) and compacted to N_{design}

5.6 Effect of aggregate temperature on mechanical properties

Temperature of the aggregate during the mixing phase of the mix design influences significantly the quality of FBM [116]. Because of this reason it has been recommended to construct pavements with FBM only if the ambient temperature is above 10°C [9, 112]. As was mentioned previously, the present experimental study mostly involved mixing and compaction at an ambient temperature of 20±2°C. However, this section has analysed the effect of aggregate temperature (which is also mixing temperature in the field) on the mechanical properties of FBM with 50 % RAP aggregate (50% RAP-FBM). The mixing was carried out at three aggregate temperatures (5°C, 20°C and 30°C). Before mixing, the aggregates were conditioned at the required temperature overnight (around 18 hours). The resulting temperature of the mixtures after foaming and mixing were found to be 10°C, 26°C and 31°C respectively for aggregate temperatures of 5°C, 20°C and 30°C. The mixtures were then compacted at an ambient temperature of 20±2°C. The mechanical tests were carried out on samples that were extracted after 24 hours and cured at 40°C for 72 hours (3 days). The results of the mechanical tests and volumetric properties of the cured specimens can be seen in Figures 5-13 to 5-16.

As can be seen in Figure 5-13 and Figure 5-15 aggregate temperature has significance influence on compaction (air voids) and stiffness (ITSM) of the FBM. The lower aggregate temperatures resulted in inferior mixture properties. Though the difference is not significant from 20°C to 30°C, the aggregate temperature of 5°C clearly resulted in higher air voids and less stiff mixtures. Similar results were also found when comparison was made in terms of strength (ITS-dry and ITS-wet) (Figure 5-14 and Figure 5-16). Moreover the retain strengths (ITSR) increased with increase in aggregate temperature, which reinforces the finding of poor mixing and compaction at lower aggregate temperature.

The major determinate for poor mixing at low aggregate temperature is the high temperature gradient between the aggregate and the foamed bitumen which influences the rate of collapse of the foam. This high temperature gradient causes rapid collapse of the foam as the film of the bitumen bubbles is thin, which allows rapid heat transfer between foamed bitumen and aggregate. Consequently, less time is available for foamed bitumen to interact with the aggregate resulting in poor coating of the aggregate particles and inconsistent dispersion of the mastic in the mixture. As can be seen in Figure 5-13 and Figure 5-15 the high temperature aggregates resulted in lower air voids in the resulting specimens. These higher densities (low air voids) could be associated with better compactability of the mixture at higher temperatures. As discussed the higher aggregate temperatures resulted in mixtures with relatively higher temperatures which helps in obtaining denser specimens [7, 18]. However, it has to be noted that the improvement in densities from aggregate temperatures of 20°C to 30°C was found to be marginal.

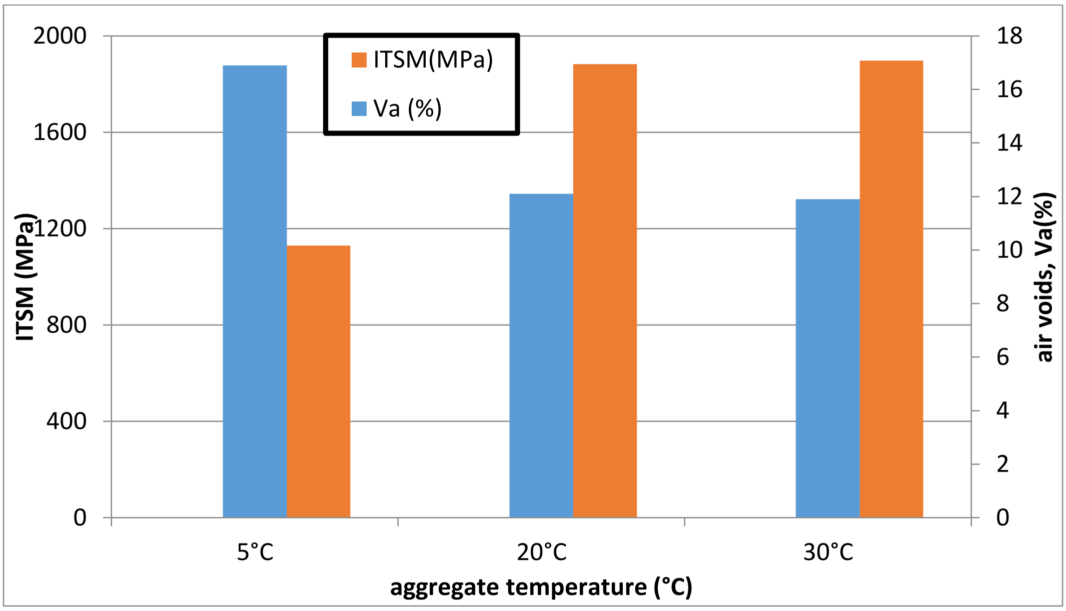


Figure 5-13 Effect of aggregate temperature on air voids and stiffness in 50%RAP - FBM

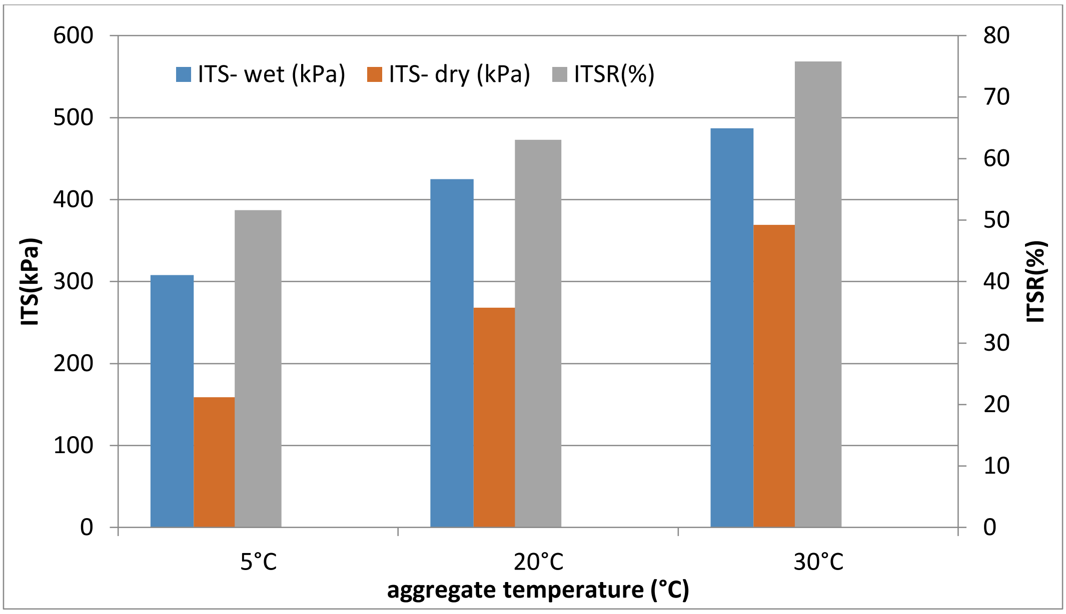


Figure 5-14 Effect of aggregate temperature on strength in 50%RAP - FBM

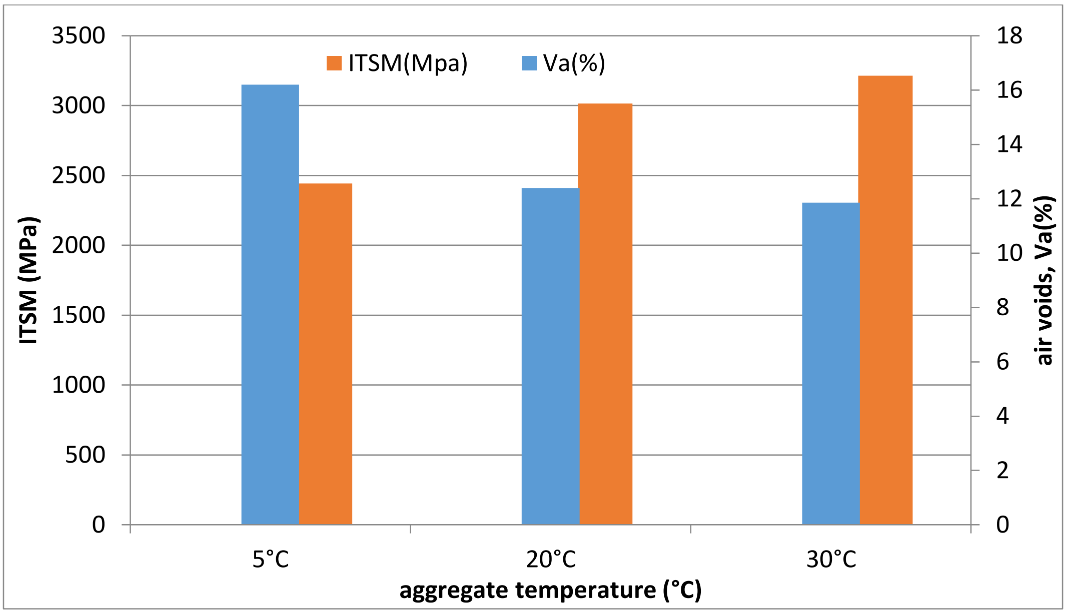


Figure 5-15 Effect of aggregate temperature on air voids and stiffness in 50%RAP + 1%Cement - FBM

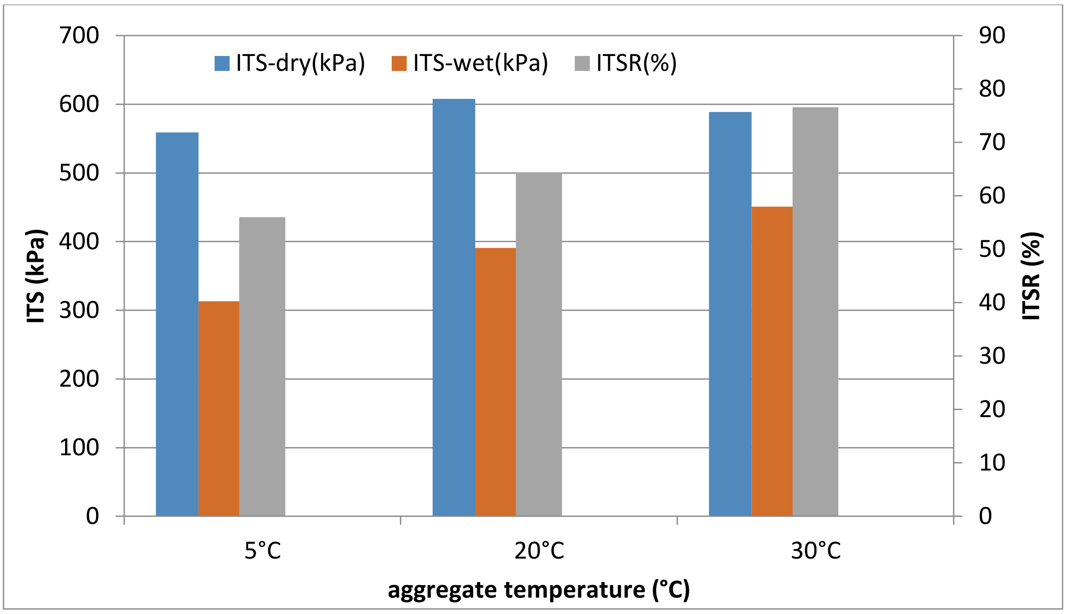


Figure 5-16 Effect of aggregate temperature on strength in 50%RAP + 1%Cement - FBM

5.7 Summary

The primary objective of this work is to propose a practical and consistent mix design procedure for Foamed Bitumen Mixtures (FBM) with the main focus being on the use of the gyratory compaction method in the proposed methodology. To attain this objective, the mix design parameters such as Mixing Water Content (MWC) and compaction effort have been optimised. This mix design parametric study was initially carried out on FBMs with virgin limestone aggregate (VA) without Reclaimed Asphalt Pavement (RAP) material and a mix design procedure was proposed. The proposed methodology was later validated on FBMs with 50% RAP and 75% RAP. Efforts were also made to optimise the Foamed Bitumen (FB) content in FBMs with and without RAP.

Optimum MWC was achieved by optimising mechanical properties such as Indirect Tensile Stiffness Modulus (ITSM) and Indirect Tensile Strength (ITS-dry and ITS-wet). A rational range of 75-85% of Optimum Water Content (OWC) obtained by the modified Proctor test was found to be the optimum range of MWC that gives optimum mechanical properties for FBMs. As this study focussed on the use of the gyratory compactor for FBM compaction, efforts were made to suggest a design number of gyrations (N_{design}) for optimum compaction of FBMs. It was found that a unique N_{design} (mixture specific) which is independent of the FB content can be established. N_{design} for the virgin mixture was found to be 140, while N_{design} for the mixtures with 50% of RAP and 75% of RAP was 110 and 100 respectively. It was also found that the presence of RAP influenced the design FB content, which means that treating RAP as black rock in FBM mix design is not appropriate.

One of the objectives during mix design optimisation is to understand the role of water and bitumen and their interaction during mixing and compaction of the mixes. To study the influence of bitumen and water, modified Proctor compaction and gyratory compaction were employed on mixes with varying amounts of water and bitumen. This work also evaluated the validity of the total fluid (water + bitumen) concept which is widely used in bitumen-emulsion treated mixes.

6 ACCELERATED CURING STUDY

6.1 Introduction

This chapter discusses the simplest yet crucial mix design consideration of FBMs; curing. Curing is the process by which FBMs gain strength or stiffness with expulsion of water. Despite the fact that a wide range of studies have been undertaken on curing, considerable issues still need to be addressed. This is because of the complexity that water brings to these mixtures. Water plays different roles during different stages of mix design. Its existence aids the mixing and compaction phases, but negatively affects the early strength. In addition to the climatic parameters like temperature, humidity and wind, other factors such as aggregate gradation and properties, layer thickness, initial water, binder, cement (or) additive content, layers applied on top, drainage condition, and finally traffic load influence the curing process [117].

A lot of research has been carried out on development of a laboratory curing protocol for these mixes to assess when to carry out the test on a mixture for its selection. The curing protocol is especially important in the structural capacity analysis of pavement design which is based on the laboratory measured stiffness values. Therefore, a laboratory mix design procedure needs to simulate the field curing process in order to correlate the properties of laboratory prepared mixes with those of field mixes. Simulation of the environment to which the pavement material will be exposed after compaction is the best approach. However, this approach is too complicated and time consuming as the process takes a long time and predominantly depends on the climatic condition that the material is exposed to in the field. An accelerated laboratory curing procedure which is curing at elevated temperature is the best available option.

Various accelerated laboratory curing and conditioning regimes exist for FBMs and are used in a variety of ways by researchers and in material specifications although the underlying question remains: how realistic are these curing and conditioning regimes in simulating pavement conditions as experienced in the field? Most accelerated curing regimes fail to reproduce the actual condition of the mix that is expected in the pavement and tests are therefore conducted at more severe conditions (i.e. higher temperatures). This results in overestimating the mix performance in the field. This is especially true when the fundamental properties of the FBM are compared with other mixes such as HMA and Warm Mix Asphalt (WMA). Moreover, considerable variation among different conditioning regimes is evident. Therefore it is necessary to develop a suitable curing protocol and establish criteria to evaluate the protocol which specifies how long a laboratory prepared FBM specimen needs to be cured before testing for mixture selection.

Assessing the in-situ curing of FBM is essential to decide when the next layer of the pavement is to be applied. However, this is not straightforward as FBM layers are not strong enough to evaluate using non-destructive testing equipment such as falling weight deflectometers (FWDs) and light weight deflectometers (LWDs). Moreover, obtaining cores from the FBM layer was also found difficult especially in the initial stage of the curing [118]. Hence, Lee [17] highlighted the importance of specifying curing condition in terms of amount of the water present in the mixture. Since then different agencies specified maximum water content in the mixtures as a requirement of a cold recycled (FBM) layer (both foamed bitumen and emulsion treated). The departments of transportation of Arizona, Iowa, South Dakota, Vermont, and Washington allow a maximum water content of 1.5% before a layer is laid over an FBM layer [119]. In Europe the usual range for residual water content to determine the ideal time for placing a bituminous layer over FBM layer is 1-1.5% [106]. To evaluate this in the field, a link is necessary between laboratory curing and in-situ curing. For this it is important to understand the curing mechanism and factors that affect curing such as temperature, time and material content (RAP, cement etc.) and the level of the extent to which these factors affect the curing mechanism.

The present chapter tries to address some of the above discussed concerns by evaluating different curing regimes, identifying important parameters that affect curing and establishing that a link is necessary between laboratory curing and in-situ curing. In view of assessing in-situ strength (or stiffness) the applicability of the maturity method, which is commonly used to estimate in-situ compressive strength of concrete before removal of formwork, to FBMs was evaluated.

6.2 Methodology adopted

To achieve the above mentioned objectives the curing study was conducted in three stages. The first stage (Stage 1) investigated the need for accelerated curing and the degree of acceleration that is realistic by studying different curing regimes and their effect on mechanical properties and performance behaviour of FBMs. A suitable curing protocol was established which specifies how long a laboratory prepared FBM specimen needs to be cured before testing for mixture selection. In the second stage (Stage 2) an experimental study was carried out to interpret the level of impact that factors such as temperature, time, presence of RAP and active filler have on the curing of FBMs. The effect of these factors on curing has been evaluated with reference to stiffness gain and water loss of specimens made of these mixtures. The third stage (Stage 3) deals with applicability of the maturity method as a tool to assess the in-situ characteristics of FBMs. Along with validating the maturity method, practical implications of the maturity method were also evaluated in Stage 3.

In the present study the terms ‘early stage curing’ (less than 7 days after compaction); ‘intermediate stage curing’ (between 8 days and 35 days) and ‘long term curing’ (between 35 days and 300 days) were used for convenience. In Stage 2 FBMs were cured up to 30 – 35 days at different curing temperatures and the curing trends were developed by monitoring water loss and stiffness gain in FBMs over this period of time. In Stage 3 the specimens were cured for long term i.e. up to 180 to 300 days and maturity – stiffness models were developed. Table 6-1 presents the mixtures considered at the three stages of the present study. The mixing and compaction was carried out as discussed in Chapter 5. It was assumed that the addition of cement doesn’t influence the optimum mixing water content (MWC), foamed bitumen content (FBC) and design number of gyrations (N_{design}) (see Section 2.4.1.1.). Therefore, the same parameters that were obtained in Chapter 5 for the mixtures without cement were used for the mixtures with cement (1% cement content). The cement used in the study is Ordinary Portland Cement (OPC).

Table 6-1 Parameters used in different FBMs

	MWC (%)	FBC (%)	N	Stages
50%RAP+1%Cement	4.8	3.25	110	1,2,3
50%RAP	4.8	3.25	110	1,2,3
75%RAP + 1%Cement	4.8	3	100	2
100%VA+1%Cement	5.2	4	140	2,3
100%VA	5.2	4	140	2,3

6.3 Effect of laboratory curing regime on mechanical properties

The work presented in this section (Stage 1) highlights the need for accelerated curing and the effect on degree of acceleration by examining different curing regimes and how they affect the mechanical and performance properties of the FBM. The experimental design is carefully selected in such a way that the curing conditions include regimes that are followed by different agencies and also conditions that are expected to simulate different climatic zones.

This study provided a valid investigation into the behavior of FBM taking into account (1) the effect of temperature, curing conditioning and curing duration (2) the influence of cement with different curing regimes. Mechanical properties that were considered in the experimental study are Indirect Tensile Stiffness Modulus (ITSM), Indirect Tensile Strength (ITS) and performance in the Repeated Load Axial Test (RLAT).

6.3.1 Selection of curing regimes

A summary of curing regimes used and recommended by different researchers is presented in Table 6-2. Table 6-2 demonstrates that the curing temperatures that have been recommended were 60°C, 40°C and 20°C. The curing temperature of 60°C was widely used until Ruckel et al., (1982) [18] expressed their concern over a curing temperature of 60°C which is above the softening point of many bitumen grades that are commonly used for foaming. This high temperature may cause a change in bitumen properties during curing, which is not desirable. Furthermore, conditioning above the softening point temperature might cause damage to the specimen by its own weight and change the internal microstructure of the specimen [111]. Consequently, the curing temperatures of 40°C and 20°C were selected for this study. In addition to these temperatures, 5°C was also considered in the study to resemble colder climatic conditions.

In terms of number of days that a specimen has to be cured at a specified temperature before testing, 3 days seems the most widely used criterion (Table 6-2). As the actual study includes emulsion treated mixtures, 28 days curing condition was also included along with 3 days conditioning period. Conditioning for 28 days is generally used in emulsion mix design [120, 121]. Though most of the researchers proposed unsealed specimen curing, some recent work [106] proposed sealing of the specimens to provide constant humidity to the specimens during the curing process. Therefore the present study considered sealed (Fully Wrapped) and unsealed (Unwrapped) conditions. In the view that only the surface of a cold recycled pavement layer is directly exposed to temperature and humidity, Batista and Antunes (2003) covered all but the tops of the specimen (emulsion treated) with a plastic film. They hypothesized that water content evolution in the field would be between laboratory specimens that are Fully Wrapped (FW) and Unwrapped (UW). The study also found good agreement between cores obtained from the field and laboratory cured specimens in term of mechanical properties. Therefore, the present study included a curing condition called COMBO which requires partially wrapping for 7 days and then wrapping for 14 days. The idea was that after 7 days of laying the FBM layer would be covered by a binder course and (or) surface course. The composition of aggregates used in the research study comprised 50% RAP and 50% virgin aggregates. The physical properties of virgin aggregate and RAP along with design gradation were as presented in Chapter 4. It was reported that in FBM the inclusion of cement influences the water content evolution [106]. Therefore, the study was also conducted on mixtures with 1% cement. Table 6-3 states the curing regimes and the mixtures that were considered for the present experimental study.

Table 6-2 Summary of curing regimes recommended for FBM

Author	Temperature (°C)	Duration (days)	Condition	In-situ comparable age(days)
Bowering, 1970. [67]	60	3	Unsealed	
Bowering and Martin, 1976 [16]	60	3	Unsealed	60
Acott, 1979 [39]	60	3	Unsealed	200
Ruckel et al., 1982 [18]	40	1,3	Unsealed	1 and 30
Ruckel et al., 1982	20	1	Unsealed	1
Roberts et al., 1984 [40]	20	3	Unsealed	
Lancaster et al., 1994 [53]	60	3	Unsealed	
Maccarone et al., 1995 [32]	60	3	Unsealed	365
Muthen, 1998 [11]	40	3	Unsealed	
Jenkins, 2000 [7]	40	3	Unsealed	
Marquis, 2003 [65]	40	3	Unsealed	
Kekwick, 2005 [122]	60	3	Unsealed	
Kim et al., 2007 [66]	40	3	Unsealed	180
Jenkins and Moloto.,2008[117]	40	3	Sealed	
Fu et al.,2010[111]	40	7,14	Sealed	

6.3.2 Specimen preparation and testing

The foaming water content, foamed bitumen content and mixing water content of the mixes were 2.9%, 3.25% and 4.8% respectively as obtained in the mix design (Table 6-1). The mixing and compaction (110 gyrations) was carried out as discussed in the previous chapter. After compaction, the specimens were left in the mould for 24 hours and this was followed by curing according to the regime as stated in Table 6-3. For wrapping the specimen aluminium foil tape was used. Figure 6-1 shows the FBM specimens in their respective curing conditions.

ITSM and ITS tests were conducted on cured specimens in accordance with BS EN 12697-26 and BS EN 12697-23 respectively. To complete this study the resistance to permanent deformation was carried out using the Repeated Load Axial test (RLAT) in accordance with BS DD 226. This test evaluates deformation resistance of bituminous mixtures. The test is a repeated load test that applies a repeated load of fixed magnitude and cycle duration to a cylindrical test specimen. The parameters used are stated below:

- Conditioning Stress: 10kPa
- Conditioning Period: 600s
- Test Stress: 100kPa
- Loading time: 1s
- Rest period: 1s
- Test Duration: 1800 pulses
- Test Temperature: 30^{o}C

Table 6-3 Curing regimes that were considered in the study

Phase	*Curing Temperature (°C)*	*Curing duration (days)*	*Curing Condition*	*Cement Content (OPC)*
1	5	28	Fully Wrapped (FW)	0%
	20			
	40			
2	5	28	Unwrapped (UW)	0%
	20			
	40			
3	5	28	Fully Wrapped (FW)	1%
	20			
	40			
4	5	28	Unwrapped (UW)	1%
	20			
	40			
5	20	21	Combination (COMBO)	0%
6	20	21	Combination (COMBO)	1%
7	40	3	Fully Wrapped (FW)	0%
8	40	3	Fully Wrapped (FW)	1%
9	40	3	Unwrapped (UW)	0%
10	40	3	Unwrapped (UW)	1%

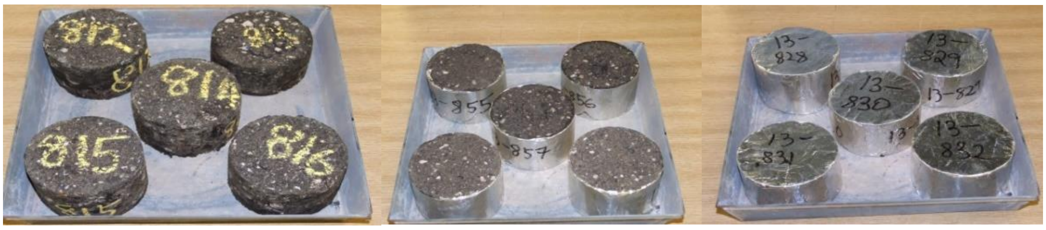


Figure 6-1 (L-R) – Unwrapped, Partially Wrapped and Fully Wrapped

6.3.3 Effect of curing condition on ITSM and ITS

Figure 6-2 present stiffness (ITSM) results at 20°C as obtained for FBM conditioned for 28 days at three different temperatures and two curing conditions (UW and FW) with and without the inclusion of 1% cement (OPC). The results as seen in Figure 6-2 show that the fully wrapped (FW) specimens had the lowest stiffness values especially those cured at 5°C with a value of 648MPa for FBM with 0% OPC. The highest stiffness values for the FW specimens with 0% OPC were observed on specimens cured at 40°C with a value of 1554MPa. At all curing temperatures, unwrapped (UW) specimens with 0% OPC reached a stiffness 2-2.5 times the FW specimens.

ITS test results, also at 20°C, are presented in Figure 6-3. The trend observed for the ITS test results is identical to that for stiffness (ITSM). The lowest strength (ITS) values were observed also for the FW specimens cured at 5°C. At 0% OPC, UW specimens achieved 1.5 – 2.5 times the strength of FW specimens.

6.3.4 Influence of temperature on curing

The influence of curing temperature on the stiffness modulus and strength development of the mixtures is illustrated in the case of 0% OPC specimens in Figure 6-2 and Figure 6-3. The stiffness modulus and strength development increases with increase in curing temperature. This was evident irrespective of the curing conditioning employed. Higher curing temperatures resulted in higher rates of stiffness and strength increase which led to higher maximum values.

A major phenomenon occurring during the curing process is water loss. Figure 6-4 shows the water content in the specimen by the end of the each curing regime. The initial water content was 4.8% which is the mixing water content of the mixture. This number neglects water loss by evaporation during mixing and compaction. As shown in Figure 6-4, there is naturally a significant difference between FW and UW specimens. Nevertheless, it is apparent from the relatively modest difference between final water contents at different curing temperatures that water loss is not the only mechanism involved in ‘curing’. High temperature is clearly responsible for additional strength/stiffness gain, possibly by facilitating binder adhesion during the curing process.

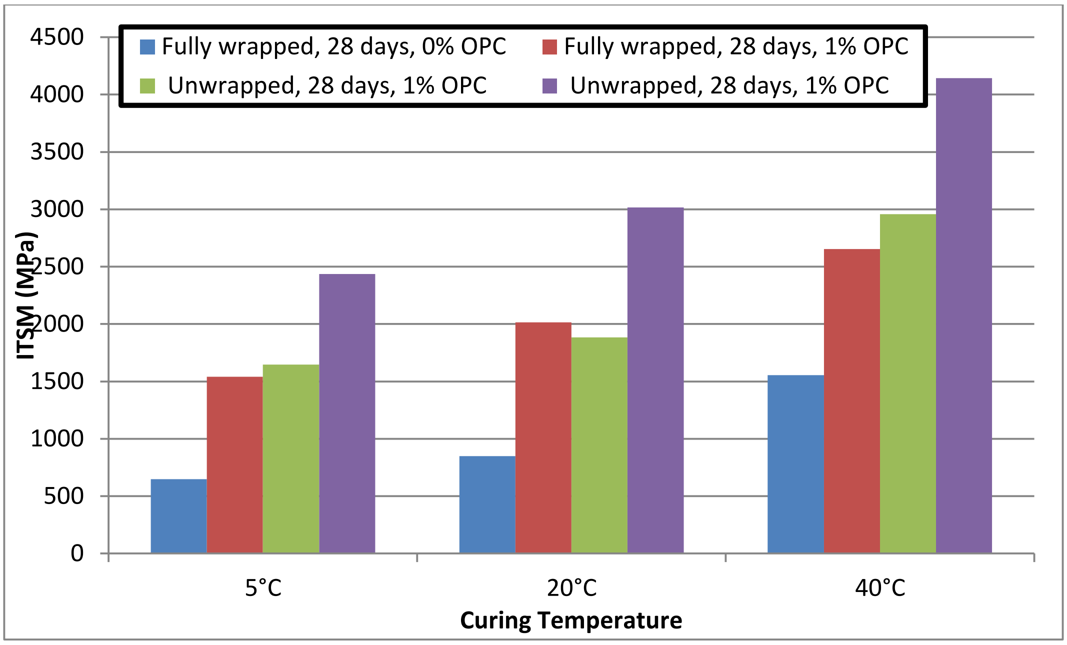


Figure 6-2 Effect of curing condition on 20°C ITSM of 50% RAP-FBM

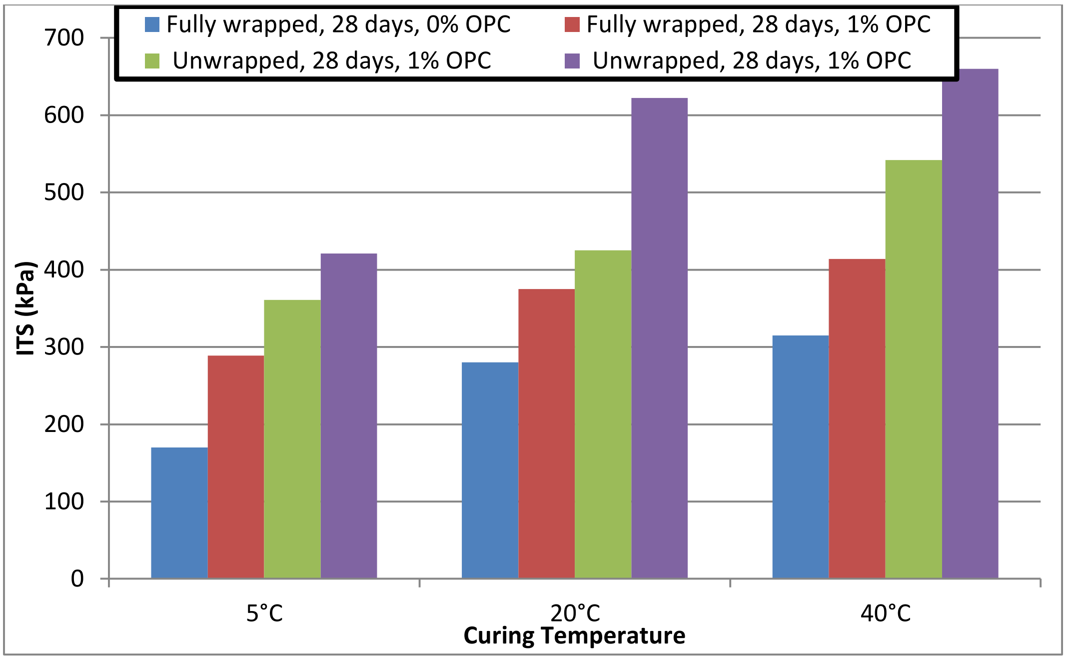


Figure 6-3 Effect of curing condition on 20°C ITS of 50% RAP-FBM

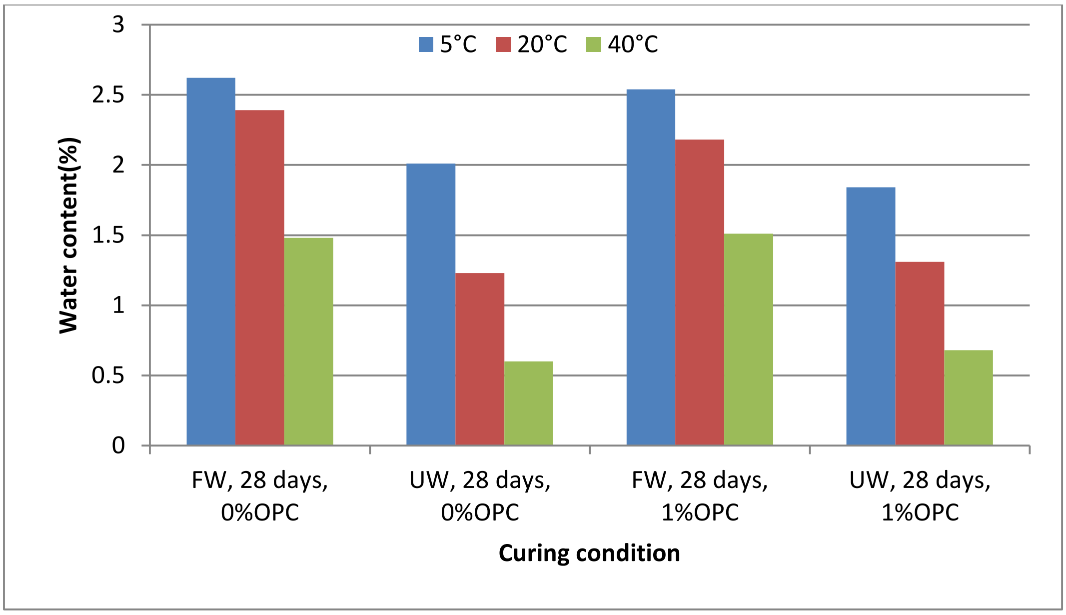


Figure 6-4 Effect of temperature on water evolution in 50% RAP FBM (Note: Mixing water content is 4.8%)

6.3.5 Influence of cement

The addition of cement increased the stiffness and strength at all curing temperatures for both FW and UW conditions (Figure 6-2 and Figure 6-3). This increase is partly due to the removal of water from the system and partly due to the formation of cementitious bonds. In terms of stiffness, the addition of 1% OPC to FW specimens increased the 28 day values to those of 0% OPC UW specimens, illustrating the usefulness of cement in countering adverse weather during curing. The effect in terms of strength was slightly less compared to stiffness but still significant. Addition of cement to UW specimens also led to strength and stiffness increase. However, it is noticeable that with curing at 40°C, the effect of cement is minor.

6.3.6 Influence of curing time

The influence of time on curing can be seen in Figure 6-5 where the 3 day and 28 day data are compared. The results shown in the figure are of Fully Wrapped specimens and cured at 40°C. Typically, it may be found that the stiffness values for the FW specimens, cured at 40°C for 3 days were about 50-60% of those achieved at 28 days and with respect to the ITS results, the proportion is about 70%.

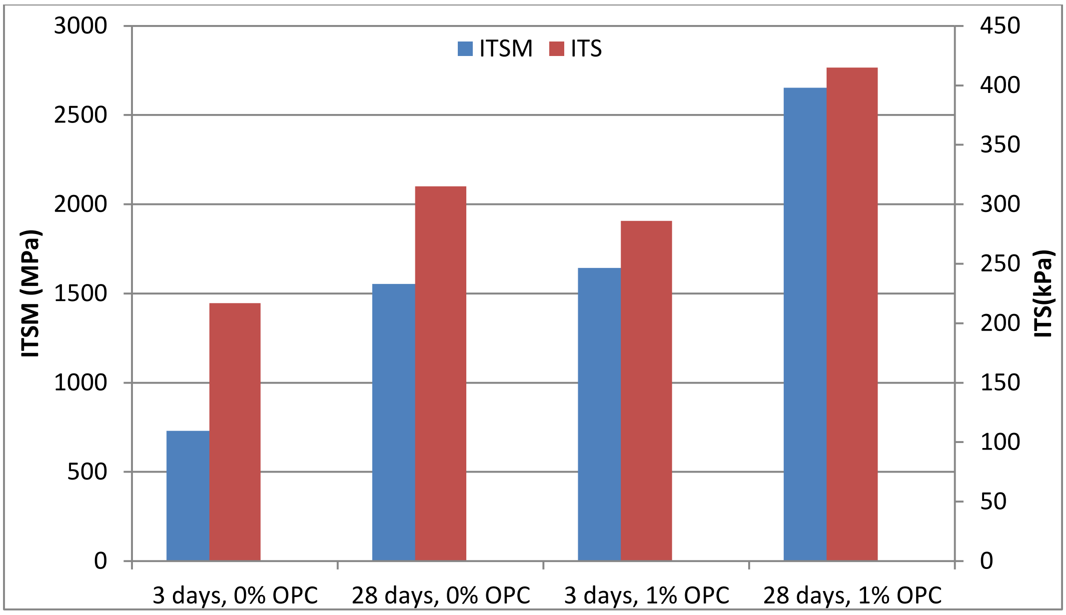


Figure 6-5 Influence of time of curing on 20°C ITSM of 50% RAP-FBM (Note: All specimens were Fully Wrapped and cured at 40°C)

6.3.7 Resistance to Repeated Loading

The results of RLAT tests at 30°C are presented in Figure 6-6 and Figure 6-7. Figure 6-7 presents permanent strain (%) of the specimen in RLAT test after 1800 load pulses against curing temperature. Similar to stiffness and strength results the curing condition has significant effect on the performance of FBMs in terms of permanent deformation resistance. The specimens that were fully wrapped (FW) were found to have higher axial strain after 1800 load pulses. As the FW specimens contain more water in them after 28 days than UW specimens, it can be interpreted that the presence of water influences the permanent deformation resistance of FBMs negatively. As can be seen from Figure 6-7, though higher curing temperature resulted in less permanent (plastic) strains, the effect is moderate. This is particularly true from curing temperature 20°C to 40°C. On the other hand, the addition of cement significantly affected the permanent deformation behaviour of FBMs. The results suggest that cement addition will improve the mechanical properties and thus performance even when the FBM layer is covered with other layers.

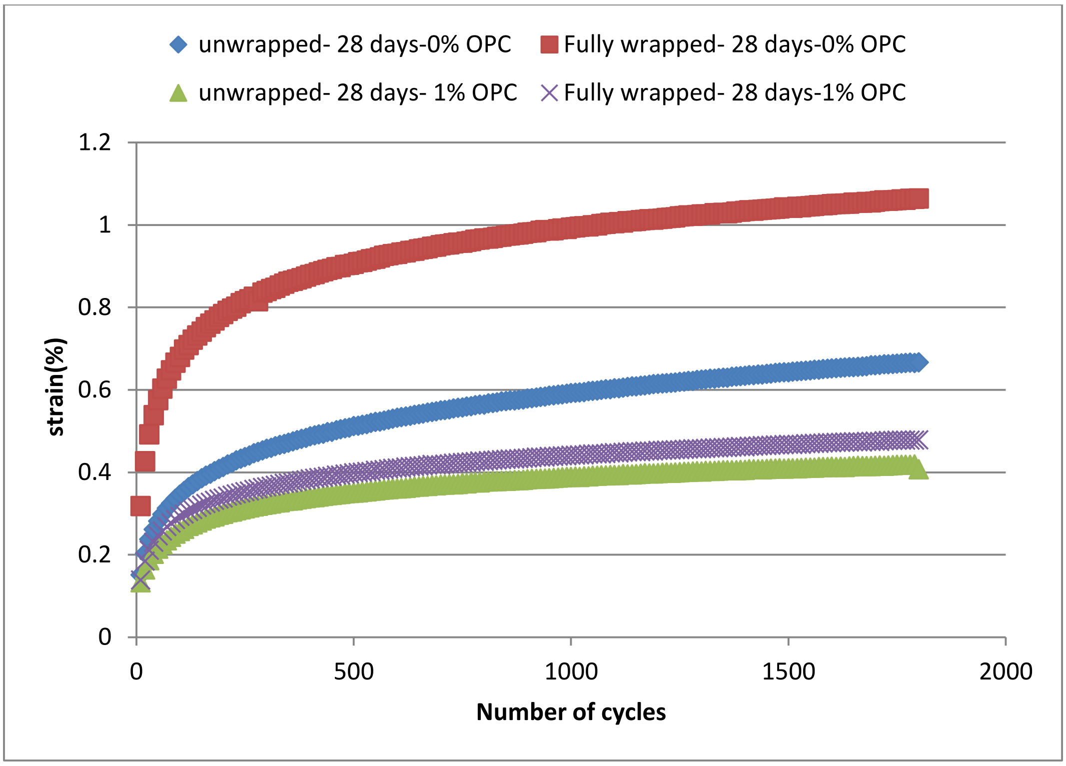


Figure 6-6 RLAT test results on 50%RAP-FBM specimens cured at 20°C

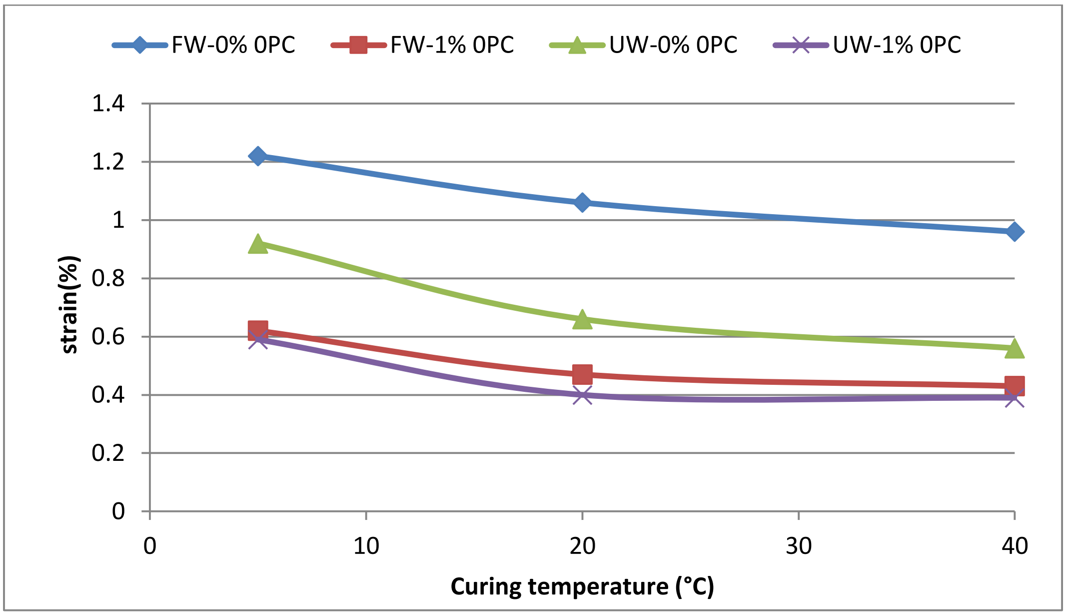


Figure 6-7 RLAT test results on the specimens cured for 28 days

6.3.8 Comparison of mechanical properties for different curing regimes

As discussed, the combination of 7 days partially wrapped and 14 days fully wrapped (COMBO) was introduced to simulate typical field conditions with a surface course and (or) binder course applied 7 days after construction of the base layer and trafficking commencing 14 days later. Table 6-4 and Figure 6-8 to 6-10 compare this curing condition to some of the well followed curing regimes for Cold Recycled Bituminous Mixes (CRBMs) across the world. It has to be noted that the lower the strains in Figure 6-10, the better the resistance to permanent deformation. It is observed from the figures that in every case, the mechanical properties for COMBO lie in between the FW and UW curing conditions of specimens cured at 20°C for 28 days. Furthermore, the UW-40°C-3 days curing regime overestimates the FBM properties and performance if the material selection criterion is based on the properties of the pavement when allowed for trafficking. The FW condition for 28 days at 20°C seems to be conservative in terms of stiffness and strength and reasonably matched for permanent deformation with COMBO. The other two curing conditions (FW-40°C-28 days and UW-20°C-28 days), though slightly overestimating the properties compared to COMBO, seem to represent the actual conditions reasonably well if COMBO is assumed to represent actual conditions in the field.

Table 6-4 Summary of mechanical properties obtained for different curing recommended

Curing regime	**Reference**	**ITSM (MPa)**		**ITS (kPa)**		**Permanent strain (%)**	
Cement content		**0% OPC**	**1% OPC**	**0% OPC**	**1% OPC**	**0% OPC**	**1% OPC**
FW-40°C-28 days	Highways Agency [7]	1554	2957	315	414	0.96	0.43
UW-40°C-3 days	Ruckel et al., [20]	2490	3378	509	594	0.6	0.48
FW-20°C-28 days	Highways Agency *	850	1883	280	315	0.66	0.56
UW-20°C-28 days	Thanaya et al., [6]	1883	3015	425	622	0.4	0.39
COMBO	Batista and Antunes [2]	1420	2519	332	431	0.91	0.65

* Curing regime recommended for emulsion treated mixtures

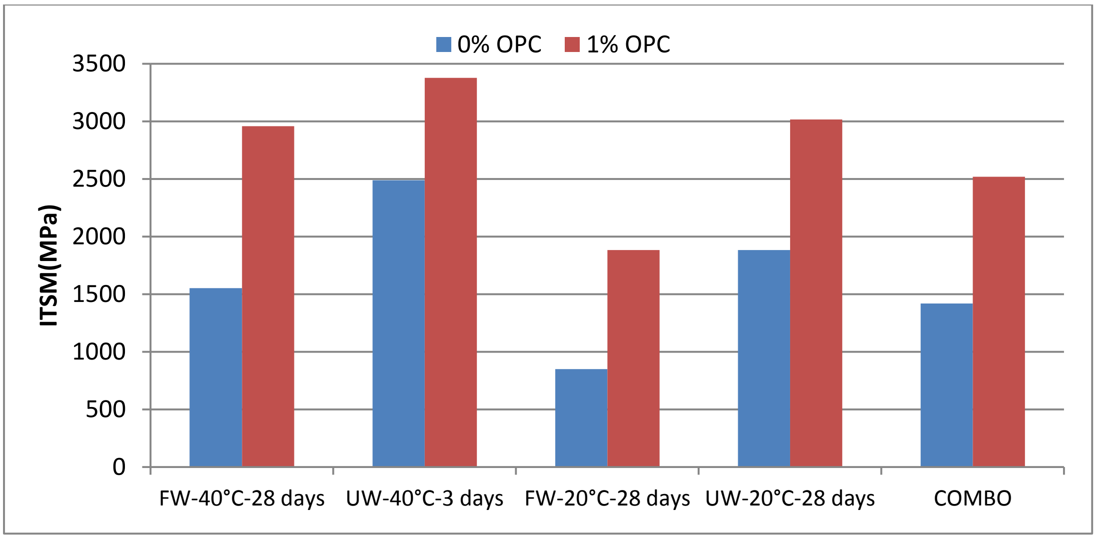


Figure 6-8 Comparison of 20°C ITSM obtained for different curing regimes (50% RAP-FBM)

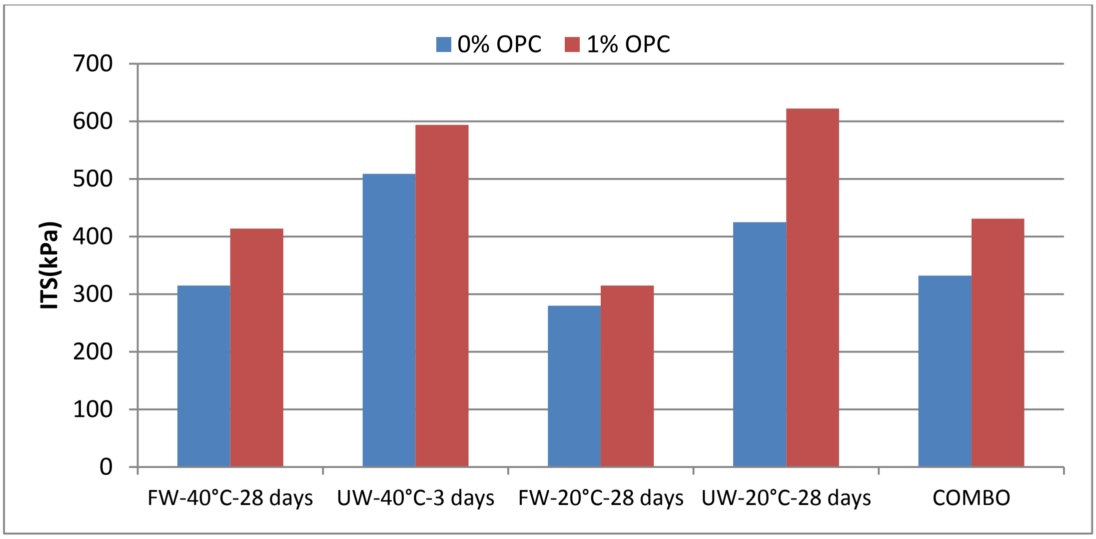


Figure 6-9 Comparison of ITS obtained for different curing regimes (50% RAP-FBM)

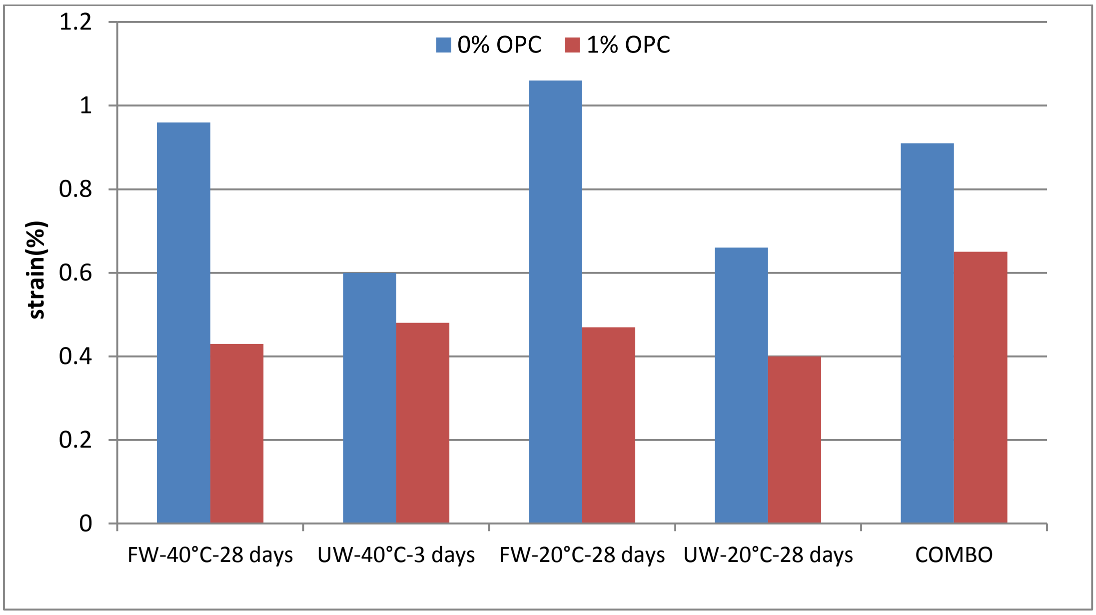


Figure 6-10 Comparison of RLAT test results obtained for different curing regimes (50% RAP-FBM)

6.4 Curing mechanism (parametric) study

While the previous section has evaluated various curing regimes and identified some crucial factors that govern curing of FBMs, this section discusses a comprehensive laboratory investigation to understand the curing mechanism (Stage 2).

As discussed previously since most of the agencies require a limit on the amount of water present in the FBM layer before laying a binder course and (or) surface, a link is necessary between laboratory curing and field water content requirements. Therefore, the present study aimed at developing curing trends in terms of water loss and stiffness gain for different temperatures and over a period of time to understand the curing mechanism which also helps in establishing a link between laboratory curing and field curing. These trends also help in understanding the acceleration of curing at elevated temperatures and to predict the properties of the FBM layers in the pavement for different climatic conditions. The water evolution trends obtained by monitoring the curing process give knowledge on the time it takes for various materials to reach equilibrium water content [117]. The results are also expected to be valuable inputs to formulate the necessary relation between laboratory properties and field trends, which gives valuable guidance for structural design of the pavements.

6.4.1 Selection of conditioning temperatures

As can be seen from Table 6-2 elevated temperatures ranging from ambient 20°C to as high as 60°C were proposed. Although other conditioning temperatures can also be found in the literature, 60°C, 40°C and 20°C have been popular. In order to avoid ageing of bitumen and to keep the specimens below the softening point of the bitumen used in the mixture, which is 45°C for the 70/100 grade bitumen used in this study, curing temperatures of 40°C, 30°C, 20°C and 5°C were considered. Though curing at 20°C and 5°C are not to be considered as accelerated curing, these temperatures were included in the study to understand the effect of these temperatures on curing.

6.4.2 Monitoring curing

The curing trends were developed by monitoring water loss and stiffness gain in FBMs over a period of time. The specimens were gyratory compacted cylindrical specimens of 100mm diameter and about 62mm high. The materials and specimen preparation procedure were as discussed in the previous chapter (Chapter 5). The non-destructive stiffness (ITSM) test was selected for assessing curing of the FBM specimen over a period of 30 days. Indirect Tensile Stiffness Modulus (ITSM) was considered in order to carry out the test on the same set of specimens to nullify variability in the mixtures and to derive reliable trends for curing evaluation. To study the effect of RAP and cement, the following mixtures were considered (a) 100% virgin aggregate (100% VA) (b) 100%VA+1%Cement (c) 50% reclaimed asphalt pavement (50% RAP) (d) 50%RAP+1% Cement (e) 75% RAP+1% Cement.

As was found in the curing regime study (Stage 1) for different conditioning times considered in the study the mechanical properties of sealed (FW) specimens were always found to be 50-70% those of unsealed (UW) specimens. So it was assumed that the sealed condition can interpreted using unsealed data and therefore only the unsealed curing condition was considered in the curing mechanism study (Stage 2).

6.4.3 Effect of time and temperature on curing trends

The water content in the 100% VA-FBM specimens which were monitored over time is plotted in Figure 6-11. A trend line which is a power curve is also included in the figure. The initial water content in the mixture during compaction was 5.2% (80% of OWC). The plots showed that water content in the specimen reached about a quarter (around 1.2%) of the initial amount after 24 hours of curing at both 30°C and 40°C. Similarly, water content reached about half (2.6%) of the initial amount when specimens were cured at 20°C and 5°C. The trend suggests that the rate of water loss is approximately proportional to the amount of water present in the mixture. In other words, the rate of water loss decreased with time. It is clear from the plots that the water loss is dependent on the curing temperature. The higher the curing temperature the faster was the water loss. However, all curves, except the 5°C curve, seem to reach a constant amount after which the loss is negligible.

The stiffness (ITSM) values that were measured over time on the 100% VA-FBM specimens cured at different temperatures are plotted in Figure 6-12. The plot shows stiffness monitored for a period of 30 days at curing temperatures of 40°C, 30°C, 20°C and 5°C. Trend lines which are logarithmic with positive tangential slope were also included in the plot. As can be seen from the figure, for all conditions stiffness increased with time of curing. However, this gain was rapid for specimens cured at higher temperatures. This could be attributed to rapid water loss that takes place at higher temperature which yields higher stiffness values with time. Moreover, in a similar way to water loss, the rate of stiffness gain is decreasing with time which implies that the stiffness gain is very much related to water loss from the specimen. This curing phenomenon can be seen in Figure 6-13 in which water loss and stiffness gain of 100% VA-FBM specimens cured at 20°C is presented. The figure illustrates the curing mechanism in which these FBMs gain stiffness with water loss during curing. The ITSM test results of this curing (water loss and stiffness gain) study for all the other mixtures that were considered in the study are presented in

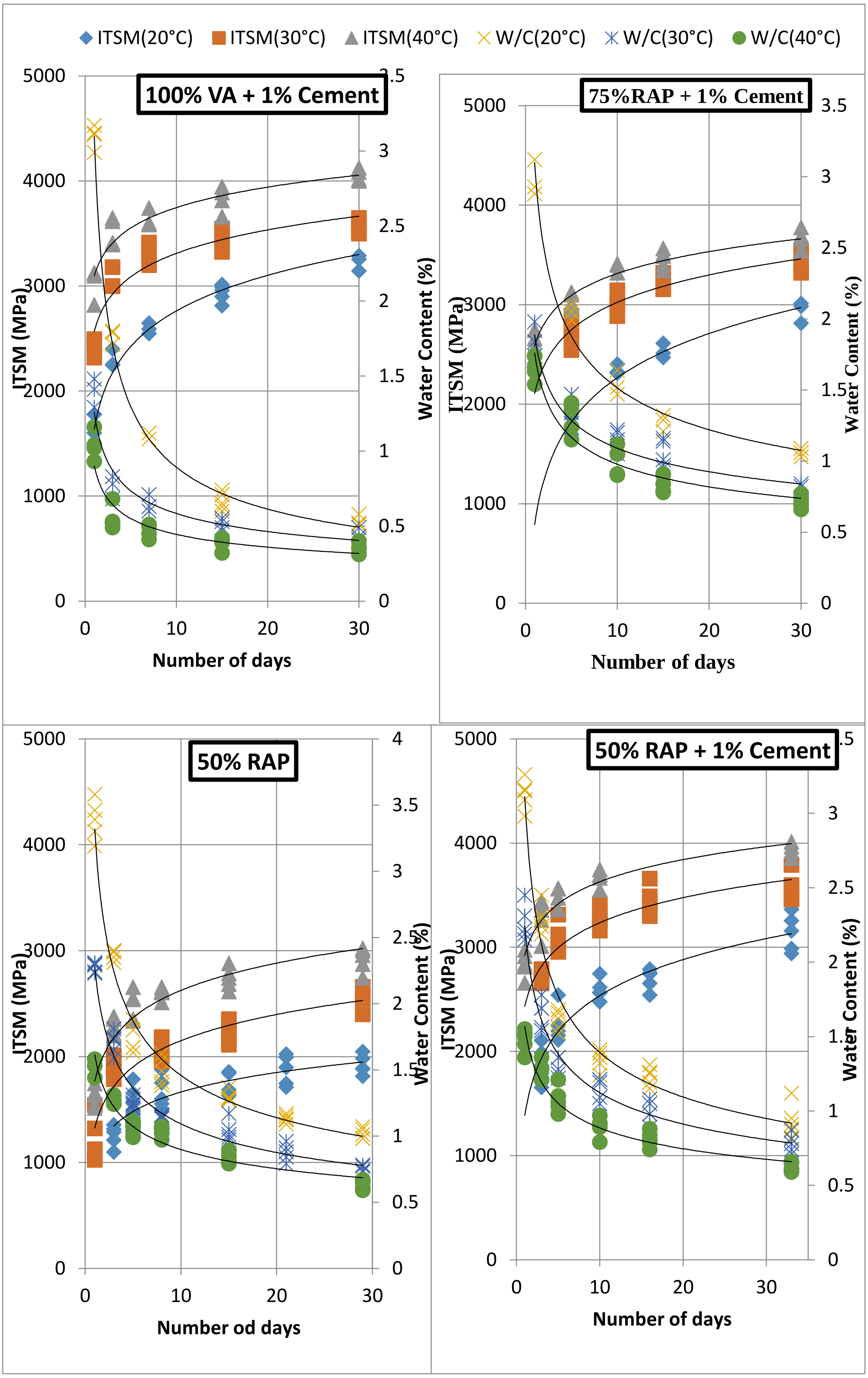


Figure 6-14. A Similar explanation is also valid for the other considered mixtures as well.

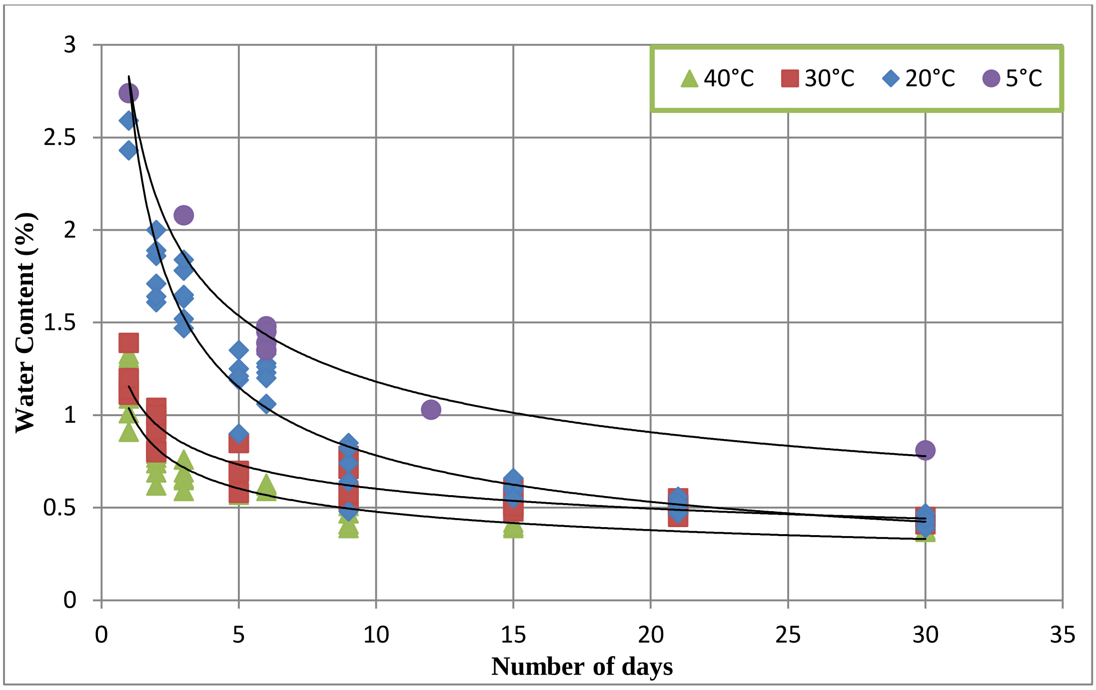


Figure 6-11 Effect of time and temperature on water loss in 100%VA-FBM (Unwrapped)

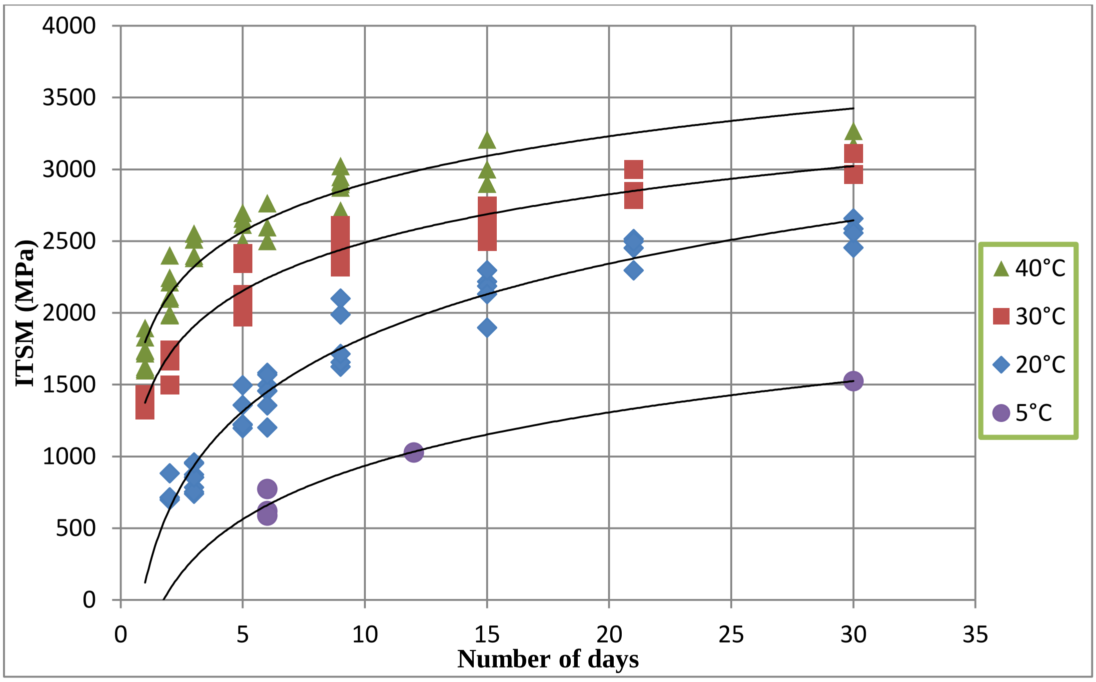


Figure 6-12 Effect of time and temperature on stiffness gain in 100%VA-FBM (Unwrapped)

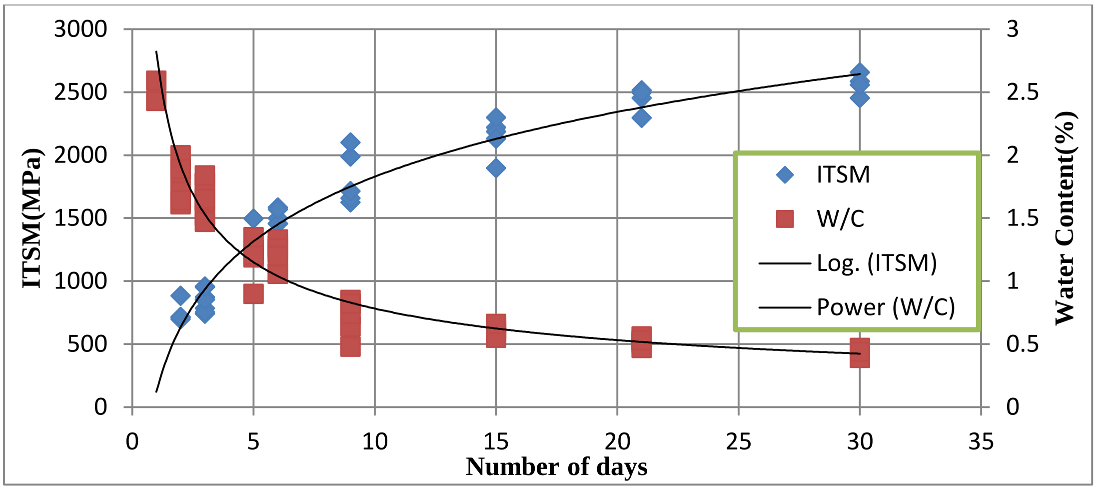


Figure 6-13 Effect of time on curing (water loss and stiffness gain) in 100% VA-FBM (Unwrapped)

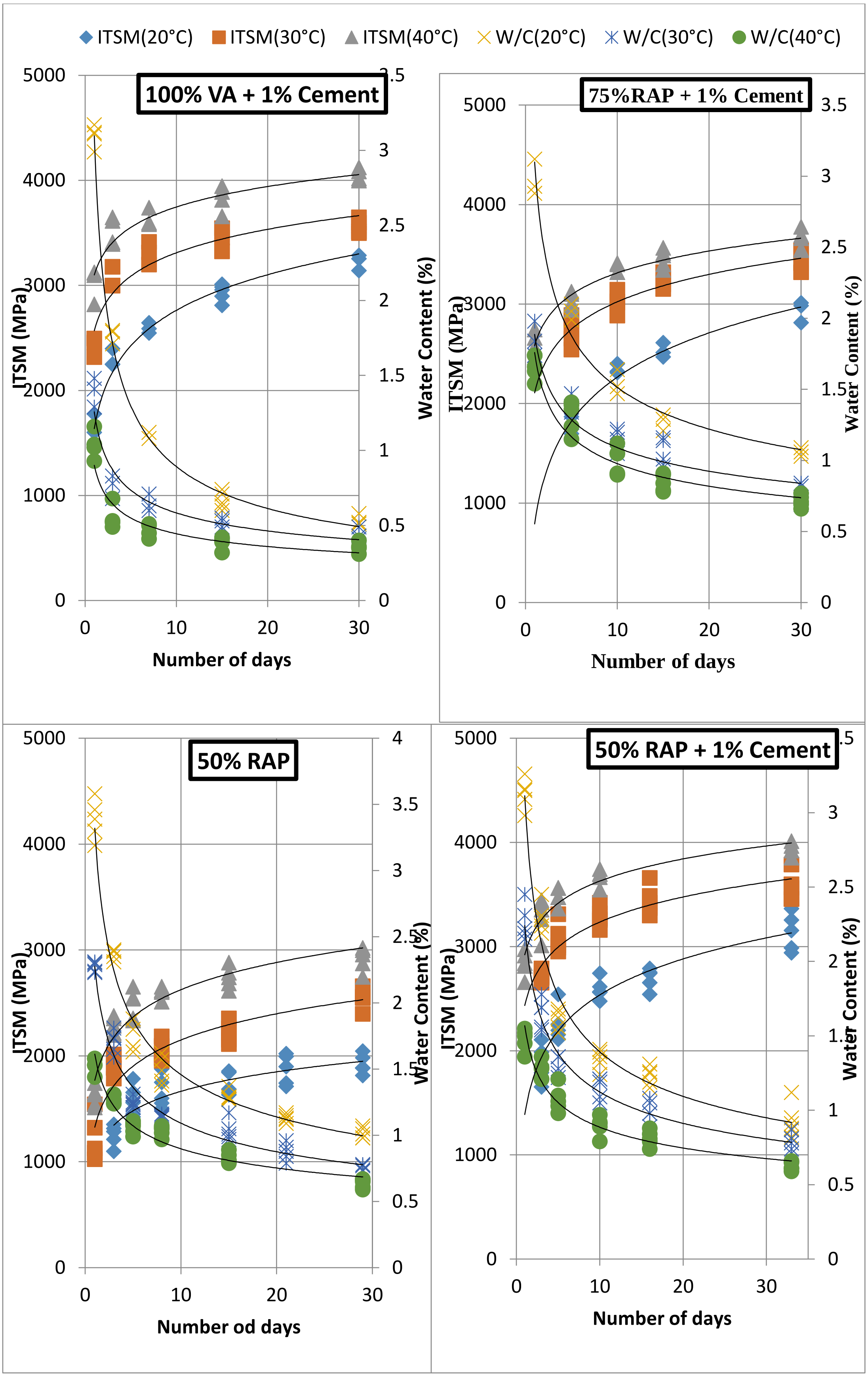


Figure 6-14 Effect of time and temperature on evolution of stiffness of FBMs (Unwrapped)

6.4.4 Effect of water content on stiffness

Figure 6-15 shows the plots of stiffness (ITSM) versus water content for the 100% VA-FBM specimens that were cured at different temperatures. As a general trend, with decrease in the water content the stiffness was increased. However, as can be seen from the figure when individual temperatures were considered it was found that the temperature has a significant additional impact on the stiffness of the mixture. For example, the specimens which were cured at 40°C having water content around 1% showed far better stiffness (ITSM) than the specimens cured at other lower temperatures at the same water content in the specimen.

To study the effect of temperature on bitumen properties, bitumen was extracted in accordance with BS 598-102 from the 100% VA-FBM samples that were cured for 30 days. Frequency sweep tests were carried out on the extracted bitumen and complex modulus (G*) master curves were plotted (Figure 6-16) to understand the effect of the curing temperature on the bitumen properties. Figure 6-16 presents the complex modulus (G*) master curves at a reference temperature of 20°C for the extracted bitumen from 100% VA-FBM specimens cured at 20°C and 40°C for 30 days. It has to be noted that the change in bitumen complex modulus before foaming and after extraction is not only because of the curing but also because of oxidation that occur during bitumen foaming. As can be seen from the plots, the difference in complex modulus for the extracted bitumen is significant, particular at the ITSM testing frequency which is approximately 5Hz which indicates that the bitumen ageing effect between 20°C and 40°C curing is considerable. Hence, from the results it is evident that water loss is not the only mechanism involved in the curing. High temperature is clearly responsible for additional stiffness gain, possibly by facilitating binder adhesion and more importantly from the increased binder stiffness during the curing process. A similar affect was also found in the other mixtures that were considered in the study which can be seen in

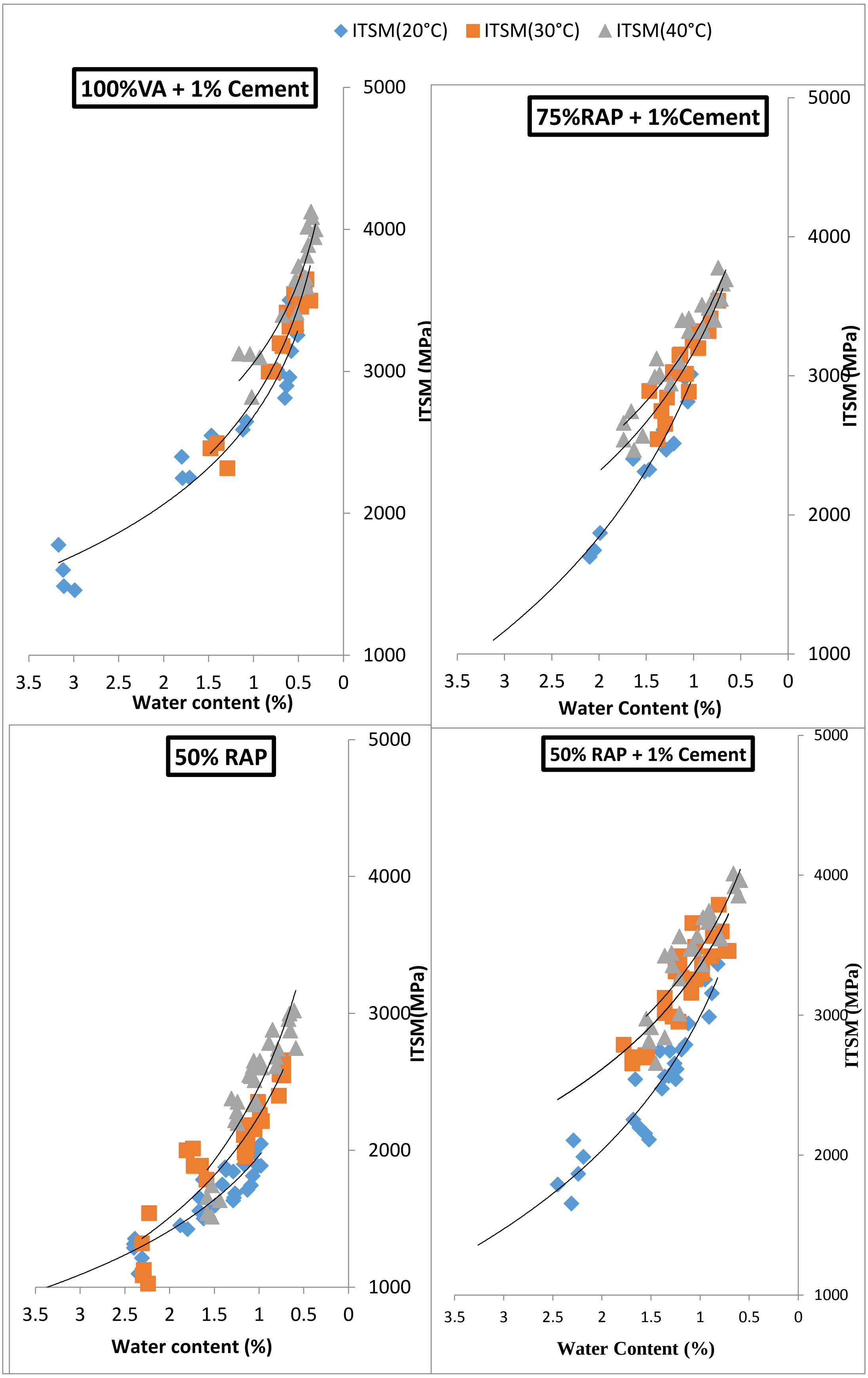


Figure 6-17.

The practical implications of these observations is that specifying a water content limit for FBM layers before allowing traffic or allowing layers (binder courses) to be placed over the FBM layer is not appropriate. For example if an agency specified a 1% water content limit, 100% VA mixture takes approximately 2 days, 7 days and 15 days respectively if the average temperature is 40°C, 20°C and 5°C (Figure 6-11). However, it reaches different levels of stiffness. In this example it reaches 1850 MPa at 40°C, 1500 MPa at 20°C and 1150 MPa at 5°C (Figure 6-15). The stiffness achieved by the mixture at 40°C is about 60% higher than that achieved at 5°C for the same water content.

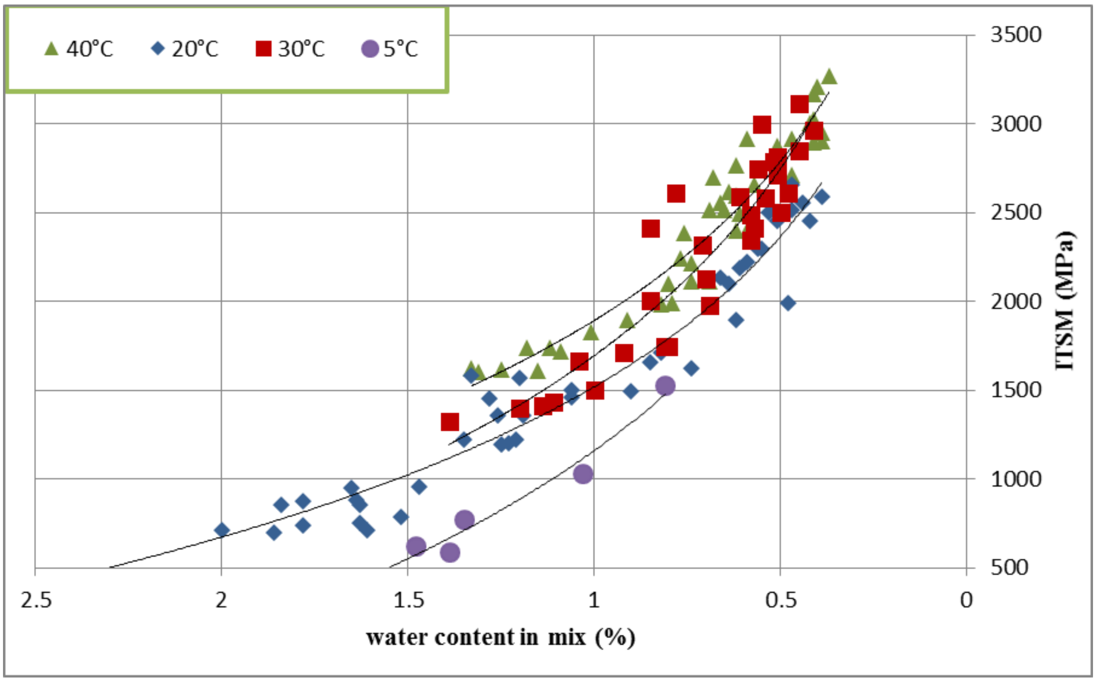


Figure 6-15 Effect of water content and temperature on stiffness in 100%VA-FBM (Unwrapped)

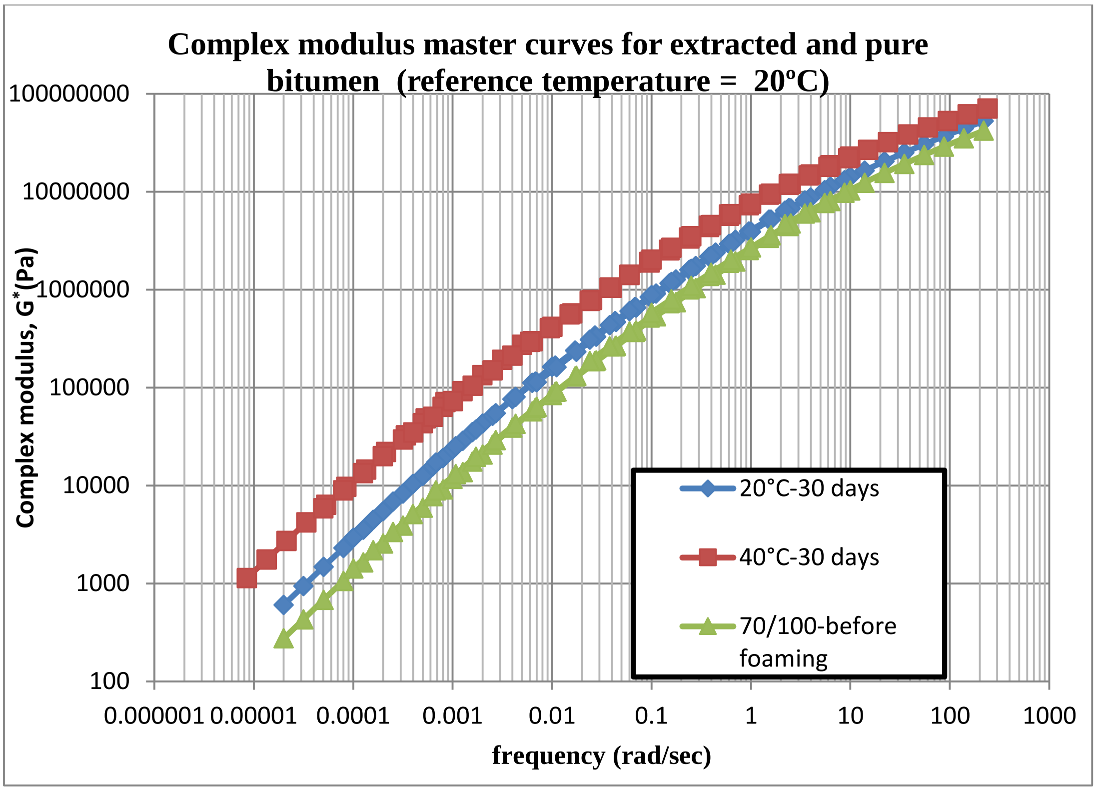


Figure 6-16 Effect of curing temperature on bitumen properties

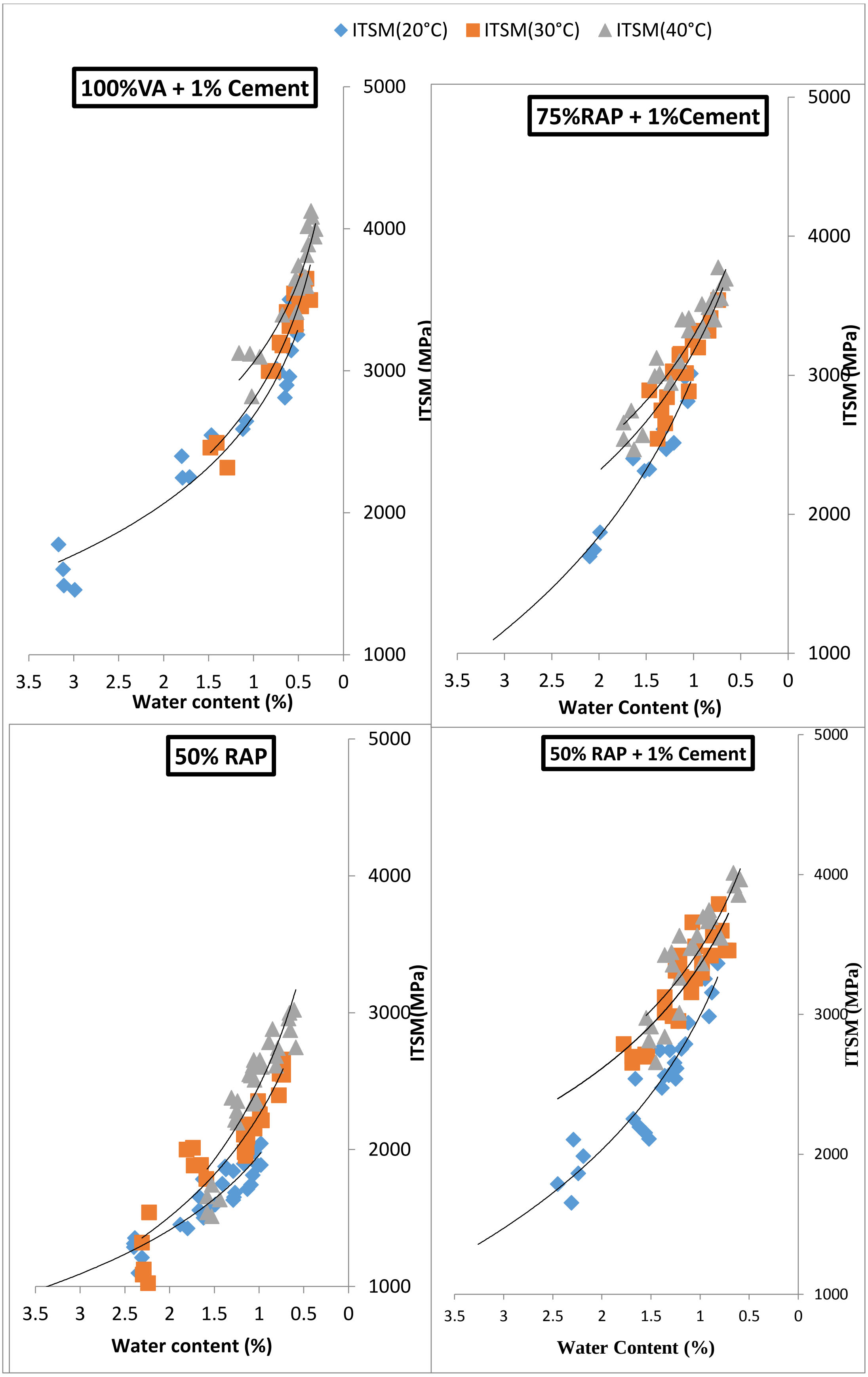


Figure 6-17 Effect of water content and temperature on stiffness in FBM (Unwrapped)

6.4.5 Effect of RAP and cement on curing

Figure 6-18 presents the water loss trends for all the mixtures that were considered and were cured at 20°C. As can be seen from the figure, the trends for 100% VA and 100% VA + 1% Cement are very close which shows that cement addition has no significant influence on the apparent water loss during curing (although a little will be consumed, at an early age, by cement hydration). This phenomenon was found for all temperatures that were considered. However, the presence of RAP has influenced the water loss phase of curing. The mixtures with RAP (50% RAP, 50% RAP + 1% Cement, 75% RAP + 1% Cement) lose water more slowly than mixtures without RAP. The trends of stiffness gain of the FBMs that were cured at 20°C are presented in Figure 6-19. It is clear from the stiffness trends of 100% VA-FBM and 50% RAP-FBM that RAP has a positive influence on stiffness during the early stage of curing. However, the mixtures then gain stiffness at significantly slower rates than mixtures without RAP which results in lower stiffness values at the later stages of curing. This is particularly true when cement is not added to the mixtures. The cement addition helped in stiffness gain in both the mixtures with and without RAP. Hence, from the test results it was understood that though RAP helped in attaining initial stiffness (possibly due to better adhesion between RAP aggregate and mastic), the later stiffness is less which could be attributed to a slower rate of water loss. The results also suggest that cement has no significant influence on the water loss phase of curing but considerably improves the stiffness at any stage of curing of the FBMs.

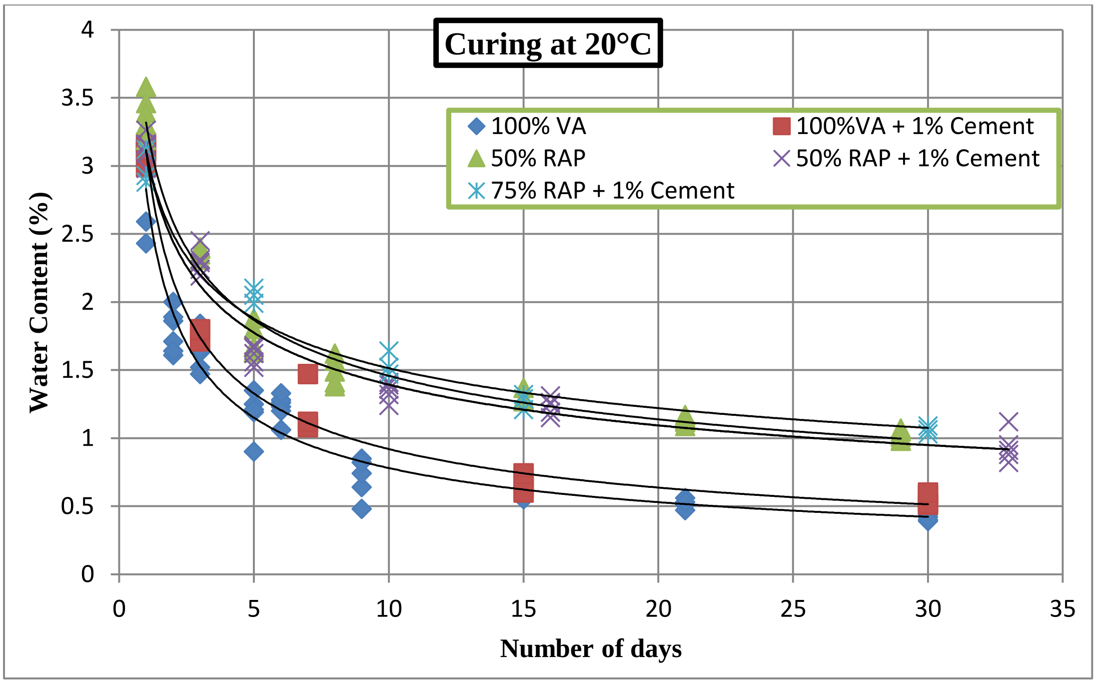


Figure 6-18 Effect of RAP and cement on water loss in FBMs (Unwrapped)

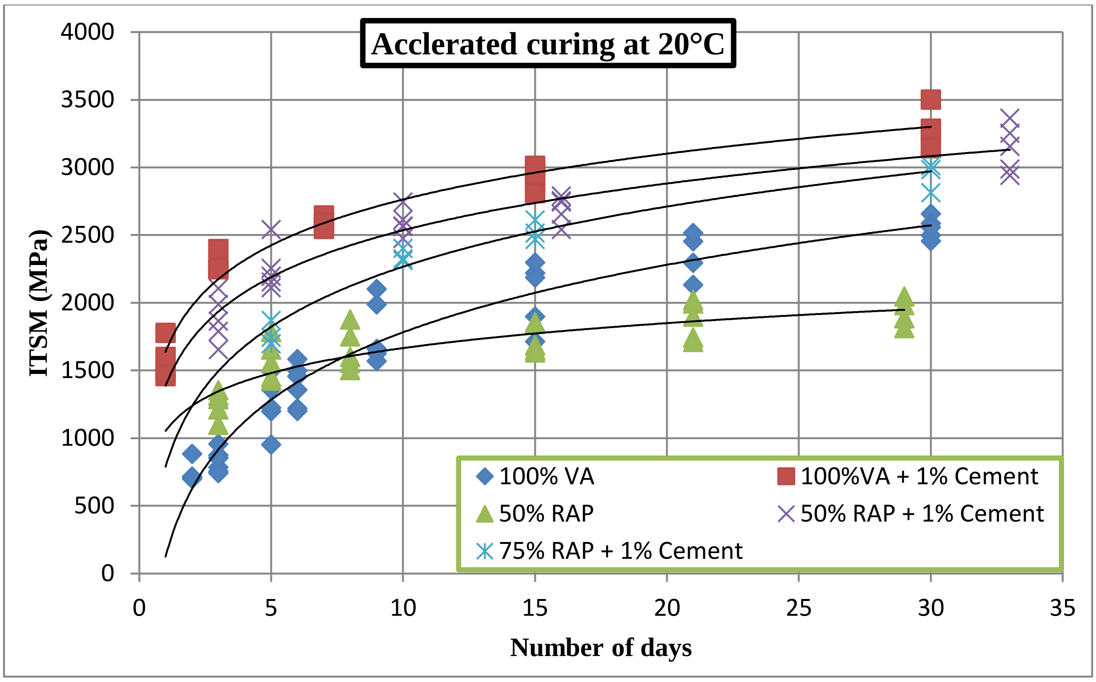


Figure 6-19 Effect of RAP and cement on stiffness gain in FBMs (Unwrapped)

6.4.6 Curing cycle

To understand the sensitivity of stiffness of FBMs after significant curing, water was reintroduced into specimens that had been cured for 30 days. The reintroduction of water was done by soaking the specimens in a water bath at 20°C for 24 hours. After taking the specimens out of the water bath, they were cured at 20°C and water content and stiffness was monitored over a period of 30 days. The reintroduction of water also simulates field conditions during wet weather (e.g. the monsoon season) when water content in pavement layers increases. The test results on specimens before soaking (cycle 1) and after soaking (cycle 2) were as presented in Figure 6-20. The results indicate that reintroduction of water did not damage the bonds formed during curing cycle 1. Moreover, during cycle 2, the stiffness was found to be less sensitive to water content. This is particularly true in FBMs with cement in them.

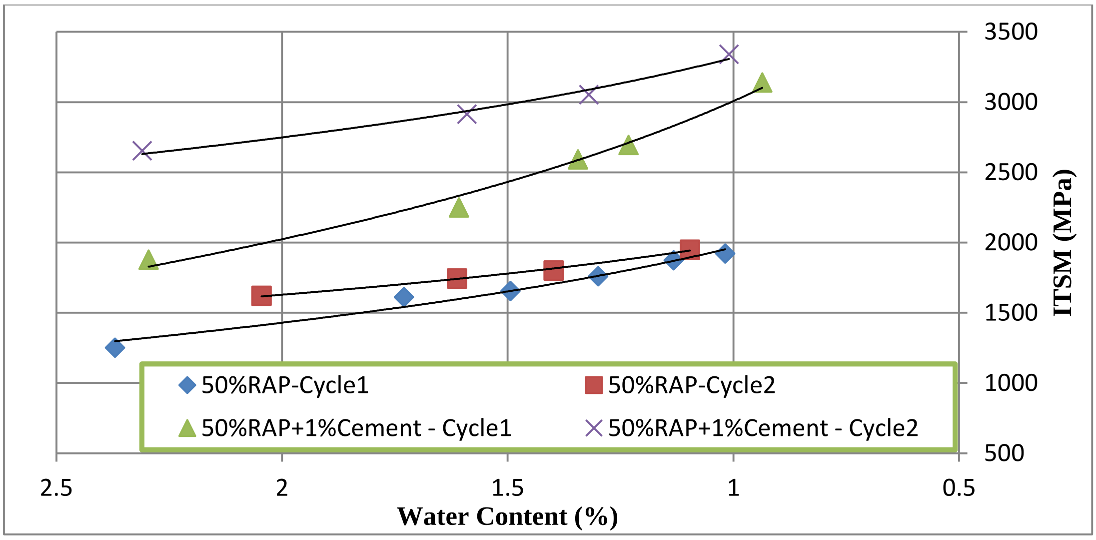


Figure 6-20 Curing cycle test results on FBM specimens cured at 20°C

6.5 Laboratory simulation of in-situ curing

Simulating in-situ condition in the laboratory is the most difficult step in mix design of cold recycled mixtures (FBMs and emulsion treated mixtures) [123]. Hence, a lot of research has been carried out on development of a laboratory curing protocol for these mixes. However, there is a considerable variation in equivalent in-situ condition proposed by different researchers as can be seen in Table 6-2. Nevertheless, as a general trend, with increase in conditioning time and temperature equivalent in-situ conditioning time is increased. In addition to that, the temperature was found to be a more influential parameter than water content on the mechanical properties. This observation suggests the use of the maturity method, which is used for estimating in-place strength of concrete based on its temperature history. The maturity method could help as a tool to estimate in-situ stiffness in view of the fact that the in-situ water content specification could result in different stiffness values for different temperature histories.

For concrete the maturity method allows estimation of in-situ compressive strength as the concrete design criterion in most cases is compressive strength. However as the objective of the present study is to propose a tool to estimate in-situ stiffness, which is an important input parameter for pavement structural design, stiffness (ITSM) was used to generate maturity – stiffness relationships. Moreover in the curing mechanism study (Stage 2) it was shown that ITS follows a similar trend as ITSM.

6.5.1 Maturity methods in concrete

The maturity method is a technique commonly used in the concrete industry to account for the combined effects of time and temperature on the strength gain [124]. This technique allows in-place concrete strength to be estimated using time and temperature history of concrete in the field [125].

6.5.1.1 Time – Temperature factor (Maturity)

Maturity functions are used to convert the actual temperature to an equivalent temperature. This concept of a time - temperature factor (maturity) can also be used to quantify the strength development of FBMs [126]. For instance, if FBM is cured either say at 40 °C or 60 °C, the maturity can be calculated and the strength of the FBM can be estimated. If curing temperature is low (say 40°C) the time to reach the required maturity will be longer than if the curing temperature is high (say 60 °C). As long as the same maturity is reached for both curing conditions, their strength should be the same. The concept of maturity is illustrated in Figure 6-21.

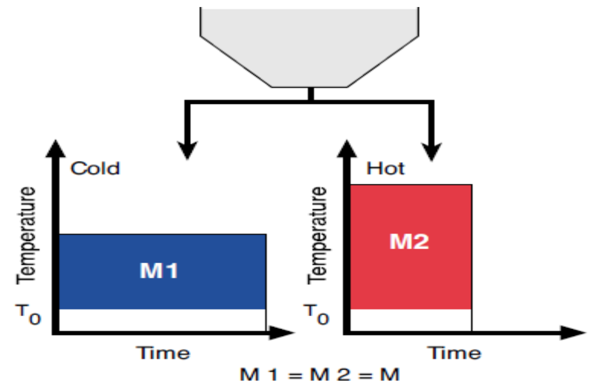


Figure 6-21 Time Temperature factor concept [127]

6.5.1.2 Selection of Maturity function

The time-temperature functions (TTF) or maturity functions are used to convert time and temperature history of in-placed concrete to a factor which is related to its strength. The two TTFs that are being used in in-place strength determination of concrete are the Nurse-Saul function and Arrhenius function.

As Malhotra and Carino (2003) [128] observed, in 1951 Saul defined maturity as the product of time and temperature based on his study on steam curing of concrete. Thus, maturity can be computed from the temperature history using the following equation called the Nurse-Saul equation (Eq. 6-1)). An illustration of the Nurse – Saul function for maturity is presented in Figure 6-22.

M = $\sum_{0}^{t}(T - T_0)\Delta t$ Equation 6-1

Where, M = maturity at age t, T = average temperature of the concrete during the time interval Δt, T_0 = datum temperature

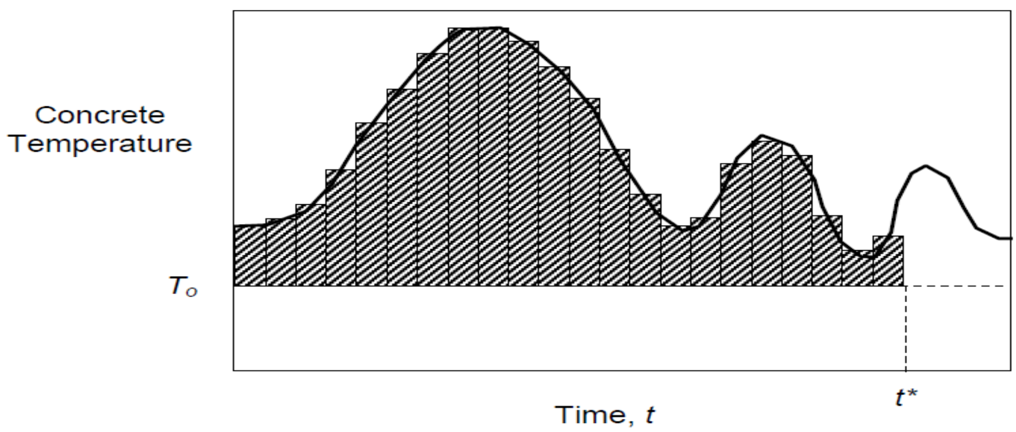


Figure 6-22 Schematic of Nurse – Saul maturity function [128]

The Nurse-Saul function can be used to convert a given temperature time curing history to an equivalent age of curing at a reference temperature as follows:

$$\boldsymbol{t_e = \frac{\sum(T-T_0)}{(T_r-T_0)}\Delta t}$$ Equation 6-2

t_e = equivalent age at the reference temperature, T_r = reference temperature

In this case equivalent age represents the duration of the curing period at the reference temperature that would result in the same maturity as the curing period at the actual temperature. The above equation can be written as follows:

$$\boldsymbol{t_e = \sum_0^t \alpha \Delta t}$$ Equation 6-3

Where, α = age conversion factor.

The age conversion factor converts a curing interval Δt to the equivalent curing interval at a standard reference temperature. For example if we assume a datum temperature of 0°C and reference temperature of 20°C, then the age conversion factor for a specimen cured at 40°C for 3 days according to Eq. 6-3 is 3 ((40-0)/(20-0)) and the equivalent age, t_e is 6 days.

It has to be noted that Eq. 6-3 assumes a linear relationship between rate of strength gain and curing temperature. However it was not true in most cases [124]. Therefore later in 1977, [129], Hansen and Pedersen proposed a new time temperature function based on the Arrhenius equation. The new function allowed the computation of the equivalent age of concrete as follows:

$$\mathbf{t_e = \sum_0^t e^{\frac{-E\left[\frac{1}{T}-\frac{1}{T_r}\right]}{R}} \Delta t}$$ Equation 6-4

t_e = the equivalent age at the reference temperature, E = apparent activation energy, J/mol, R = universal gas constant, 8.314 J/mol-K, T = average absolute temperature of the concrete during interval Δt, Kelvin, and T_r = absolute reference temperature, Kelvin.

Carino (2001) [124], suggested that obtaining accurate values for activation energy is not practical and proposed a simplified equation (Eq. 6-5). It was also shown that both the equations (Eq. 6-4) and Eq. 6-5) give the same strength values from maturity – strength models. Moreover it was also stated that the sensitivity factor in Eq. 6-5 has more physical significance compared with activation energy. In the present study Eq. 6-5 is used for its simplicity.

$$t_e = \sum_0^t e^{B(T-T_r)} * \Delta t$$ Equation 6-5

Then the expression for the age conversion factor becomes the following:

$$\boldsymbol{\alpha = e^{B(T-T_r)}}$$ Equation 6-6

B = temperature sensitivity factor, 1/°C

Figure 6-23 illustrates the importance of equivalent age and the summation involved in Eq. 6-5. The figure exemplify a situation in which the FBM curing condition was changed to 20°C after curing for 40 days at 5°C. As discussed earlier (Figure 6-12), the rate of stiffness gain is dependent on the stiffness achieved by the mixture. Therefore the equivalent age term considers this effect and adjusts the stiffness increase to the rate associated with the new conditioning temperature and the current stiffness. To explain graphically, when the curing condition changed from 5°C to 20°C on the 40th day, the stiffness gain rate was given by a horizontally transferred 20°C curve rather than the rate of the actual 20°C curing curve on the 40th day.

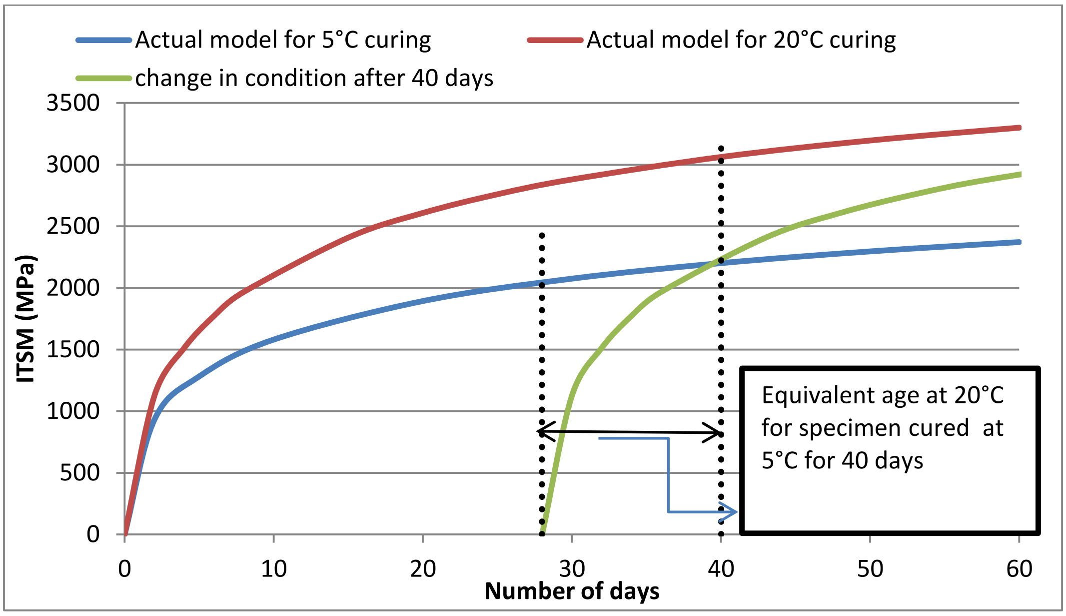


Figure 6-23 illustrating equivalent age (Curing temperature was changed from 5°C to 20°C at 40 days)

6.5.1.3 Formulating the effect of temperature on strength gain

As rate of strength or stiffness gain is dependent on the curing temperature it is important to formulate temperature dependency in maturity – strength models. This can be done by a temperature dependent factor called the rate constant (k), which is the initial slope of the strength (or) stiffness versus time of the curing curve at a specific constant temperature. Carino et al., (1992) [130] examined the following relationship for describing the rate constant k(T) as a function of temperature.

$$\boldsymbol{k(T) = Ae^{BT}} \qquad \text{Equation 6-7}$$

A = the value of the rate constant at 0 °C

6.5.1.4 Strength – Maturity Relationships

Many equations have been proposed to model the strength gain of concrete, but three are often used. These three functions are the exponential, hyperbolic, and logarithmic functions, which are defined as follows [128]:

Exponential:

$$\boldsymbol{S = S_u e^{-\left[\frac{\tau}{M}\right]^{\beta}}} \qquad \text{Equation 6-8}$$

S = compressive strength (MPa), Su = limiting compressive strength (MPa), M = maturity index (hours), τ = characteristic time constant (hours), and β = shape parameter.

Hyperbolic:

$$\boldsymbol{S = S_u \frac{k(M-M_0)}{1+k(M-M_0)}} \qquad \text{Equation 6-9}$$

$$\boldsymbol{S = S_u \frac{\sqrt{k(M-M_0)}}{1+\sqrt{k(M-M_0)}}} \qquad \text{Equation 6-10}$$

Mo = maturity when strength development is assumed to begin (°C • hours or hours), and k = rate constant (1/[°C • hours] or 1/hours).

The differences between Eq. 6-9 and Eq. 6-10 as stated by Carino (2001) [124] are in terms of the hydration kinetics of individual cement particles. Eq. 6-9 is derived based on linear kinetics, which means that the degree of hydration of an individual cement particle is a linear function of the product of time and the rate constant. Eq. 6-10 is based on parabolic kinetics. Thus Eqs.6-9 and 6-10 are called as the linear hyperbolic and parabolic hyperbolic models.

Logarithmic:

$$\boldsymbol{S = \frac{a}{S_0} + \frac{b}{S_0} \log M} \qquad \text{Equation 6-11}$$

Where, S_0 is strength when strength development is assumed to begin. The constants *a* and *b* are related to the water – cement ratio of the concrete and the type of cement.

6.5.2 Applicability of Maturity method to FBMs

As discussed the maturity method is a technique commonly used in the concrete industry to account for the combined effects of time and temperature on the strength gain. The notion of applying the maturity method arises from the fact that the stiffness of a given FBM is dependent on age and temperature history as seen in the previous two sections. Moreover, similar to concrete, at early ages temperature has a predominant effect on stiffness gain in FBMs. On account of this temperature dependency it is not straightforward to estimate the in-situ properties based on the laboratory test data obtained under standard laboratory conditions [128]. A function of time and temperature was considered an appropriate approach to describe the behaviour of these materials [126]. Therefore applicability of the maturity method to FBM stiffness gain is explored and modifications are also proposed for predicting stiffness as accurately as possible.

Modifications to the maturity or time-temperature functions were needed as the mechanism by which FBMs gain strength is different from that of concrete. The primary difference is in the definition of curing in the concrete industry and for FBMs. With regard to concrete, curing is a process in which the concrete is protected from loss of water and the concrete strength development occurs during the curing period when water is available for cement hydration. In contrast to that, for FBMs curing is a process in which the compacted material discharges water through evaporation and gains stiffness or tensile strength by formation of continuous bituminous mastic film [112]. The other characteristic of concrete is its 'crossover behaviour' , where concrete cured at higher temperature initially has higher strength but later has lower strength than concrete cured at lower temperature. The crossover behaviour of concrete is illustrated in Figure 6-24. This crossover is because a higher initial temperature results in more than a proportional increase in the initial rate of hydration resulting in reacted products not having time to uniformly distribute within the pores of the hardening paste [124]. However, this crossover effect is not seen in the FBM curing process, as seen in Figure 6-12 and Figure 6-14. Similar results have also been found in other research investigations [126, 131]. It has to be noted that the results from these figures were for a curing period of around 30 days whereas crossover relates to the long-term strength of concrete. Nonetheless, analogous results were found for much longer curing periods which will be shown in subsequent sections.

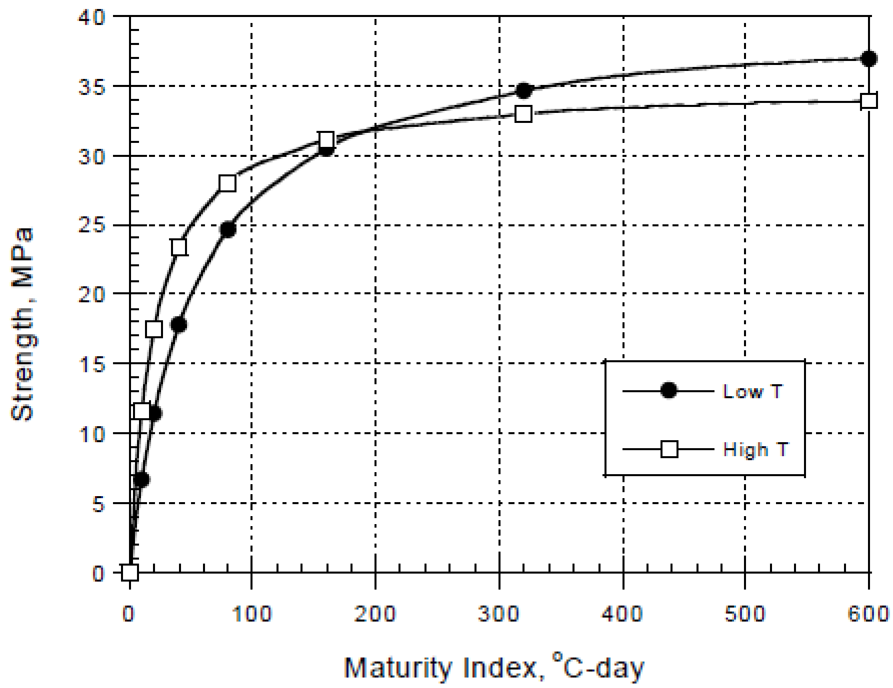


Figure 6-24 The crossover effect in concrete [124]

In the present discussion, maturity refers to the function of temperature and equivalent age computed using a time-temperature function. This is a slightly different interpretation from the definition given by Malhotra and Carino [128] in which maturity was referred to either a temperature – time factor computed using the Nurse-Saul function (Eq. 6-1) or equivalent age computed using any of the maturity functions (Eq. 6-2, Eq. 6-4 and Eq. 6-5). The general steps followed in developing the maturity - stiffness model for each mixture considered are presented below

- The FBM specimens were cured at different curing temperatures (5°C, 20°C, and 40°C).

- ITSM (stiffness) tests were performed at different intervals of time. The values of rate constant (k) at each temperature were determined by fitting stiffness data to stiffness – age relationships for isothermal conditions.
- Temperature sensitivity factor (B) was determined by fitting the Arrhenius equation (Eq. 6-7) to rate constant (k) versus temperature data.
- Age conversion factor (α) was calculated (Eq. 6-6) for each time period that the ITSM test was conducted.
- Equivalent age (t_e) was obtained for each time period using Eq. 6-5
- Maturity was calculated for all test data and plotted against stiffness.
- A Best fit maturity – stiffness relationship was obtained by fitting maturity versus stiffness data.

The methodology adopted for maturity – strength relationship will be explained with reference to 50%RAP + 1%cement mixture.

6.5.2.1 Stiffness-age relationships

It was identified in the previous section that a logarithmic fit best describes the stiffness increase of FBMs over time (Figure 6-12). This is because the logarithmic function predicted the high initial rate of stiffness gain well. However, the logarithmic function overestimated the stiffness of these mixtures over a long period of time. This can be seen in Figure 6-25 to Figure 6-27. Figures present the average ITSM test results for the FBM (50%RAP + 1% Cement) cured and monitored over a long period (296 days). The short term curing data (up to 30 days) is the same as that presented in Figure 6-12 and Figure 6-14. The specimens cured at 30°C were not cured long term and were used for validation of the approach. Because of the overestimation of the logarithmic function in the long term, a hyperbolic equation is used to fit the stiffness data over time. Hyperbolic equations for strength gain under isothermal curing for concrete are as shown in Eq. 6-12 and Eq. 6-13. Theoretical analysis of the expressions (hyperbolic equations) to describe the strength development of concrete can be found in Carino (1984)[132]. In the analysis it was also shown that under isothermal conditions the strength gain of the concrete can be described by a hyperbolic curve (Eq. 6-12 and Eq. 6-13). Maturity term (M) was replaced with time term (t) from Eq. 6-8 and Eq. 6-9.

$$\boldsymbol{S = S_u \frac{k(t-t_0)}{1+k(t-t_0)}} \qquad \text{Equation 6-12}$$

$$\boldsymbol{S = S_u \frac{\sqrt{k(t-t_0)}}{1+\sqrt{k(t-t_0)}}} \qquad \text{Equation 6-13}$$

Where, S is strength at age t; S_u is the asymptotic value of the strength for the hyperbolic function that fits the data; k is the rate constant which is related to the rate of strength gain at a constant temperature; t_0 is age at the start of strength development. It was also shown by Carino (1984) [132] that these parameters are temperature dependent and can be obtained by conducting the regression analysis on strength data obtained at different curing temperatures.

In the present study as the material property considered is stiffness (ITSM), strength (S) is replaced by stiffness (E) in Eq. 6-12 and Eq. 6-13. It is assumed that the stiffness development starts at age t_0 as per these models. However, in the present study it was assumed that stiffness development starts immediately after compaction (t_0 = 0). Thus the two equations (Eq. 6-12 and Eq. 6-13) can be re-written as follows.

$$E = E_u \frac{kt}{1+kt} \quad \text{Equation 6-14}$$

$$E = E_u \frac{\sqrt{kt}}{1+\sqrt{kt}} \quad \text{Equation 6-15}$$

The stiffness – time data for the three curing temperatures (5°C, 20°C and 40°C) was fitted by the hyperbolic relationships given by Eq. 6-14 and Eq. 6-15. There are several approaches to determine the parameters limiting stiffness (E_u) and k. The simplest approach is to conduct regression analysis using the least square curve fitting technique. The data in Figure 6-25 to Figure 6-27 were subjected to regression analysis and the best fit curves for logarithmic and the two hyperbolic curves can be seen in the figures. As can be seen, the parabolic hyperbolic equation (Eq. 6-15) best described the trend. The coefficient of determination, R^2, found from regression analysis for the test data is presented in Table 6-5 for all the mixtures tested. The table confirms that the parabolic hyperbolic equation is the best fit to the stiffness over time.

The limiting stiffness, E_u and the rate constant, k obtained from the regression analysis for each curing temperature are summarised in Table 6-6.

Long term curing data fitted to the hyperbolic parabolic model for 50%RAP+1% Cement – FBM for all curing temperatures is presented in Figure 6-28. It is evident from the figure that the limiting stiffness (E_u) is directly dependent on curing temperature. The higher the curing temperature the higher was the limiting stiffness which means there was no crossover effect found in FBMs. Moreover, the temperature dependency of the limiting stiffness was nearly linear.

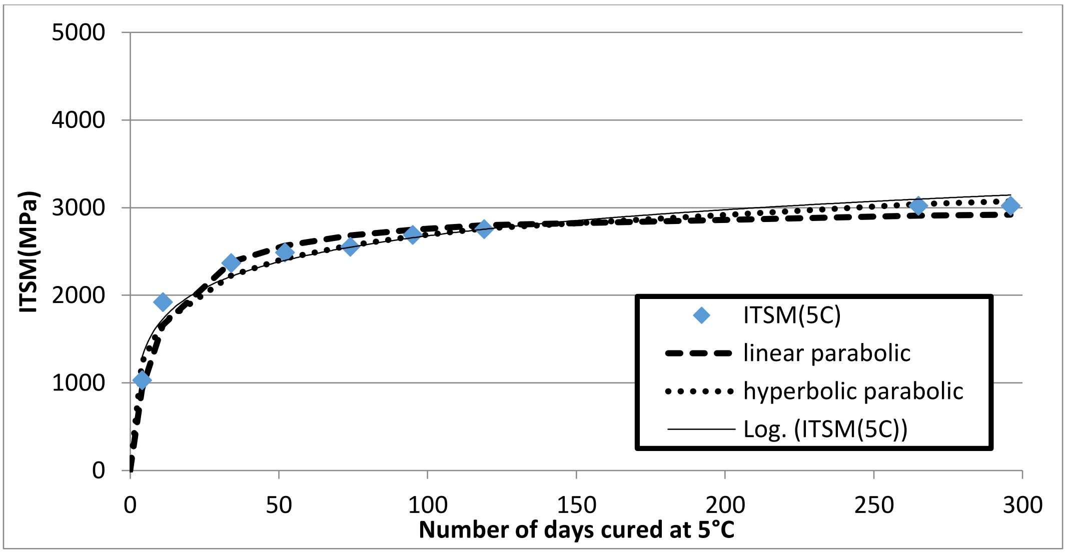


Figure 6-25 Stiffness development relationships for 50%RAP + 1%Cement- FBM cured at 5°C

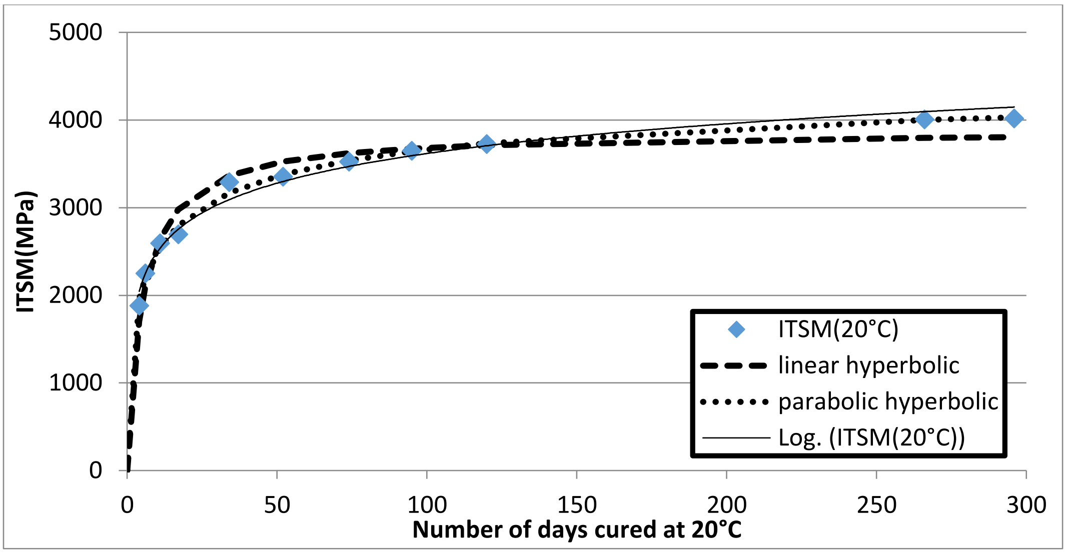


Figure 6-26 Stiffness development relationships for 50%RAP + 1%Cement- FBM cured at 20°C

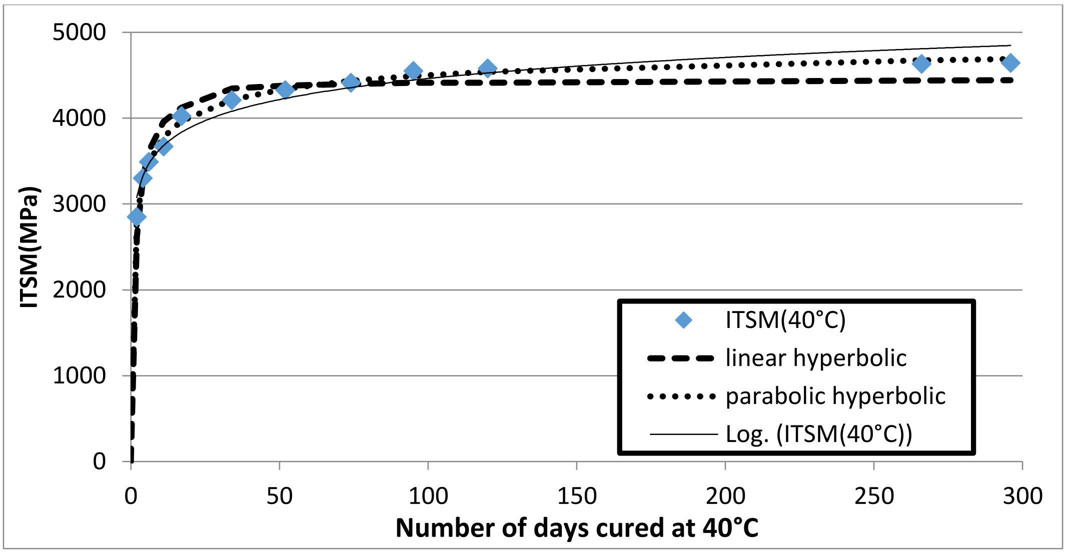


Figure 6-27 Stiffness development relationships for 50%RAP + 1%Cement- FBM cured at 40°C

Table 6-5 Coefficient of determination (R^2) obtained by fitting different age-stiffness models

Coefficient of determination (R^2) from regression analysis									
Age-stiffness models	logarithmic			linear hyperbolic			parabolic hyperbolic		
Curing temperature	5°C	20°C	40°C	5°C	20°C	40°C	5°C	20°C	40°C
50%RAP+1%Cement	0.94	0.97	0.94	0.96	0.97	0.97	0.98	0.99	0.97
50%RAP	0.99	0.96	0.78	0.98	0.95	0.99	0.98	0.99	0.96
100%VA+1%Cement	0.98	0.93	0.94	0.91	0.96	0.95	0.97	0.98	0.99
100%VA	0.99	0.9	0.84	0.9	0.98	0.95	0.99	0.97	0.99

Table 6-6 Summary of parameters obtained by fitting age versus stiffness data

Temperature	5°C		20°C		40°C	
Parameters	k(1/day)	E_u(MPa)	k(1/day)	E_u(MPa)	k(1/day)	E_u(MPa)
50%RAP+1%Cement	0.06	3622	0.13	4687	0.9	4977
50%RAP	0.01	2842	0.08	3377	0.72	3489
100%VA+1%Cement	0.07	3199	0.28	4253	2.16	4556
100%VA	0.02	2620	0.05	3834	0.58	3895

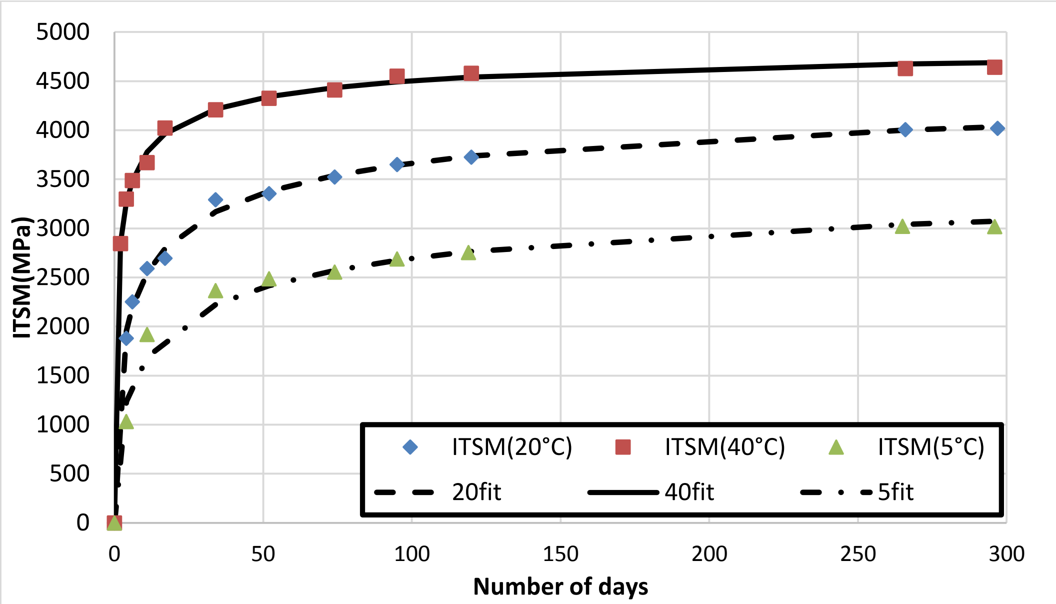


Figure 6-28 Long term curing data fitted to hyperbolic parabolic model for 50%RAP+1% Cement - FBM

6.5.2.2 Temperature sensitivity factor

This section examines the variation of the value of the rate constant (k) with curing temperature in order to obtain temperature sensitivity factor (B). As was discussed, the rate constant (k) is related to the rate of stiffness gain at a constant temperature, and it was obtained from an appropriate equation of stiffness gain versus age. In order to describe stiffness development under variable curing temperatures it is necessary to determine how the rate constant is affected by curing temperature. For this the k values in Table 6-6 are plotted against curing temperature as can be seen in Figure 6-29. The best fit Arrhenius equation (Eq. 6-7) was determined. The parameters that were obtained in the analysis are tabulated in Table 6-7. A is the value of the rate constant at 0°C and B is the temperature sensitivity factor.

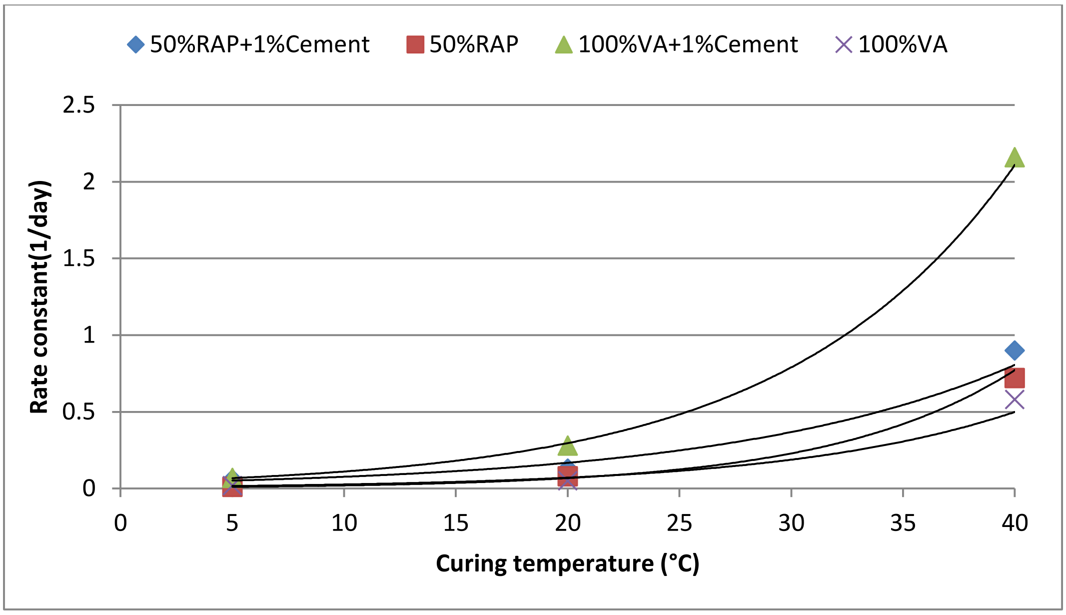


Figure 6-29 Variation of rate constant (k) with temperature

Table 6-7 Values of temperature sensitivity factors (B)

Mixture type	A	B(1/°C)	R-square
50%RAP+1%Cement	0.035	0.078	0.97
50%RAP	0.006	0.121	0.99
0%RAP+1%Cement	0.041	0.098	0.99
0%RAP	0.01	0.097	0.96

6.5.2.3 Age conversion factor and equivalent age

The age conversion factors for different temperatures were calculated using Eq. 6-6. The equation utilised the temperature sensitivity factors from Table 6-7. The reference temperature to which age was converted was assumed as 20°C. The age conversion factor (α) obtained for different mixtures is presented in Figure 6-30. The age conversion factor (α) at 20°C is 1 as the reference temperature considered in the study is 20°C. The equivalent age is the duration of time that a specimen would need to be cured at a specified reference temperature (in the present case 20°C) to equal the maturity of the specimen cured at various temperatures. The equation for equivalent age used in the present study is as given in Eq. 6-5. For developing the maturity – stiffness relationships, the specimens are cured at isothermal conditions. Therefore, for isothermal condition in Eq. 6-5, Δt can be replaced with t (Σ Δt) and summation is not needed. Thus Eq. 6-5 can be re written as Eq. 6-16 in which t is the elapsed time of test since compaction, B is temperature sensitivity factor (Table 6-7), T is curing temperature and T_r is reference temperature (in present case T_r was considered as 20°C).

$$t_e = e^{B(T-T_r)} * t \qquad \text{Equation 6-16}$$

It has to be noted that for calculating in-situ maturity where variable temperature exists Eq .6-5 has to be used as it is. The equivalent age obtained using Eq. 6-16 for a reference temperature of 20°C is plotted against stiffness in Figure 6-31. The equivalent age – stiffness data for 20°C condition remains the same as in Figure 6-26 as the reference temperature considered was 20°C.

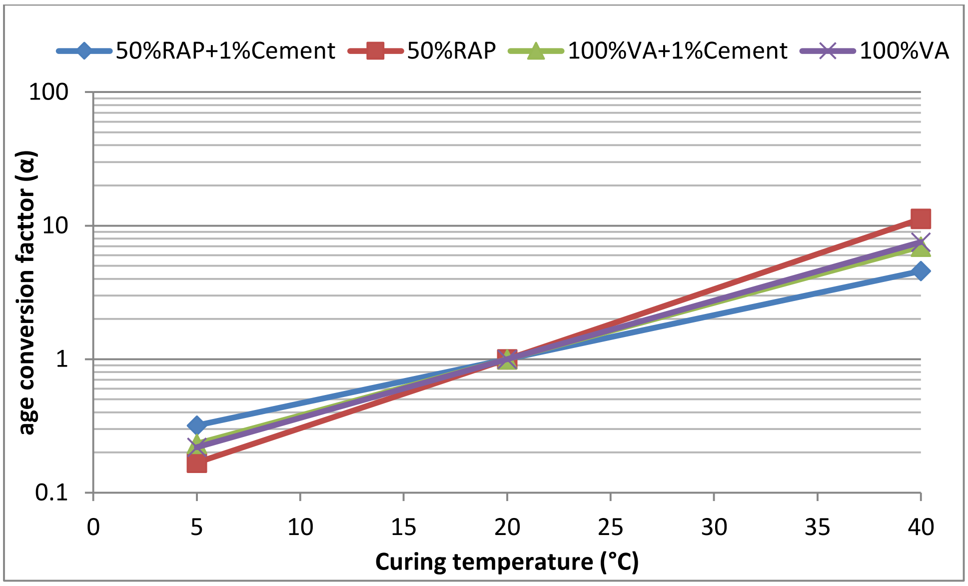


Figure 6-30 Age conversion factor for reference temperature of 20°C

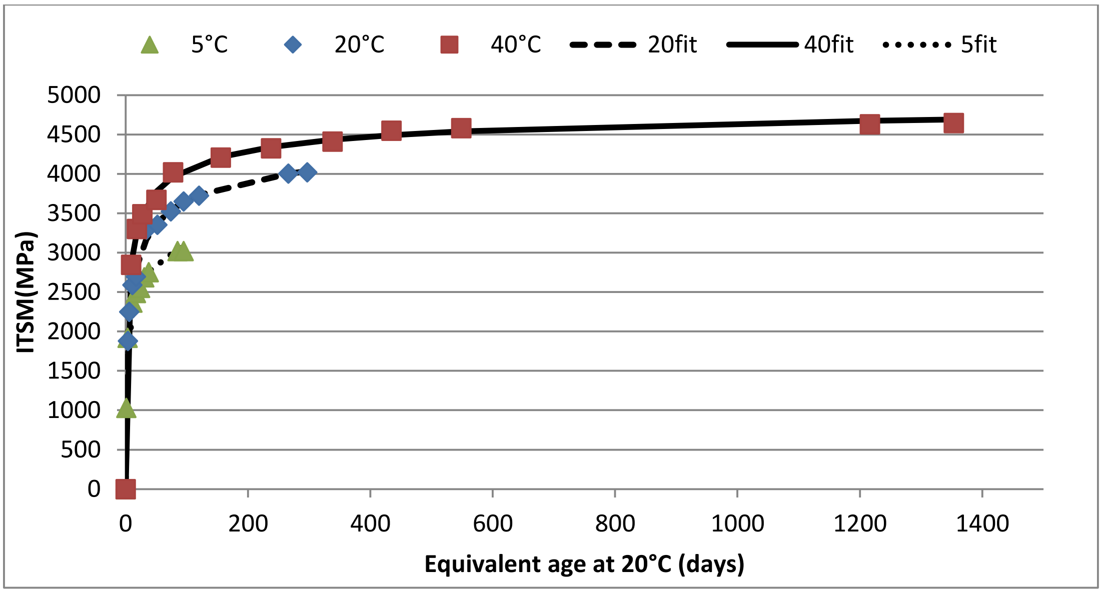


Figure 6-31 Equivalent age at 20°C for 50%RAP + 1% Cement - FBM

6.5.2.4 Maturity – stiffness relationship

According to the standard definition of maturity, samples of the same FBM should have equal stiffness if they have equal maturities irrespective of their curing histories. As the present study is dealing with the equivalent age approach the term maturity refers to equivalent age. Figure 6-31 clearly shows that the equivalent age doesn't have a unique relationship with stiffness. The traditional maturity rule is therefore an approximation in the present case as it assumes that the limiting stiffness or strength of the mixture subjected to continuous curing is not affected by early age temperature history. However, as was shown in Figure 6-24 and Figure 6-28, this was not the case in either FBMs or concrete. In both cases it is clear that the curing temperature not only affects the initial rate of stiffness but also the limiting values. Therefore the traditional maturity rule results in inaccurate results.

To resolve the nature of the above problem Carino (1984) [132] introduced a model for concrete called a rate constant model which states that there exists a unique relationship between relative strength (S/Su) and equivalent age. The relative strength can be obtained by dividing stiffness by the limiting stiffness obtained for the curing temperature that the specimen experienced (Table 6-5). Figure 6-32 is a plot of the relative stiffness versus equivalent age obtained for the 50%RAP + 1% Cement mixture. The data points very much group around a single curve and therefore it can be interpreted that there exists a relationship between relative strength and equivalent age. However, in FBMs unlike concrete there is no crossover effect as was shown in Figure 6-28. Moreover, the limiting stiffness (Table 6-6) was found to have nearly linear relationship with temperature. Therefore a temperature term is introduced into the equivalent age function (Eq. 6-16). The resulting function is Eq. 6-17. This equation is valid for isothermal curing. However, for variable temperature conditions the equation becomes as shown in Eq. 6-18. This equation is analogous to the Nurse-Saul maturity function (Eq. 6-1) in which the time term (Δt) is replaced with equivalent time (Δt_e) and the datum temperature is 0°C.

$$M = t_e * T = e^{B(T-T_r)} * t * T \qquad \text{Equation 6-17}$$

$$M = \sum_0^t \Delta t_e * T \qquad \text{Equation 6-18}$$

Maturity using Eq. 6-17 was calculated and plotted against stiffness in Figure 6-33. The datum temperature (T_0) was assumed as 0°C as there is direct influence of water evaporation on the mixture stiffness gain. The figure clearly shows that the data points are grouped around a single curve whose equation is as in Eq. 6-19 which is the same equation as Eq. 6-10 except that the datum temperature was assumed as 0°C. The stiffness values were fitted to maturity values with Eq. 6-19 using the least square curve fitting technique and the parameters E_u and k were obtained. The parameters obtained for all the mixtures considered in the present study are tabulated in Table 6-8.

$$\boldsymbol{E = E_u \frac{\sqrt{kM}}{1+\sqrt{kM}}} \qquad \text{Equation 6-19}$$

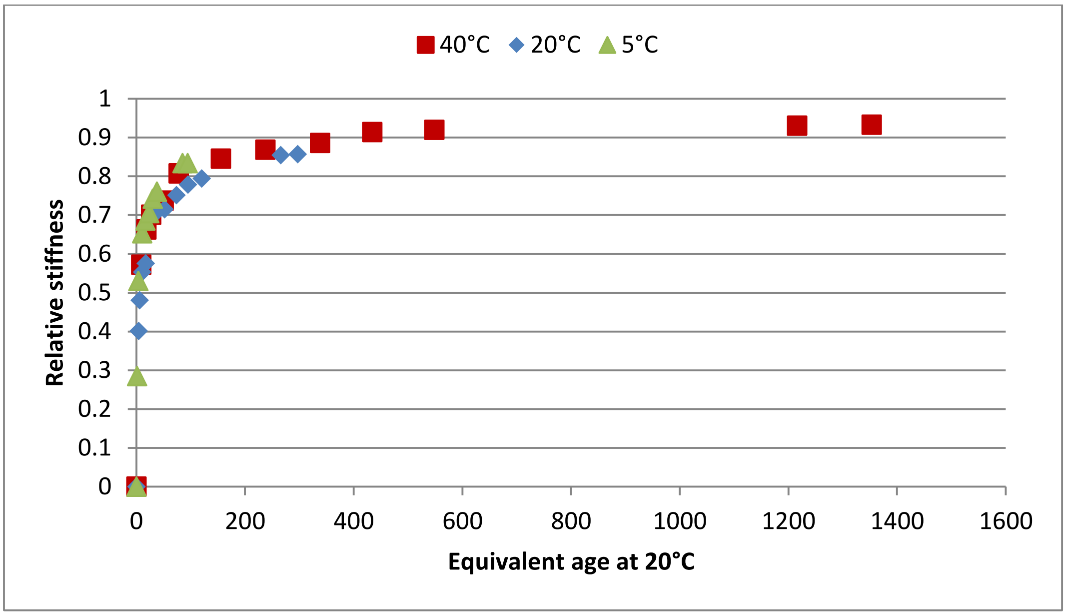


Figure 6-32 Relative stiffness versus equivalent age at 20°C for 50%RAP + 1%Cement - FBM

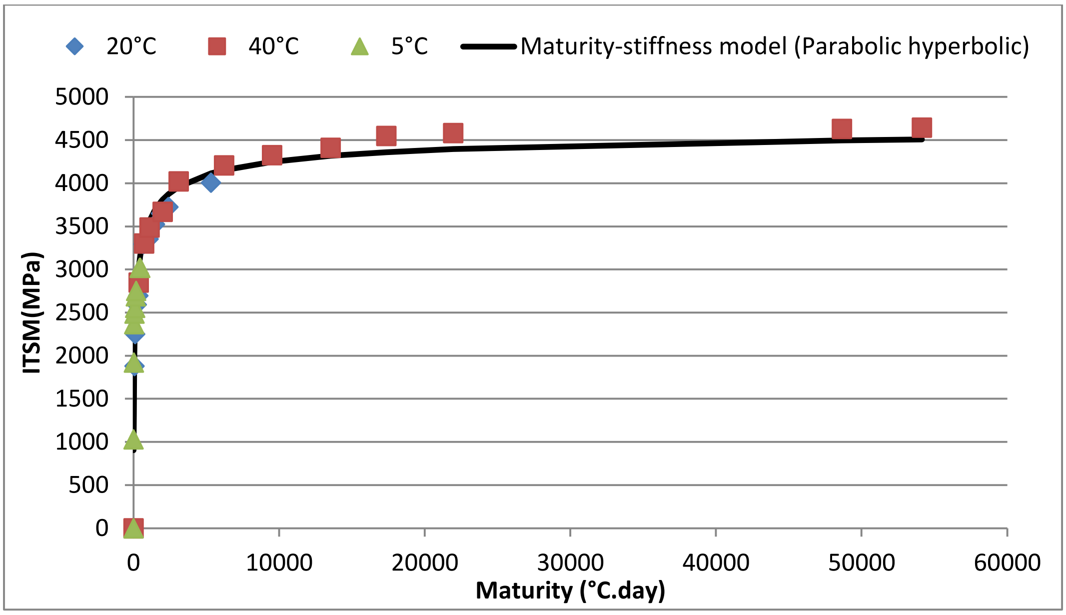


Figure 6-33 Maturity and stiffness relationship for 50%RAP + 1%Cement- FBM

Table 6-8 Parameters obtained for Maturity – Stiffness model

FBM - Type	E_u (MPa)	k(1/day)
50%RAP+1%Cement	4712	0.008
50%RAP	3187	0.011
0%RAP+1%Cement	4384	0.013
0%RAP	3764	0.002

6.5.2.5 Validation of the method

The experimental study also included validating the applicability of the maturity – stiffness models to other temperatures (isothermal) that were not included while developing the models. The study also tried to verify how well the model can address the effect of variations in temperature on FBM curing. For this, three conditions namely short term (less than 10 days), intermediate (between 10 days and 35 days) and long term (between 35 days and 300 days) were considered.

Isothermal curing temperature

For verifying isothermal condition the ITSM test data that was obtained for the curing mechanism study at 30°C was used. The stiffness (ITSM) data measured on the specimens cured at 30°C obtained for short term and intermediate term was plotted against the stiffness values obtained from the maturity-stiffness model that had been developed (Figure 6-34 to Figure 6-37). From these figures values corresponding to '30°C-30°C' are results for isothermal curing which indicate specimens were conditioned at 30°C during the early stage (less than 7 days after compaction) and as well as during the intermediate stage (between 10 days and 35 days). Figure 6-34 to Figure 6-37 indicate that the maturity-stiffness model estimates the stiffness reasonably well in all cases except for 50%RAP-FBM in which the model has overestimated slightly.

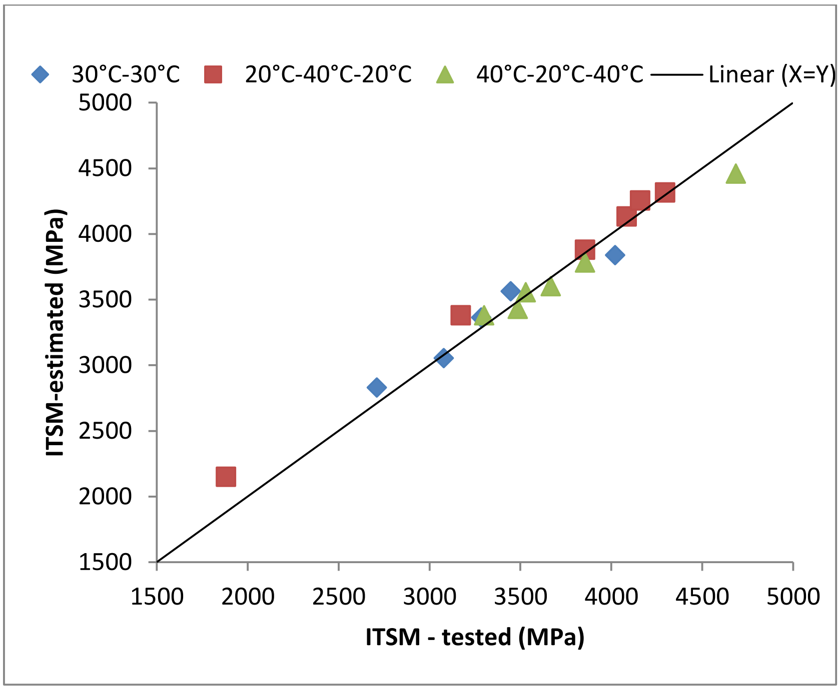


Figure 6-34 Verification of Maturity - Stiffness model for 50%RAP + 1%Cement - FBM

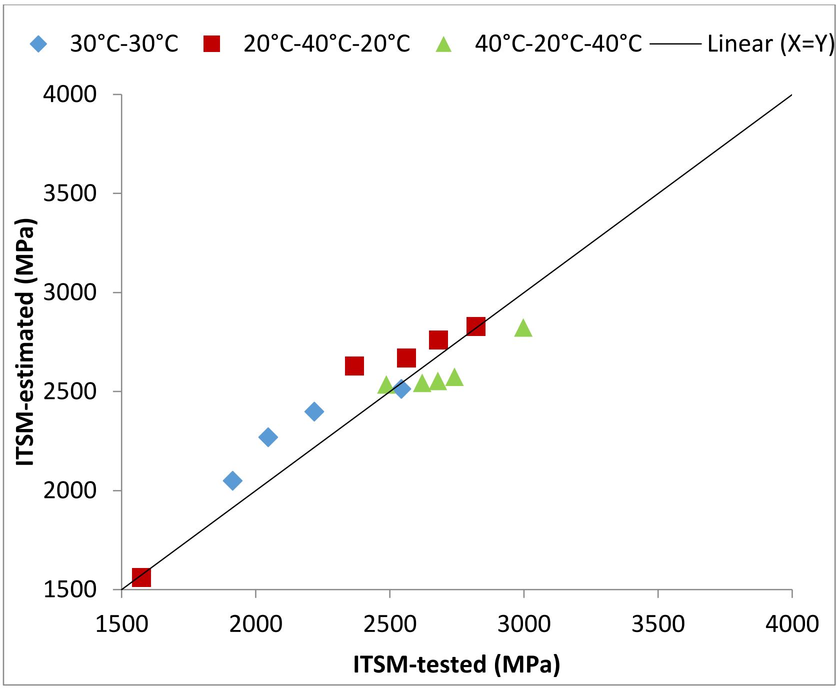


Figure 6-35 Verification of Maturity - Stiffness model for 50%RAP - FBM

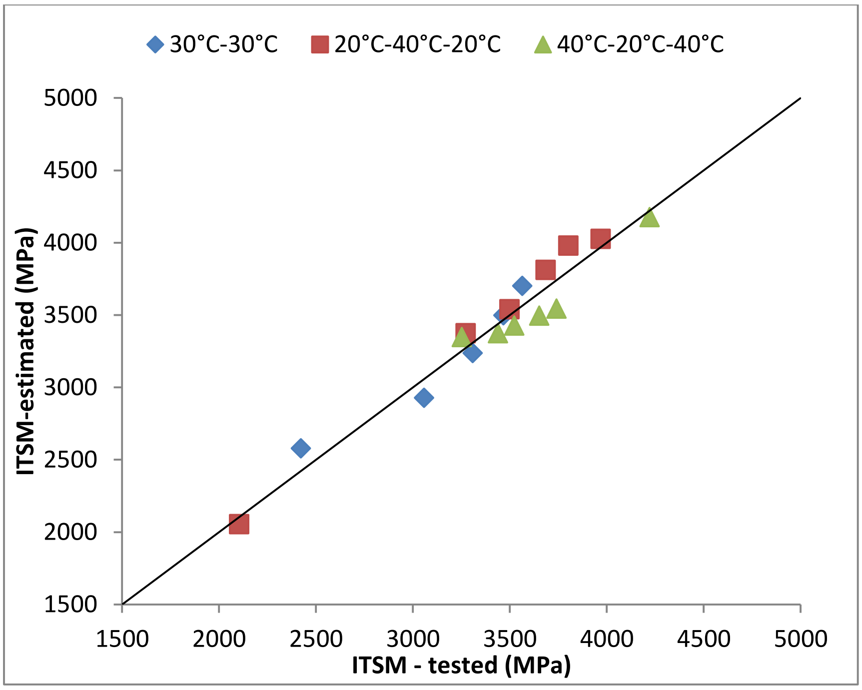


Figure 6-36 Verification of Maturity - Stiffness model for 100%VA+1%Cement - FBM

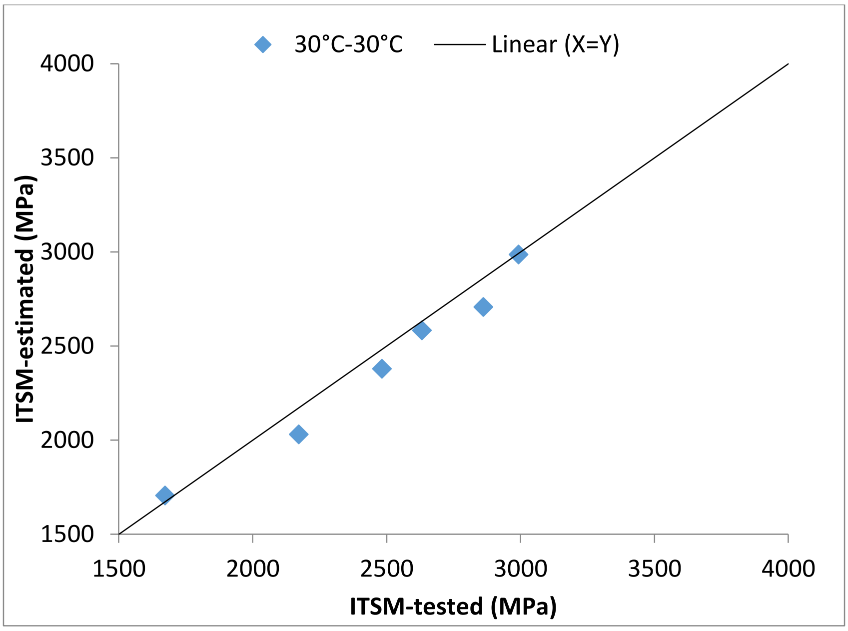


Figure 6-37 Verification of Maturity - Stiffness model for 100%VA - FBM

Variable curing temperatures

To verify how well the Maturity-Stiffness model addresses varying temperatures, the specimens were cured at two variable temperature conditioning sequences. The first case is conditioning the specimens at 20°C during the early stage curing (up to 7 days after compaction) and then changing to a 40°C cabinet to cure during intermediate stage (7 days to 35 days) and finally the specimen was transferred back to the 20°C conditioning cabinet for long term curing. This conditioning sequence is termed as '20°C-40°C-20°C'. Similarly the second sequence studied was 40°C-20°C-40°C. The specimens were tested for ITSM at different intervals of time.

Table 6-9 presents the curing sequence for 20°C-40°C-20°C for 50%RAP+1%Cement-FBM. It also shows the calculations for maturity and from it the stiffness values that were predicted. Maturity was calculated using Eq. 6-16 and Eq. 6-18. Stiffness (ITSM) was estimated using Eq. 6-19 using corresponding values from Table 6-8 and maturity values. The first three values calculated in Table 6-9 are presented in Figure 6-38 for comparison. Figure 6-38 illustrates graphically the estimation of stiffness comprising the tested value and estimated value for stiffness from the Maturity-Stiffness model. As can be seen in the figure the conditioning has changed from 20°C initially to 40°C at 4 days. The dotted lines in the figure demonstrate the actual path of the stiffness increase for this particular sequence of curing. The figure suggests that though the stiffness values obtained from the maturity-stiffness model were marginally higher than the tested values, the model predicted the stiffness adequately in the particular case of 50% RAP + 1%FBM as can be seen in Figure 6-34. Figure 6-35 and Figure 6-36 shows how well the model predicted stiffness gain for variable temperature conditions for 50% RAP and 100% VA + 1%Cement FBMs.

It is worth noting that the model slightly underestimated the stiffness of the samples which were cured initially at 40°C and marginally overestimated specimens that were cured at 20°C. Underestimation for the 40°C-20°C-40°C sequence could be attributed to the slightly conservative nature of the model (Figure 6-33). The reason for overestimation could probably be the failure of the mixture to gain stiffness at the rate the model estimated after changing the conditioning temperature. This shortcoming can also be seen in Figure 6-38. From the above discussion it can be stated that though estimated stiffness from the developed model marginally differs from the test results, they are close enough to state that the maturity approach provides a satisfactory basis for estimating the stiffness of FBMs. Furthermore, the model also quantifies the stiffness gain well for varying temperature conditions.

Table 6-9 Maturity calculations for 20°C-40°C-20°C sequence of curing for 50%RAP+1% Cement-FBM

Curing sequence-20°C-40°C-20°C						
ITSM test (MPa)	Time (days)	Δt (days)	T (°C)	Δt_e (days)	Maturity (°C.day)	ITSM-Predicted (MPa)
1880	4	4	20	4	80	2152
3171	8	4	40	18	732	3382
3856	21	13	40	59	2458	3880
4084	39	18	40	82	5750	4133
4159	61	22	40	101	9773	4255
4295	242	181	20	181	13393	4316

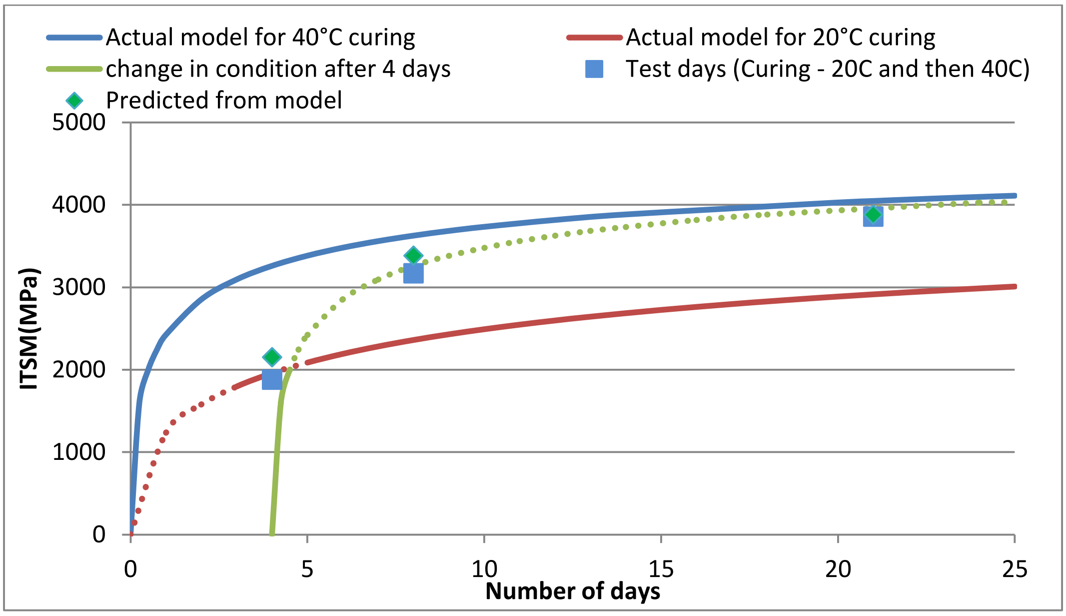


Figure 6-38 Effect of temperature variation on stiffness (50%RAP + 1% Cement)

6.5.2.6 Practical implications

As seen in the previous section the maturity method was found to be capable of estimating the stiffness under varying temperature conditions, and it provides a tool to estimate in-situ stiffness after compaction of FBM layers. This method is particularly helpful to assess when the FBM layer can be covered with overlying layers if the in-situ temperature data is available. To explain the applicability of the maturity method three hypothetical pavement sections were considered. The locations were selected so that each represented moderate to cold temperature so as to evaluate the effect of ambient temperature on the stiffness gain of FBM. The temperature data was obtained from the National Climatic Data Centre (NCDC) website (www.ncdc.noaa.gov). Figure 6-39 presents the mean daily temperatures for April, 2014 in the locations selected. It was assumed that the construction (compaction) of the FBM layer was carried out on 1st of April and the mixture started gaining stiffness from the first day. The stiffness values for each day were calculated using the cumulative maturity approach as discussed in the previous section.

The stiffness results estimated using the maturity method are presented in Figure 6-40. As expected the higher ambient temperatures resulted in higher stiffness values. The results show the stiffness trend for mixtures with 50%RAP, one with 1% Cement and the other without any cement, to explain the benefit of cement addition. These trends help in taking decisions regarding when the overlying layer can be placed on the FBM layer or when the traffic can be allowed to pass on it. For example if an agency requirement is to place the binder course or surface course over the FBM layer only if it reaches a stiffness of 1000 MPa , in such circumstances these trends can be very helpful. Table 6-10 shows the effect of location and presence of cement on the number of days required to reach a stiffness of 1000 MPa. As can be seen from the table both location and presence of cement significantly influence the stiffness trend and thus decisions such as trafficking and layer placements. For moderately warmer (Nottingham) and colder (Stockholm) conditions the difference in days to reach 1000 MPa was found to be 7 days and 9 days respectively for mixtures with cement and without cement. The influence of the cement on the early stage of curing was significant as the difference between mixtures with cement and without cement for the colder condition (Stockholm) considered in the present study was as high as 13 days.

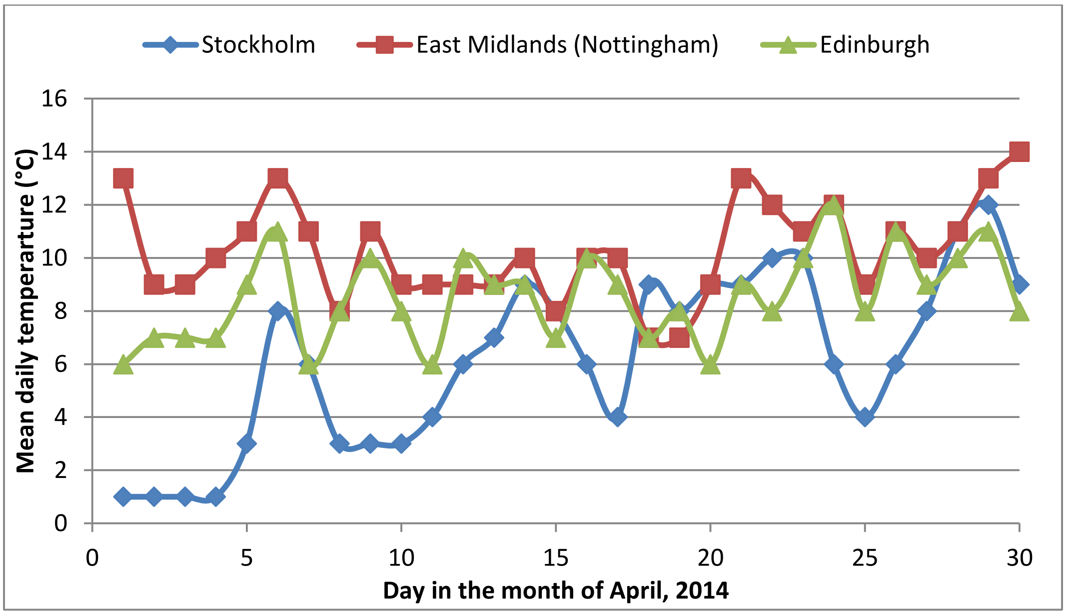


Figure 6-39 Mean daily temperatures for month of April, 2014 in locations considered in the study.

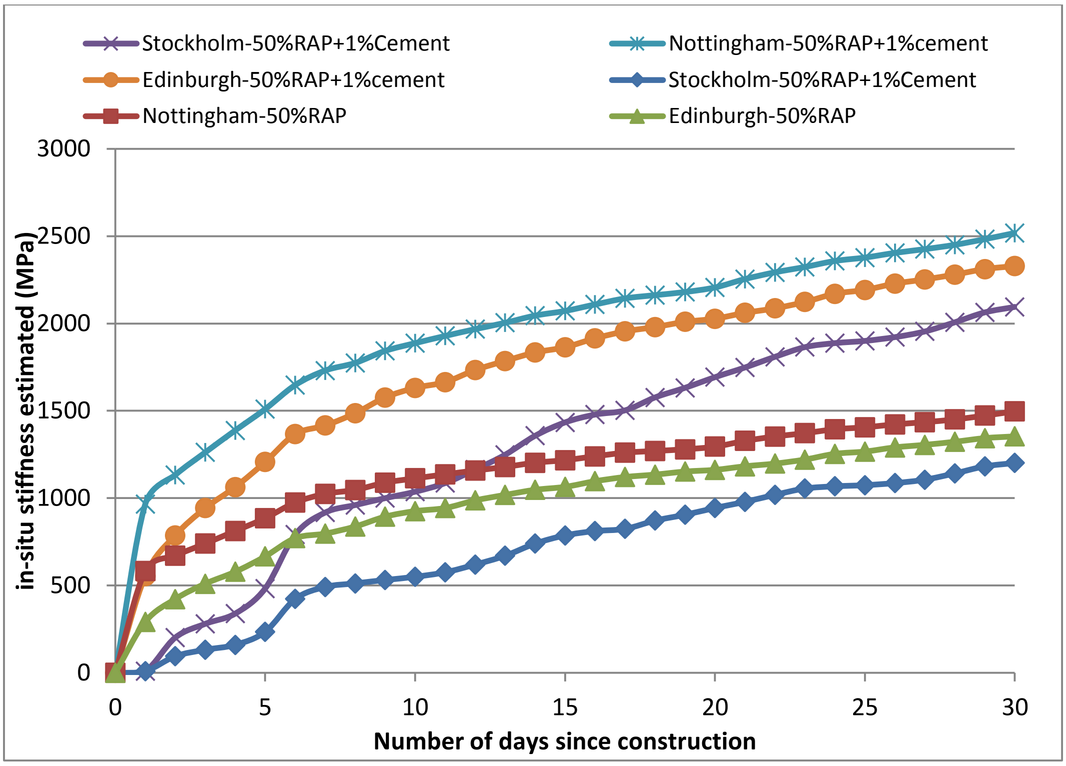


Figure 6-40 Effect of location and cement on stiffness gain (hypothetical sections)

Table 6-10 Practical implication of maturity method

Number of days to reach stiffness of 1000 MPa			
	Stockholm	Edinburgh	Nottingham
50%RAP + 1%Cement	9	4	2
50%RAP	22	13	7

6.6 Summary

This chapter was intended to better understand the curing mechanism of FBMs and to lessen the gap between laboratory curing and field evolution of these mixtures. This was achieved by evaluating different curing regimes that are being followed by different agencies and researchers, as well as identifying important parameters that affect curing. In achieving this, a link was established between laboratory mix design and field performance by evaluating applicability of the maturity method.

The curing regime study provided a valid investigation into the behaviour of FBM taking into account the effect of temperature, curing conditioning (Sealed or Unsealed), curing duration and the influence of cement with different curing regimes. The following inferences were drawn based on the experimental study performed.

- Curing temperature has a significant effect on the rate of curing and the 28-day strength and stiffness values. The ratio of 28-day stiffness results of specimens cured unwrapped to those cured fully wrapped is typically 2-2.5, independent of curing temperature. The equivalent indirect tensile strength ratio is 1.5-2.5. The inclusion of 1% cement in the fully wrapped case leads to strength and stiffness results approximately equal to those of unwrapped specimens without cement, suggesting that 1% cement addition is an appropriate means of avoiding the effects of adverse weather during curing.
- Strengths and stiffness results at 3 days are approximately 50-70% of those at 28 days, illustrating the danger of damage due to early trafficking.
- A 2-stage curing regime (COMBO) intended to simulate site practice produced strength and stiffness data that lay between the wrapped and unwrapped 28-day results. Although the curing conditions FW-40°C-28 days and UW-20°C-28 days slightly overestimating the properties compared COMBO, they seem to best represent the actual conditions during trafficking if COMBO is assumed to represent actual conditions in the field. Exactly the same trends appeared from tests for permanent deformation resistance as for strength and stiffness indicating that mechanical tests (ITSM and ITS) are sufficient for the mixture selection during mix design.
- The study also found that specifying in-situ water content as a criterion is not an appropriate approach as the same water content levels might give different stiffness values if cured at different temperatures. This leads to the suggestion that specifying in-situ stiffness could be a rational approach and a tool for estimating in-situ stiffness was developed using the maturity method.

The study also aimed to understand the curing mechanism and to interpret the level of impact that factors such as temperature, time, presence of RAP and active filler have on the curing of FBMs. The effect of these factors on curing was evaluated with reference to stiffness gain and water loss of specimens made of these mixtures. The following observations were made.

- The temperature is as important a parameter as time, as temperature has a greater influence on curing rate and also on bitumen properties.
- Higher curing temperatures resulted in higher rate of stiffness gain. This trend is not only because of rapid water loss but also with an increase in binder stiffness at higher curing temperatures. Though the presence of RAP improved the early stage stiffness of FBMs, it slowed down the rate of water loss from the specimens which resulted in smaller stiffness values at a later stage.
- The cement addition has no influence on water loss trends, but improved the stiffness significantly during all stages of curing.

The study also evaluated the applicability of the maturity method as a tool to assess the in-situ characteristics of FBM layers in the pavement. It was found that replacing the time term with an equivalent age term in the maturity function aided in estimating stiffness rather than relative stiffness. This was possible because of the characteristic curing of FBM in which the limiting stiffness that these mixtures reach strongly depends on the curing temperature at least for the length of curing stages considered in the present study. A strong correlation was found between maturity and the stiffness values obtained from the laboratory tests which resulted in development of maturity-stiffness relationships. The application of the method to assess the in-situ stiffness was presented using three hypothetical pavement sections. The results showed the influence of ambient temperature and the importance of cement addition to FBMs.

7 PERFORMANCE CHARACTERISTICS

7.1 Introduction

This chapter discusses the performance characteristics of Foamed Bitumen Treated Mixtures (FBMs) in terms of their permanent deformation and fatigue life. The performance indicators that were determined using the laboratory tests on FBM were also compared with the conventional Hot Mix Asphalt (HMA) characteristics. Therefore, the experimental study includes the testing of three FBM mixes namely, mixture with 50% of RAP material (50%RAP-FBM), 50% of RAP material and 1% of ordinary Portland cement (50%RAP+1%Cement-FBM) and 75% of RAP material and 1% of ordinary Portland cement (75%RAP+1%Cement-FBM), and a conventional HMA which is a dense binder course recipe mixture with 90 pen bitumen (DBM 90). HMA specimens were manufactured in accordance with the specifications outlined in BS 4987-1:2005 with 20mm maximum size of the aggregate and the bitumen used was the same bitumen that was used in the foaming process (70/100 penetration grade bitumen). The physical properties of the bitumen and the manufacturing procedure for FBMs can be found in Chapter 5.

7.2 Permanent deformation characteristics

In the pavement layers with bituminous mixtures that are subjected to traffic loads, stresses are induced which are usually very small when compared to their strength. These stress pulses cause strain most of which is recoverable. The irrecoverable strain is very small for one load application. However, for repeated loads which are the actual case in pavements, these small irrecoverable strains accumulate. This accumulated permanent strain induced by traffic loads manifests itself as permanent deformation in pavement layers. A number of researchers have studied the effect of different factors on permanent deformation in FBMs such as the grade of bitumen used for foaming [85], foamed bitumen content [88], active filler type and its content [43], RAP content and age [85, 87], water content in the specimen during the test [87] and applied stress [50, 81].

From the literature review it was understood that though most researchers have considered temperature as a major factor that affects the permanent deformation behaviour of FBMs little effort has been made in undertaking experimental investigation of this effect. However, a considerable amount of investigation has been carried out on the mechanical properties such as stiffness and strength [21, 79, 133]. Saleh (2006) and Li (2011) compared the temperature susceptibility in terms of stiffness and strength of FBM with HMA. The results showed that the strength and stiffness properties of HMA were more sensitive to temperature change than the corresponding properties of FBM. In view of these findings, the present study aimed at studying the effects of the applied stress on the temperature sensitivity in terms of permanent deformation, which is essential to the understanding of the behaviour of FBM and to increase confidence in pavement design using these materials.

7.2.1 Repeated Load Axial Test (RLAT)

The Repeated Load Axial Test is the most widely used standard mechanical test used in the United Kingdom for assessing the permanent deformation characteristics of bituminous mixtures. The test applies a repeated pulse load to simulate the traffic and measures the permanent deformation for each repeated load. The protocol for the RLAT test can be found in BS DD 185: 1994. In this project the tests were carried in the Nottingham Asphalt Tester (NAT). Figure 7-1 shows the configuration of the RLAT. Specimens of 63.5 mm height and diameter of 100 mm after compaction were cured in a forced draft oven for a further 72 h at 40°C. Then the two faces of the specimen were coated with a thin layer of silicone grease with graphite powder before testing in the NAT. Graphite powder is applied in order to help eliminate inward stresses on the specimen due to friction between specimen and platen that might act to constrain specimen deformation.

In the test, the specimen was positioned vertically between the upper and lower steel loading platens, which are slightly wider than the specimen. The repeated load was applied axially, while the vertical deformation of the specimen was measured by two LVDTs mounted on the upper loading platen as shown in Figure 7-1. The repeated load consists of a square wave form with a frequency of 0.5 Hz, i.e. a pulse of one second duration followed by a rest period of one second duration. This simulates the slow moving traffic that leads to the most deformation in a real road. The input parameters for testing are temperature, specimen thickness and diameter, stress and number of load pulses including those for the conditioning stage.

The standard RLAT test uses a vertical stress of 100 kPa at a temperature of 30°C for 1800 pulses. However, as the aim of the study is to study the effect of the temperature and stress level on the permanent deformation, varying temperatures (30°C, 40°C and 50°C) and stress levels (100kPa, 200kPa and 300kPa) were used, resembling in-situ stresses in pavement bases. Before the specimen was tested by applying the required axial stress, the specimen was pre-loaded with a conditioning load equivalent to a stress of 10 kPa for 600 seconds. The test results are the axial strains under 1800 load cycles.

For each observation level, 3 specimens were tested and the quoted strains are averages of the 3 specimens unless otherwise stated. The results of the test were tabulated in Table 7-1 and graphically plotted in Figure 7-2 through Figure 7-5 to make a comparison between different mixtures and to study the influence of test temperature and stress level on the permanent deformation behaviour. Table 7-1 presents the accumulated permanent strain (ε_p) of the test specimen after 1800 load repetitions along with the creep strain curve intercept and slope (CSS). The intercept of the dynamic creep curve represents the initial strain which is the deformation due to densification of the mixture. This initial deformation depends mainly on the method of compaction and therefore it is unfair to use only total axial strain for evaluation of the permanent deformation behaviour of the mixture. Therefore the slope of the dynamic creep strain curve (CSS) was also considered as a parameter to evaluate the deformation behaviour. The parameters CSS and the intercept of the creep curve are also included in Table 7-1. As can be seen from the table the strain data for mixtures tested at 50°C with 300kPa repeated load was not available as the specimens failed well before reaching 1800 cycles.

Figure 7-1 RLAT test configuration within NAT apparatus

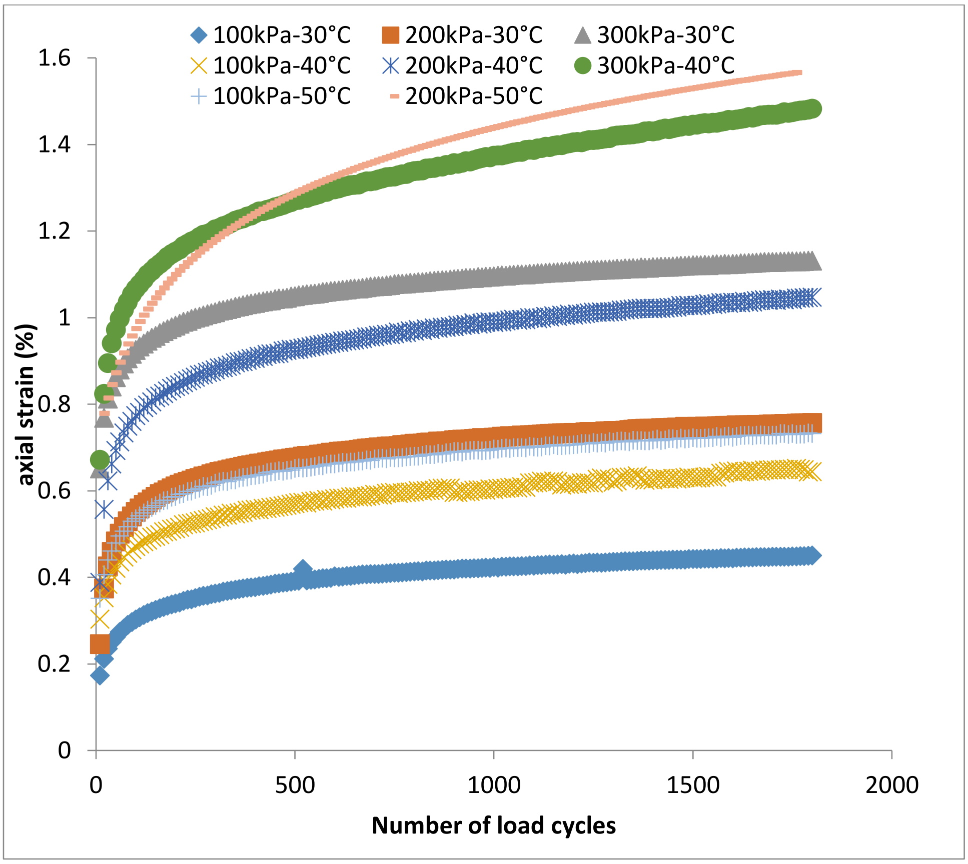


Figure 7-2 Influence of test temperature and stress level on permanent deformation behaviour of 50%RAP-FBM

Table 7-1 Summary of RLAT test results

Stress (kPa)	Test temperature (°C)	Mixture type	Total strain (%)	CSS	Intercept
100	30	50%RAP - FBM	0.48	0.051	0.065
100	30	50%RAP+1% Cement - FBM	0.41	0.044	0.079
100	30	75%RAP+1% Cement - FBM	0.39	0.041	0.083
100	30	DBM 90	0.49	0.042	0.175
200	30	50%RAP - FBM	0.75	0.077	0.173
200	30	50%RAP+1% Cement - FBM	0.61	0.059	0.169
200	30	75%RAP+1% Cement - FBM	0.64	0.053	0.243
200	30	DBM 90	0.56	0.051	0.178
300	30	50%RAP - FBM	1.13	0.076	0.56
300	30	50%RAP+1% Cement - FBM	0.77	0.070	0.245
300	30	75%RAP+1% Cement - FBM	0.89	0.064	0.41
300	30	DBM 90	0.64	0.066	0.152
100	40	50%RAP - FBM	0.64	0.062	0.175
100	40	50%RAP+1% Cement - FBM	0.61	0.055	0.198
100	40	75%RAP+1% Cement - FBM	0.6	0.058	0.165
100	40	DBM 90	0.58	0.052	0.19
200	40	50%RAP - FBM	1.04	0.101	0.283
200	40	50%RAP+1% Cement - FBM	0.81	0.068	0.3
200	40	75%RAP+1% Cement - FBM	0.84	0.065	0.353
200	40	DBM 90	0.92	0.069	0.403
300	40	50%RAP - FBM	1.48	0.143	0.408
300	40	50%RAP+1% Cement - FBM	0.99	0.088	0.33
300	40	75%RAP+1% Cement - FBM	1.06	0.091	0.378
300	40	DBM 90	1.42	0.107	0.618
100	50	50%RAP - FBM	0.73	0.069	0.213

100	50	50%RAP+1% Cement - FBM	0.74	0.061	0.283
100	50	75%RAP+1% Cement - FBM	0.79	0.063	0.318
100	50	DBM 90	0.87	0.071	0.338
200	50	50%RAP - FBM	1.57	0.139	0.532
200	50	50%RAP+1% Cement - FBM	1.51	0.093	0.813
200	50	75%RAP+1% Cement - FBM	1.66	0.095	0.948
200	50	DBM 90	1.66	0.109	0.843

7.2.2 Effect of test temperature and stress level on permanent deformation behaviour

Figure 7-2 presents the dynamic creep curves of the 50% RAP-FBM mixture. As the figure demonstrates, both test temperature and stress level have significant influence on the deformation behaviour of FBM. The ratio of total strain after 1800 load cycles at 50°C to the same at 30°C was found to be 1.52 and 2.09 respectively for stress levels of 100kPa and 200kPa. A similar trend can also be found with the other mixtures considered in the study indicating that the effect of stress on the deformation increased with an increase in test temperature. Figure 7-3 to Figure 7-5 present the total permanent strains after 1800 load cycles for the mixtures considered in the study. The results reiterate as discussed that both test temperature and stress level have significant influence on the deformation behaviour of FBM and HMA (DBM 90). It is important to note that though addition of the cement to FBMs considerably improved the resistance to permanent deformation, both of the FBMs with cement (50%RAP+1% Cement-FBM and 75%RAP+1%Cement-FBM) were found to have nearly the same permanent strain values except for test results at 50°C. This shows that the RAP content in the mixture has limited influence on the permanent deformation resistance, unlike stiffness as seen in Chapter 5. This is in consistent with other works on similar mixes [85]. Figure 7-3 to Figure 7-5 also suggest that at 50°C there was no evident decrease in total strain for mixtures with cement when compared with 50%RAP.

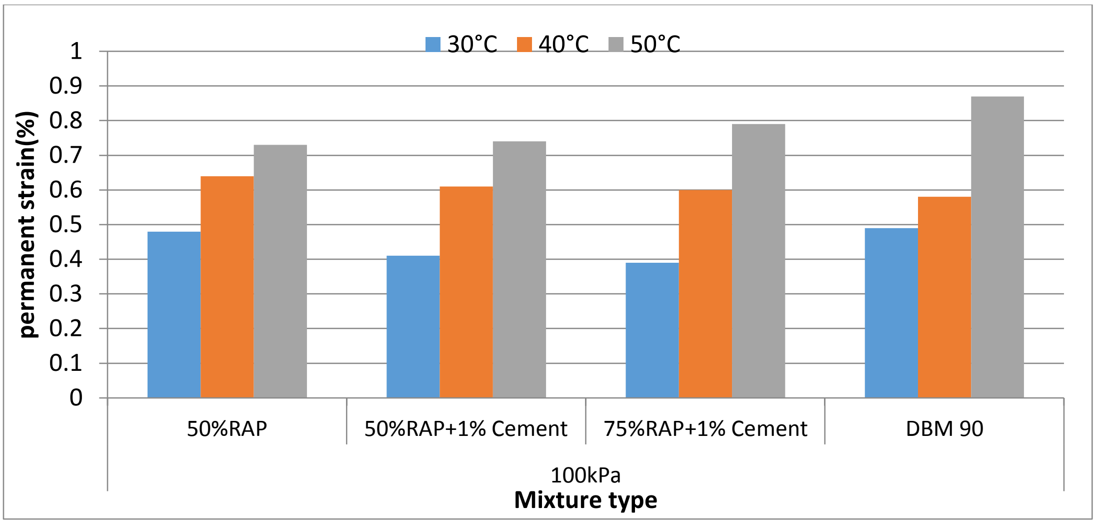


Figure 7-3 Influence of test temperature on permanent strain in different mixtures at 100kPa stress level

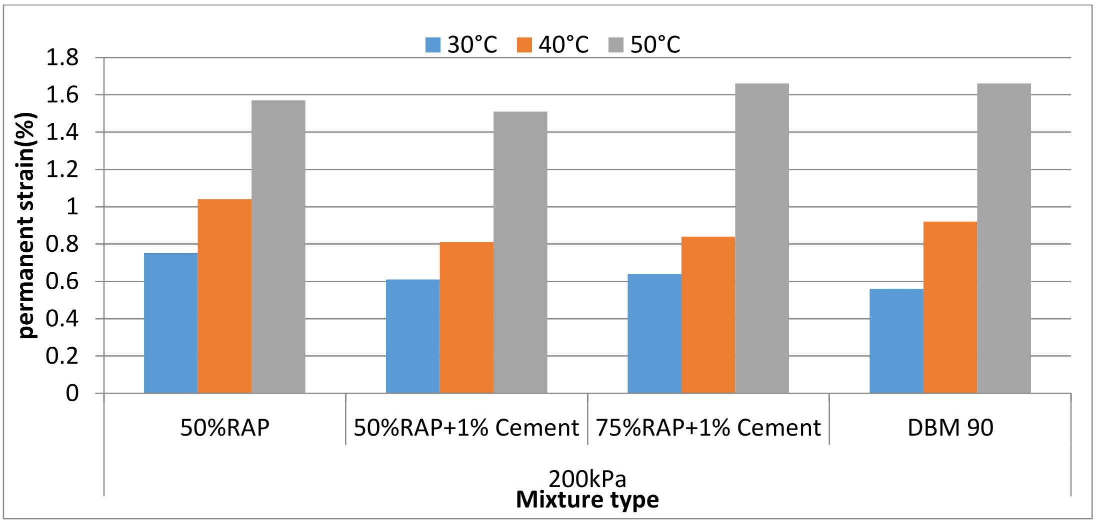


Figure 7-4 Influence of test temperature on permanent strain different mixtures at 200kPa stress level

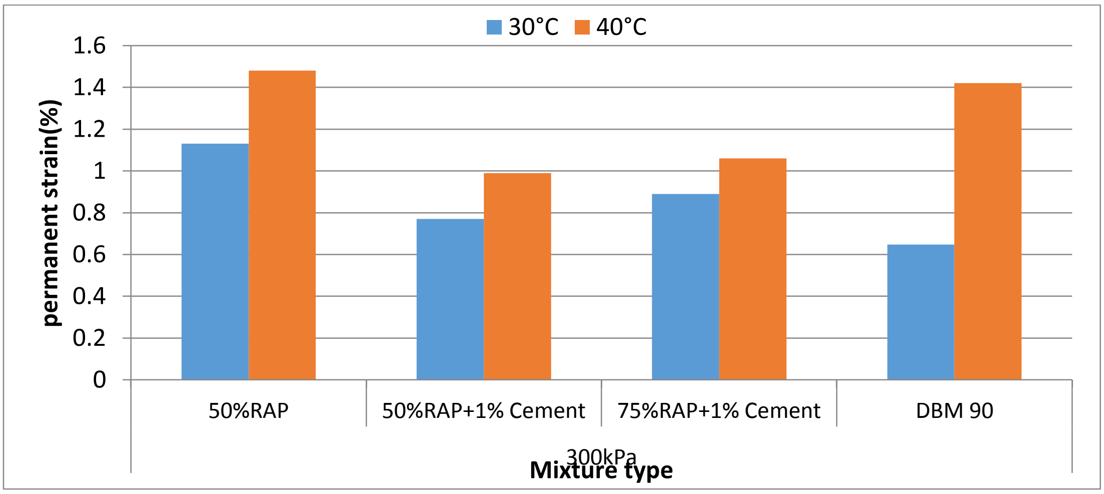


Figure 7-5 Influence of test temperature on permanent stain different mixtures at 300kPa stress level

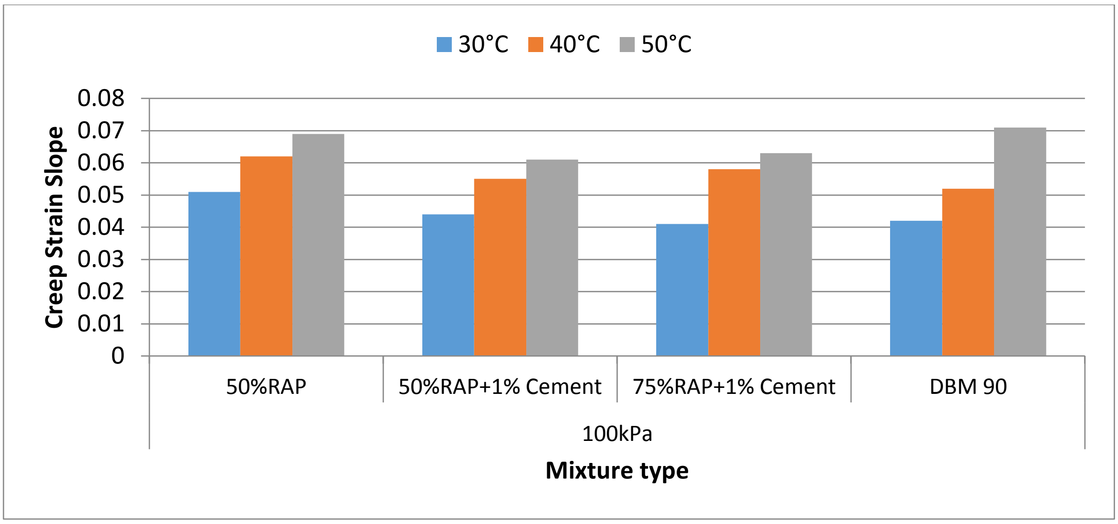


Figure 7-6 Influence of test temperature on creep strain slope of different mixtures at 100kPa stress level

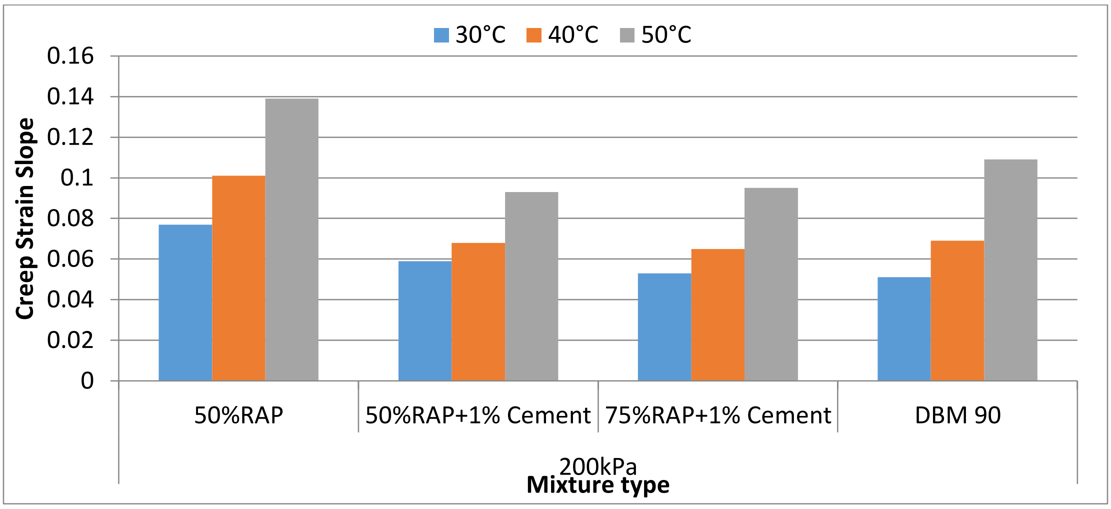


Figure 7-7 Influence of test temperature on creep strain slope of different mixtures at 200kPa stress level

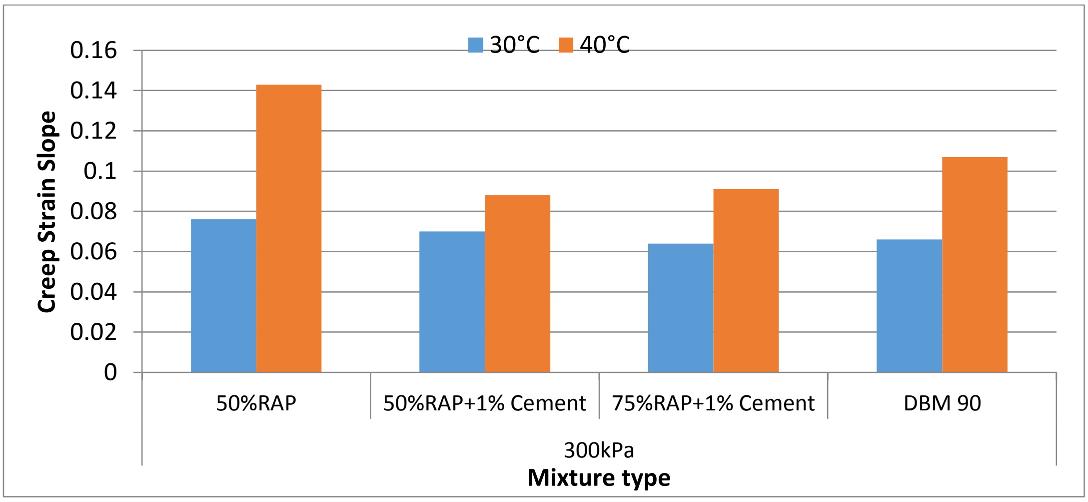


Figure 7-8 Influence of test temperature on creep strain slope of different mixtures at 300kPa stress level

Figure 7-3 to Figure 7-5 also suggest that HMA (DBM 90) is more sensitive to temperature than the FBMs. For example if we compare the test results at 200kPa stress level, the total strain of DBM 90 mixture at 50°C was found to be 2.96 times that of the total strain at 30°C. The corresponding values for FBM without cement (50%RAP-FBM) and with cement (50%RAP+1%Cement-FBM and 75%RAP+1%Cement-FBM) were 2.09 and 2.47 & 2.59 respectively. A similar trend was also found for other stress levels. Therefore it can be said that FBM is less temperature sensitive in terms of deformation behaviour when compared with conventional HMA. In addition to that, among FBMs the mixture without cement was found to be less temperature susceptible, primarily because of the insignificant effect of cement on deformation behaviour at the higher temperature of 50°C. In general, FBM with cement performed (in terms of permanent deformation) equal to DBM 90 while FBM without cement performed worse than DBM 90.

The above discussion is valid when the total permanent strain is taken as the criterion. However, if CSS is taken as the comparison criterion, slightly different conclusions can be drawn. As can be seen from Figure 7-6 to Figure 7-8, while the CSS for DBM 90 was found to be less than 50%RAP-FBM at 200kPa, it was almost the same at 100kPa indicating the high strain levels for 50%RAP-FBM at 200kPa was mostly because of the initial deformation. Nevertheless the ratio of CSS at 50°C to 30°C of DBM 90 remains higher (1.7 at 100kPa and 2.13 at 200kPa) than that of FBMs (1.35-1.51 at 100kPa and 1.57-1.8 at 200kPa) indicating the higher temperature sensitivity of HMA when compared with FBMs.

7.3 Fatigue characteristics

Fatigue failure is one of the main failure modes of a pavement structure which results in degradation of the pavement materials and finally the pavement structure. FBMs are usually considered inferior to HMA in terms of performance because of their highly porous nature and lower early strength because of the presence of water [134]. Though addition of cement is found to resolve issues such as low early strength and improve the permanent deformation behaviour as seen in the previous section, a very limited number of studies have focused on the effect of cement on the fatigue properties of these mixes. As discussed in Chapter 2, there have been a number of tests to predict the fatigue performance of bituminous mixes in the laboratory. However, for assessing the fatigue behaviour of CBMs, the Indirect Tensile Fatigue Test (ITFT) [134-136]and 4 point bending (4PB) test [76, 137] have mostly been employed. Nevertheless, the fatigue behaviour of these mixes is not fully understood. Some previous findings based on ITFT results for CBMs suggest that these mixtures have inferior fatigue characteristics when compared with HMA [134]. Moreover in the same study Thanaya (2007) reported that though addition of 2% cement significantly improved the permanent deformation resistance of the CBM it reduced the fatigue life when compared to HMA. In the view of the above discussion the present study has tried to validate the fatigue behaviour using the ITFT, with FBMs containing 1% and 3% of cement, and to compare the fatigue life with that of HMA.

As the ITFT is a simple and inexpensive test method and has the advantage of being able to test cylindrical specimens cored from in-service pavements it has been widely used ahead of other fatigue tests. However, the ITFT suffers from drawbacks such as the absence of stress reversal and the accumulation of permanent deformation[138]. These shortcomings of the ITFT test have been well documented in the literature[138]. In spite of these shortcomings the ITFT has been considered as a routine practical method for evaluating the fatigue life of bituminous paving mixtures. This is particularly true for CBMs as it has been found difficult to manufacture beams for beam fatigue tests [76, 97, 139]. Though in the present study FBM beams were made without aggregate break out from the beams during saw cutting, the variability of test results was found to be very high using the 4 point beam test. A similar outcome has also been mentioned in other works on beam fatigue teats [76, 139]. Moreover past research findings suggest that the ITFT is too conservative a test for evaluating the fatigue life of CBMs because of their high void content [137, 140]. In view of the above observations, alongside the ITFT the present study has also evaluated fatigue behaviour in terms of a uniaxial test in which compressive load is applied in the axial direction and indirect tensile strain was monitored in the radial direction.

7.3.1 Diametral Indirect Tensile Fatigue Test

In diametral indirect tensile test mode (both ITFT and ITSM), a compressive load is applied across the vertical diameter of a cylindrical specimen and this results in a biaxial stress distribution in the specimen as shown in Figure 7-9. As found in the figure the magnitude of both vertical compressive stress (σ_{vx}) and horizontal tensile stress (σ_{hx}) induced on the horizontal diameter of the specimen varies along the diameter. Both σ_{vx} and σ_{hx} reach a maximum value at the centre of the specimen. By measuring the horizontal deformation, the values of both the maximum strain and the stiffness modulus of the specimen can be calculated. However, these calculations are based on following assumptions [90].

- 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

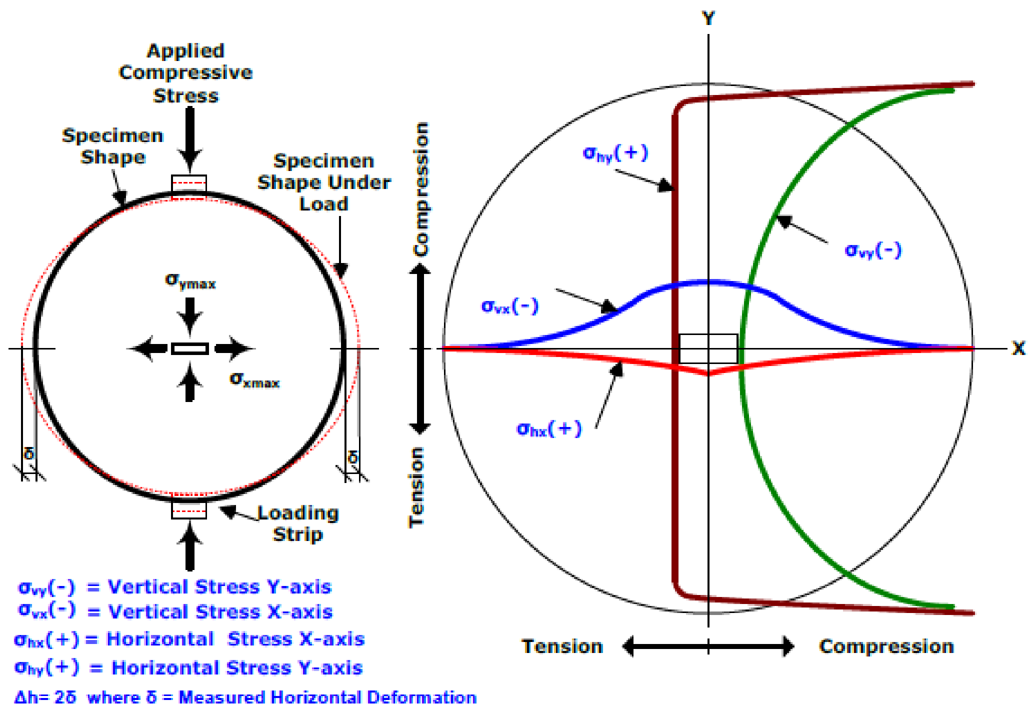


Figure 7-9 Stress distribution along diameter of the specimen in Indirect Tensile Test mode[3]

7.3.1.1 ITFT set up

The Diametral Indirect Tensile Fatigue Test (ITFT) was conducted using the Nottingham Asphalt Tester (NAT), performed in accordance with BS DD ABF: 2000 and BS DD ABF: 2003. The test was carried out at a standard temperature of 20±1°C using 100±3 mm diameter specimens. Though a thickness of 40±2 mm is recommended in the standards, in the present study 60±2 mm thick samples were used as it was found difficult to trim the thicker samples to 40 mm because the trimming compromises the integrity of the sample. The test was performed at various stress levels (initial strain levels) at a rate of 40 pulses/minute. Each specimen was repeatedly loaded until it failed by cracking or a total vertical deformation of 9 mm had been achieved (as larger permanent deformation would likely compromise reliable measurement of the fatigue behaviour). The target test stress levels were selected to give as wide a range of lives as possible.

Configuration of the test frame and specimen position in ITFT mode is similar to that in ITSM mode, but without the mounting frame and the alignment jig for LVDT instalment (Figure 7-10). The cylindrical specimen is positioned in between the loading platens. Prior to the ITFT test, an ITSM test in stress controlled mode was conducted on specimens at the required test stress level for the ITFT to determine the horizontal deformation and corresponding ITSM values. Once these tests were done, Eq. 7-1 and Eq. 7-2 were used to determine the maximum stress and maximum strain at the centre of the specimen as recommended by the standard. The initial maximum tensile strain at the centre of the specimen for each respective stress level as determined for each specimen using Eq. 7-2 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 to obtain the fatigue relation in the form of Eq. 7-3.

$$\sigma_{x,max} = \frac{2*P_L}{\pi*d*t}$$ Equation 7-1

$\sigma_{x,max}$ is the maximum tensile stress at the centre of the specimen; P_L is the applied vertical load in kN; d is the diameter of the specimen in m; t is the thickness of the specimen in m.

$$\varepsilon_{x,max} = \frac{\sigma_{x,max}}{S_m} * (1 + 3\upsilon) * 1000$$ Equation 7-2

$\varepsilon_{x,max}$ is the maximum tensile strain at the centre of the specimen; υ is Poisson's ratio which was assumed as 0.35; S_m is the indirect tensile stiffness modulus at $\sigma_{x,max}$ in MPa

$$N_f = C * (\frac{1}{\varepsilon_{x,max}})^m$$ Equation 7-3

N_f is number of cycles to failure; C is antilog of the intercept of the logarithmic relationship between fatigue life and tensile stress and m is slope of the logarithmic relationship between fatigue life and tensile stress.

Figure 7-10 ITFT test set up in NAT machine

7.3.1.2 ITFT results

The results of the ITSM tests over the range of stress levels which are used in calculating the maximum tensile strain ($\varepsilon_{x,max}$) at the centre of the specimen using Eq. 7-2 are tabulated in Table 7-2. As can be seen from Table 7-2, the stress levels selected for FBM testing are less than the stress levels recommended by DD ABF 1995 as the recommended stress levels of 500kPa were found to be too harsh on the FBM specimens since the test lasted for less than 20 cycles. Therefore the target test levels were selected in such a way as to give as wide a range of lives as possible. It has to be noted that the stiffness decreased with increasing stress level as can be seen in Figure 7-11 in which the stiffness trend (Linear fit) was plotted with respect to the stress levels considered for each mixture. In general at all stress levels the stiffness of the mixture with 3% of cement was significantly higher than all other mixtures while as expected the stiffness of the FBM without any added cement (50%RAP-FBM) was found to have the least stiffness.

The calculated $\varepsilon_{x,max}$ values for each mixture considered in the study were plotted against the number of repeated load cycles to failure (N_f). The resulting plot can be found in Figure 7-12. Linear regression was carried out on the data points obtained for each mixture to fit the model in Eq. 7-3. The corresponding coefficients obtained after the fitting are tabulated in Table 7-3. R^2 values from Table 7-3 suggest that the data collected for each mixture provided a good fit with the fatigue equation (Eq. 7-3) and the R^2 values satisfied the BS DD ABF: 2003 requirement of 0.9 except for 75%RAP+1%Cement-FBM for which the R^2 value was found to be marginally less than the standard requirement.

Table 7-2 ITSM test results to be used in calculating maximum tensile strain

Target stress levels range and corresponding stiffness range		
Mixture type	stress range (kPa)	stiffness range (MPa)
50% RAP - FBM	100 - 300	3062 - 772
50% RAP+1% Cement - FBM	150 - 350	3154 - 990
75% RAP+1% Cement - FBM	150 - 350	2914 - 1062
50% RAP+3% Cement - FBM	400 - 750	8412 - 4995
DBM 90	200 - 550	4980 - 3415

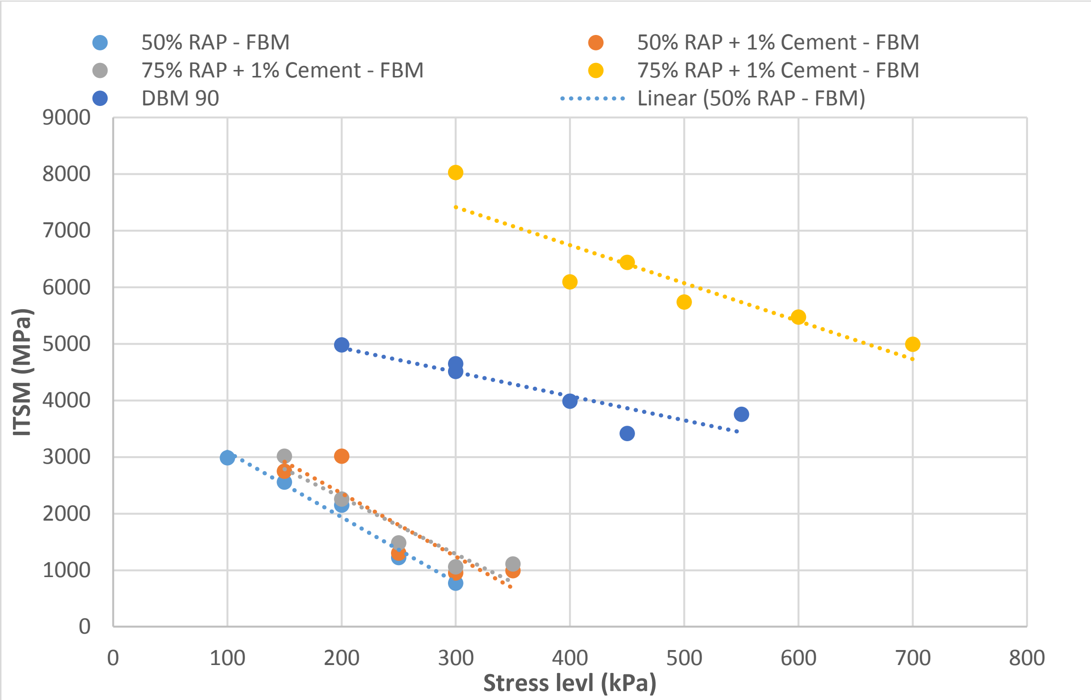


Figure 7-11 Effect of stress level on stiffness of the mixture

Table 7-3 Regression coefficients for fatigue equation from ITFT test

Mixture type	C	m	R^2	strain at 1 million cycles (με)	number of cycles at 100με
50% RAP - FBM	2E+11	3.411	0.97	35.25	30132
50% RAP+1% Cement - FBM	1E+12	3.425	0.94	66.15	141254
75% RAP+1% Cement - FBM	7E+11	3.321	0.84	74.44	159624
50% RAP+3% Cement - FBM	2E+14	4.625	0.96	48.07	112468
DBM 90	3E+14	5.078	0.98	59.47	20947

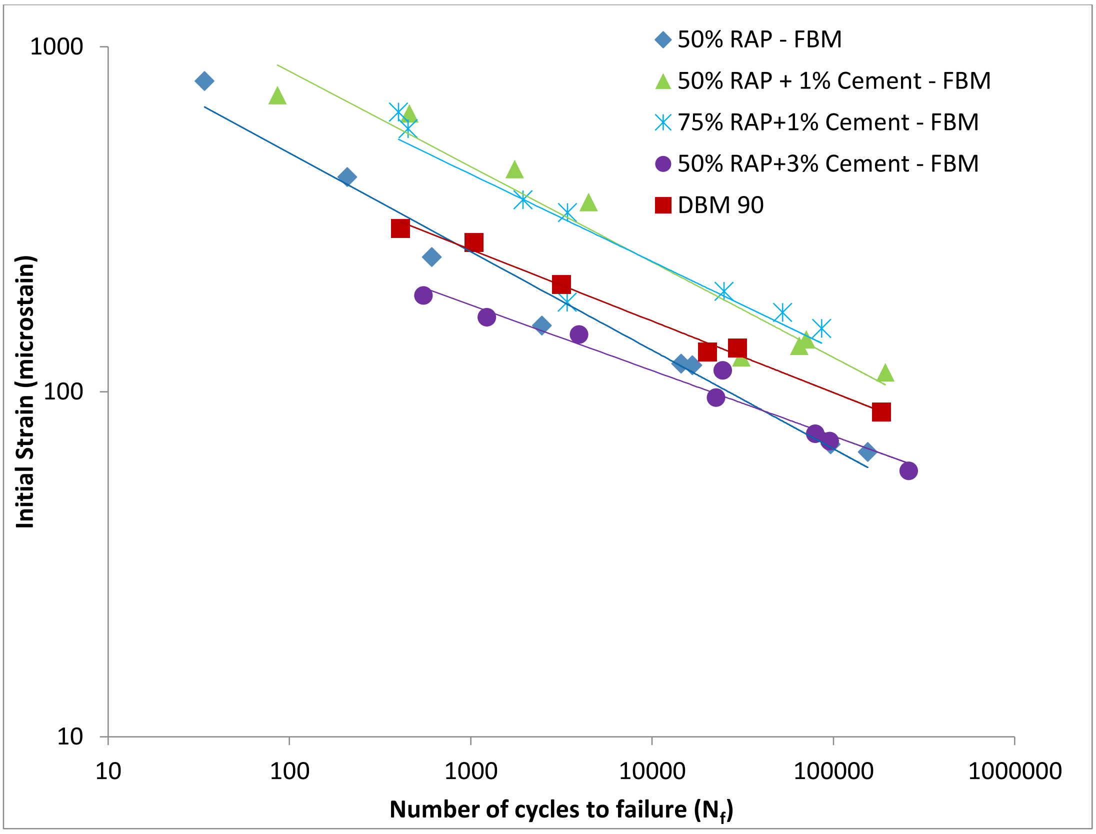


Figure 7-12 Fatigue performance of FBM compared with HMA (ITFT)

7.3.1.3 Effect of cement

Referring to Table 7-3 and Figure 7-12, it can be clearly seen that the fatigue lines of the both FBMs with 1% added cement (50%RAP+1%Cement-FBM and 75%RAP+1%Cement-FBM) performed better than the FBM without cement (50%RAP-FBM). At a projected life to failure (N_f) of one million load cycles, 50%RAP+1%Cement-FBM and 75%RAP+1%Cement can withstand 66 and 74 micro-strain respectively while the 50%RAP-FBM can only withstand 35 micro-strain. Similarly, the number of cycles to failure (N_f) for 50% RAP-FBM at a strain value of 100 micro-strains (30000 cycles) were also much lower than that of the FBM with cement (141000 and 160000). This indicates that incorporating 1% cement significantly improves the fatigue performance of FBMs, indirect tensile mode. However, the FBM mixture with 3% added cement performed worse than all other mixtures. This could be attributed to the loss in flexibility of the mixture due to additional of extra cement. It is worth noting that for both the mixtures with 1% cement, although these have different RAP contents, the fatigue performance is nearly the same. The above discussion indicates that the cement content and its presence in the mixture had greater influence than the amount of the RAP present in the mixture. The insignificant difference in fatigue life was analogous to the findings for permanent deformation performance discussed in the previous section.

7.3.1.4 Comparison with HMA

Compared with HMA, FBM usually has higher air voids and less cohesive characteristics, and therefore it is expected that the performance of these mixtures is inferior to HMA. The addition of cement has been found to improve the cohesion of these mixtures, although it has insignificant influence on volumetric properties as seen in Chapter 5. This expected inferior fatigue performance can be identified in Figure 7-12 in which DBM 90 performance was found to be better at least for low initial strain levels, i.e. these in the range that pavements usually experience. However, Figure 7-12 suggests that 1% cement addition significantly improved the fatigue performance of these mixtures. Nevertheless as stated earlier, 3% cement addition caused significant flexibility loss to these mixtures resulting in very low fatigue life.

7.3.2 Axial Indirect Tensile Fatigue Test

The major reason for the wide acceptance of the ITFT over other tests is that it is simple and less expensive and most importantly that cores from in-service pavements can be tested. A major assumption in the ITFT as discussed is linear elasticity of the materials. As FBMs are found to behave non-linearly [7] even at small stress/strain levels the stress distribution is more complex [141]. This brings into question the use of ITFT as a performance indicator. Moreover, as discussed beams and trapezoidal shaped specimens for flexural testing cannot be easily obtained because of the reduced cohesive capability of FBMs. In view of these limitations of ITFT applicability to FBMs, in this study the applicability of uniaxial compression test was studied. This test was chosen as it has least complex stress conditions compared to other tests such as ITFT and flexural beam tests [142]. Moreover, the test specimen can be prepared from a core taken from an in-service pavement for validating the in-service condition of a pavement layer.

The test comprises application of a sinusoidal load on a 100mm by 100mm cylindrical specimen in the axial direction at a frequency of 1 Hz. If a very low friction system is provided between loading plates and specimen, for a uniaxial stress condition the principal failure plane exists parallel to the stress direction and principal strain is perpendicular to the failure plane. Therefore the testing involves application of high stress levels so as to induce high radial strains to make sure the test lasts for a reasonable length of time.

7.3.2.1 Test set up

The test apparatus used for applying the sinusoidal load consists of an INSTRON loading frame, a 100kN servo-hydraulic actuator with ±50 mm movement, an axially mounted load cell and temperature controlled cabinet mounted on the loading frame. RUBICON software, which is a digital servo control system, was used for operating the loading frame and for data acquisition.

The sinusoidal load of 1Hz was applied on the specimen through two loading plates (top and bottom). During the test the axial and radial deformations were measured by axial and radial LVDTs mounted on the specimen. The LVDTs for axial deformation measurement were placed on the top plate (Figure 7-13).These vertical LVDTs were fixed to magnetic supporting arms. The axial deformation was then the average of the two axial LVDT measurements. The radial deformation of the specimen during the test was measured with an LVDT fixed to a collar which was mounted at mid height of the specimen. The range of the horizontal LVDT was ±10 mm and it was calibrated using the set-up shown in Figure **7**-**14** to directly measure the radial deformation of the specimen.

Since in the present study the aim was to induce radial tensile strain along the height of the specimen, it was important to have uniform radial deformation throughout the height of the specimen. To ensure this, a friction reducing system was used in between the specimen and loading plate. The system consists of a 50 µm polyethylene foil in combination with a thin layer of silicon compound, a grease-like material made by DOW CORNING®, both between the foil and the specimen. The major problem experienced because of the friction reducing system was sliding of the specimen during the test which could cause damage to the instrumentation, particularly to the collar for horizontal deformation measurement. Therefore care had to be taken while testing to make sure that the system did not slide.

After setting up the test, as explained above, the stress controlled test (sinusoidal load at 1Hz frequency) was carried out. The test stress levels were selected in such a way that to give as wide a range of lives as possible (Table 7-4). Table 7-4 shows the range of stresses applied to the specimen and resulting initial horizontal (radial) strain values. Initial strain values were average values of radial recoverable (peak to peak) strains over the first 10 cycles of the test. Before the start of the test a compressive holding load (pre-load) of 10N was applied to the specimen to make sure it was held stably during the test. Following the pre-loading the sinusoidal target load was applied to the specimen. Visual inspection of the specimen during the test revealed that above a radial deformation of 5 mm (approximately 5% radial strain) the specimen had wide vertical cracks (Figure 7-15) and the axial strain increased at a rapid rate which could damage the instrumentation. Therefore the test was stopped once the total horizontal deformation reached 5mm.

Figure 7-13 Test set up for uniaxial indirect tension test

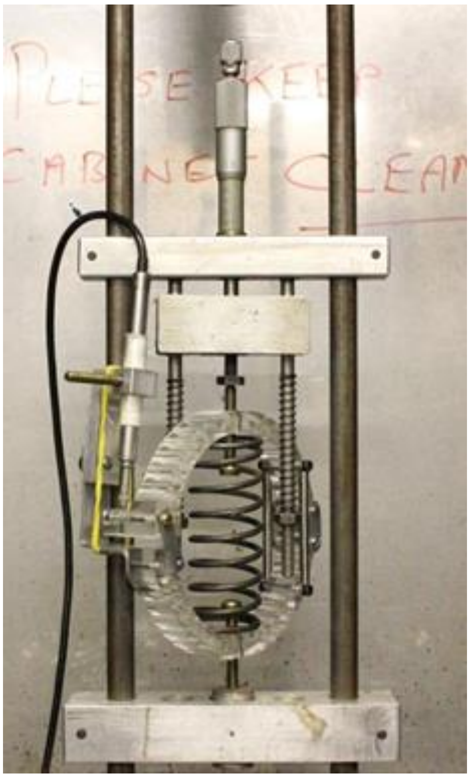


Figure 7-14 **The set-up for calibration of radial LVDT**

Table 7-4 Range of applied stress and resulting initial radial strains for fatigue life plots

Target stress levels range and corresponding initial strain range		
Mixture type	stress range (kPa)	initial strain range(με)
50% RAP - FBM	600-2100	78-421
50% RAP+1% Cement - FBM	800-2400	96-384
75% RAP+1% Cement - FBM	800-2400	78-381
DBM 90	1000-3000	86-550

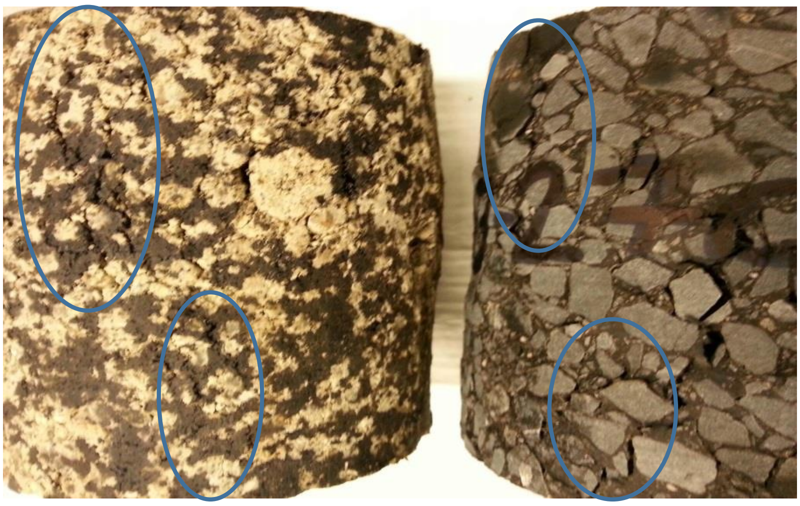


Figure 7-15 FBM and DBM 90 samples after testing

7.3.2.2 Failure analysis

The major advantage of this test over the ITFT is that the failure characteristics (stiffness, permanent deformation etc.) can be monitored during the test. In this study the failure criterion used for this test under controlled stress condition was a reduction of the initial stiffness of 50% (50% retained stiffness) which was assumed to coincide with complete failure. Figure 7-16 shows the fatigue lines plotted as initial strain against number of cycles to reach 50% stiffness. Linear regression was carried out on the data points obtained for each mixture to fit the model in Eq. 7-3. The corresponding coefficients obtained after the fitting are tabulated in Table 7-5. R^2 values from Table 7-5 suggest that the data collected for each mixture fitted the fatigue equation very well (Eq. 7-3). Figure 7-16 demonstrates that there is no clear difference in fatigue life among the mixtures considered in the study. This is particularly true at lower initial strain levels.

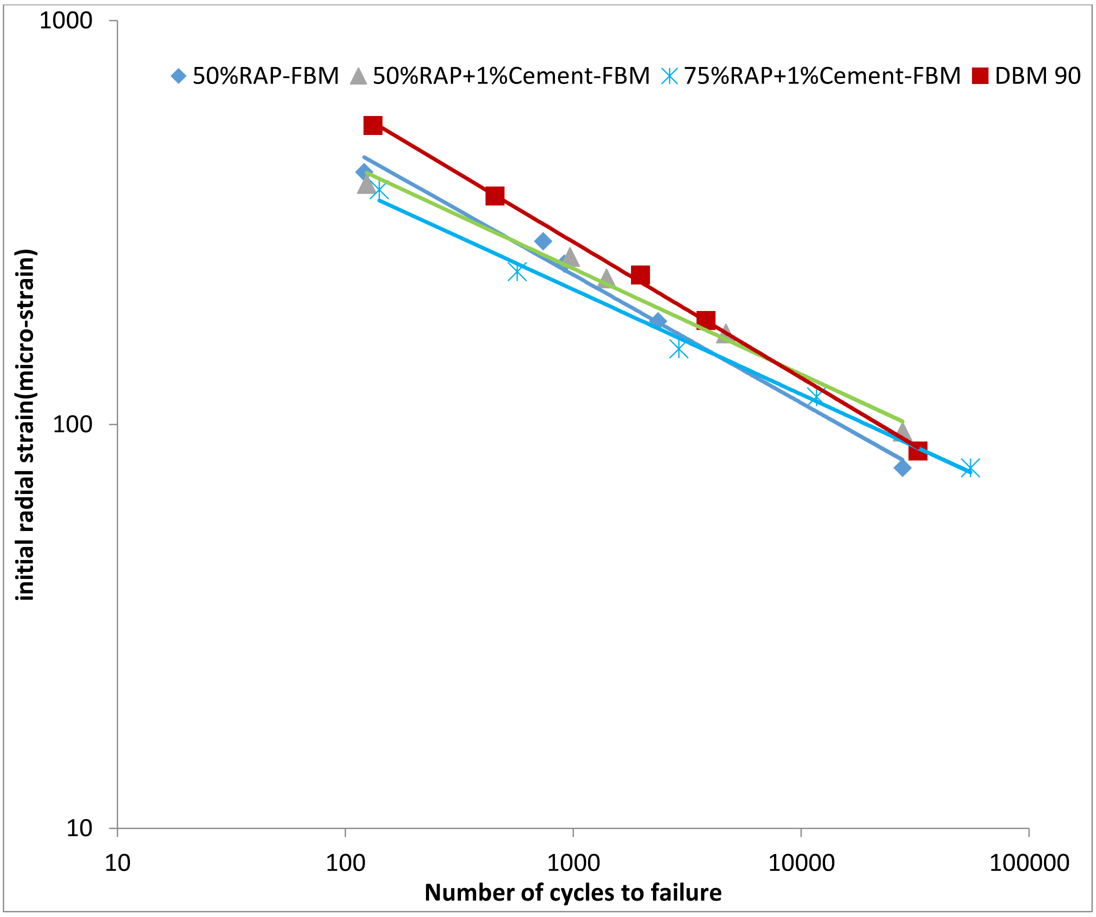


Figure 7-16 Uniaxial indirect fatigue test results obtained for different mixtures

Table 7-5 Regression coefficients for fatigue equations from uniaxial test

Mixture type	C	m	R^2
50% RAP - FBM	2.00E+10	3.114	0.98
50% RAP+1% Cement - FBM	1.00E+12	3.774	0.98
75% RAP+1% Cement - FBM	9.00E+11	3.825	0.97
DBM 90	2.00E+10	2.968	0.99

7.3.2.3 Stiffness evolution

Figure 7-17 shows the first four load cycles of a test on 50%RAP-FBM. As can be seen from the figure the applied stress curve and resulting strain curve were at 1Hz frequency. The ratio of peak stress and peak strain is termed as stiffness (S) in this discussion. The stiffness for the entire test duration was calculated accordingly and is plotted in Figure 7-18 to Figure 7-21. The initial strain level was defined as the average of the first 10 radial strain cycles, corresponding to the initial strain in the ITFT test, in which the initial maximum strain is at the centre of the specimen. Though tests were carried out at five radial strain levels, for brevity stiffness evolution curves for the three middle strain levels are presented in the figures. As expected the initial radial strain level significantly affected the stiffness reduction rate. The higher the initial strain level the shorter the time to reach 50% stiffness. In order to analyse the stiffness evolution independent of the initial strain level the stiffness data was therefore normalised, as seen in Figure 7-22 to Figure 7-25. For each data point on the specific strain level curve the relative time was obtained by dividing the actual number of cycles corresponding to that data point by the number of cycles corresponding to 50% stiffness. As can be seen from Figure 7-22 all normalised stiffness curves for 50%RAP-FBM followed approximately the same path irrespective of initial radial strain level. A similar observation is also valid for the other mixtures with the slight exception of the 550με curve of DBM 90.

In order to use these trends in pavement analysis and design the normalised stiffness curves of all the mixtures considered in the study were averaged and best-fit polynomial curves were fitted as depicted in Figure 7-26. It is clear from the figure that the stiffness evolution of FBM and HMA is very different. The DBM 90 tends to have a three stage evolution process during the test, similar to 4 point bending tests [138, 143]. After a rapid reduction of stiffness (phase I), due to the internal heating phenomenon, stiffness decrease becomes more linear (phase II). Fracture occurs in the final stage (phase III) and it is characterised by an acceleration of stiffness drop. However, this three stage stiffness evolution was not identified in the FBMs that were considered in the study. The stiffness increased rapidly to reach a maximum and then decreased at a slower rate. The maximum for all the mixtures was identified as between 0.1 and 0.3 of the relative time to reach 50% of stiffness. This increase in stiffness could be associated with densification of the mixtures during the test in which high stress levels of 800 to 2400kPa were applied. It is a sensible assumption as the FBMs have high air void content in the range of 14-18% depending on the mixture type, whereas the average air voids of DBM 90 was found to be 5.3%. To verify this assumption volumetric strain was calculated from axial strain and radial strain measured during the test. The volumetric strain of 50%RAP-FBM specimen that was tested at an initial radial strain level of 300με is plotted against relative time for 50% stiffness in Figure 7-27. As can be seen from the figure, the volumetric strain decreased below zero indicating densification during the initial stage of test, followed by strain increase.

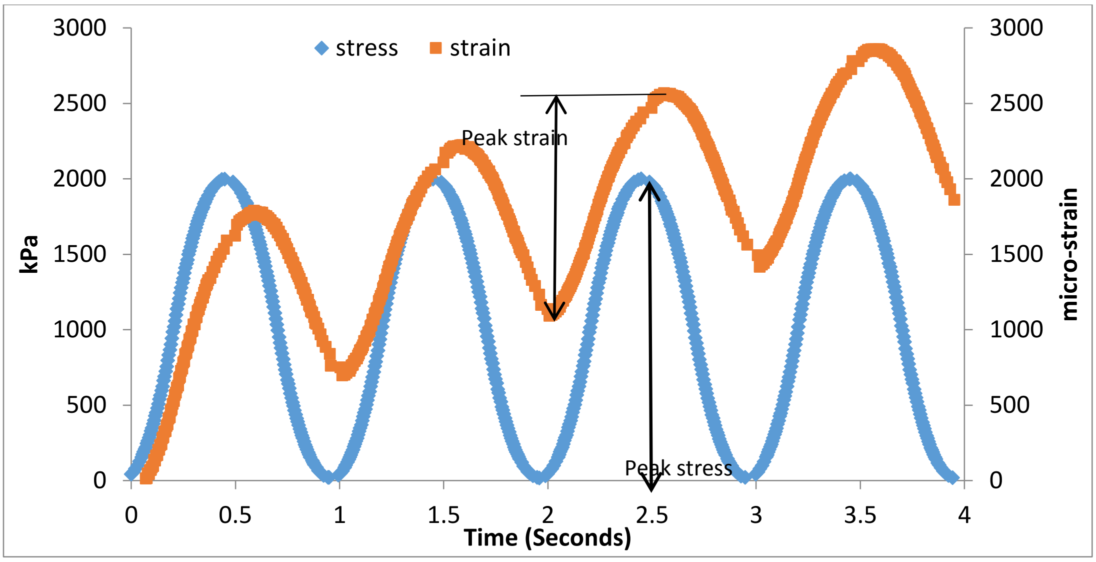


Figure 7-17 Applied stress and resulting axial strain of 50%RAP-FBM

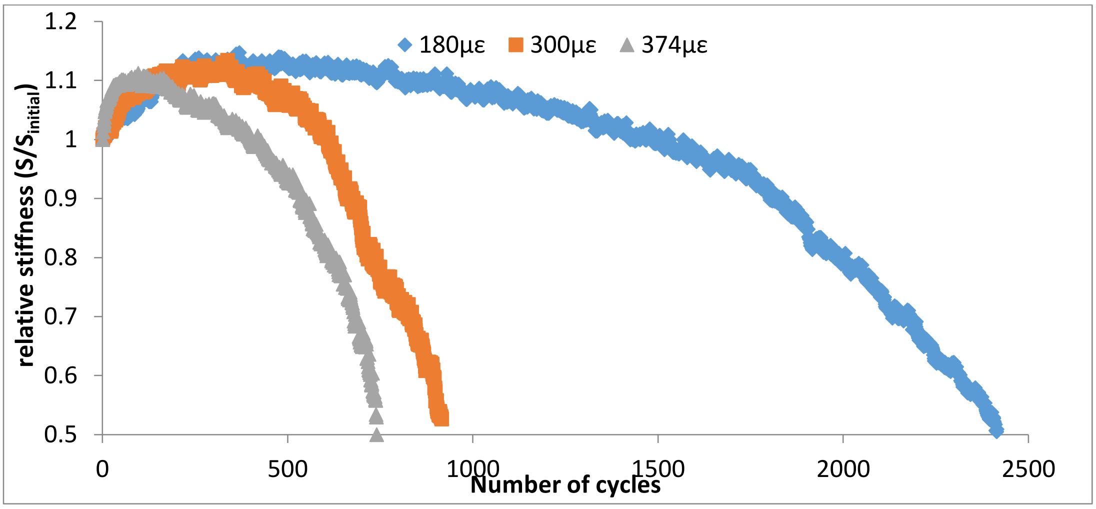


Figure 7-18 Stiffness evolution curve for 50%RAP-FBM at different initial radial strain levels

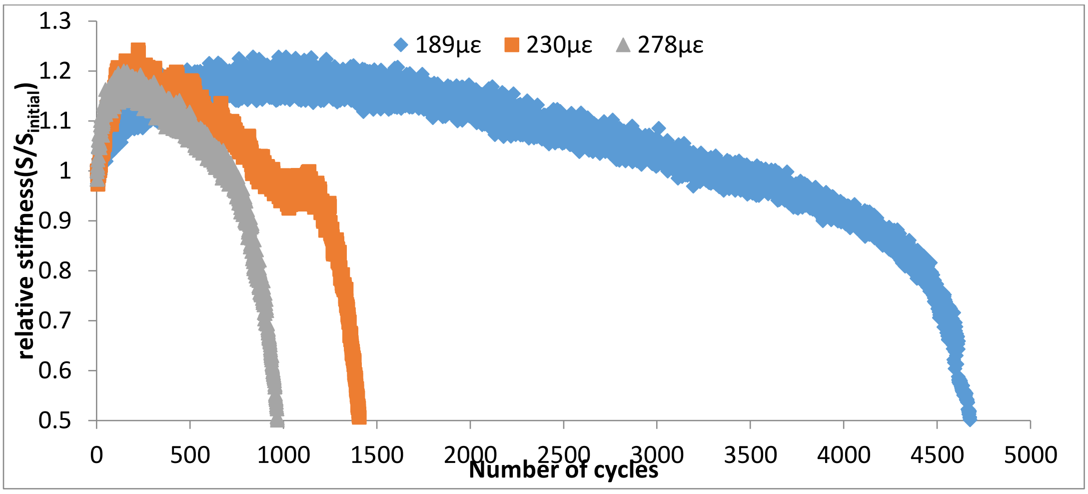


Figure 7-19 Stiffness evolution curve for 50%RAP+1%Cement-FBM at different initial radial strain levels

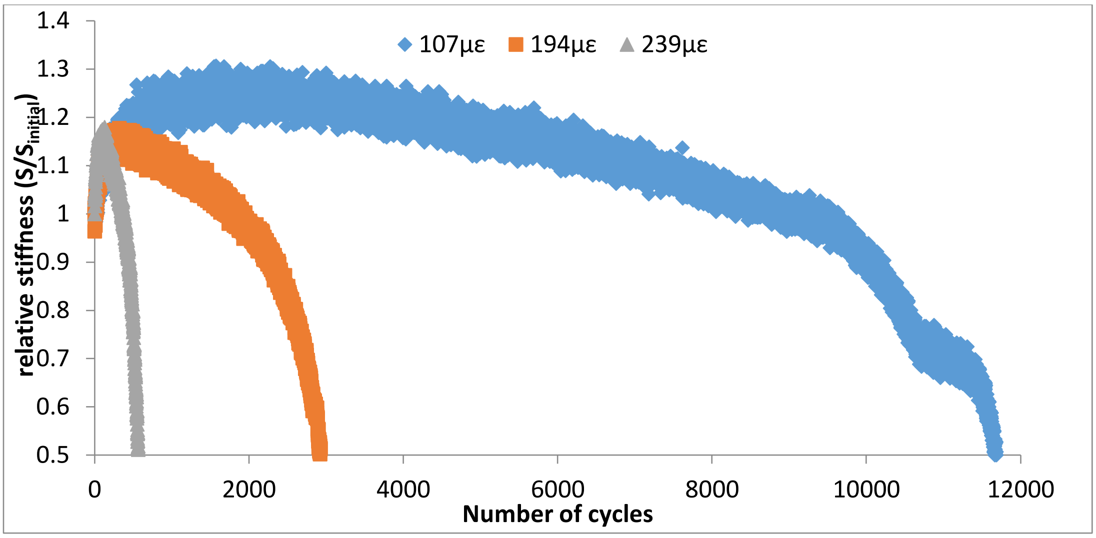


Figure 7-20 Stiffness evolution curve for 75%RAP+1%Cement-FBM at different initial radial strain levels

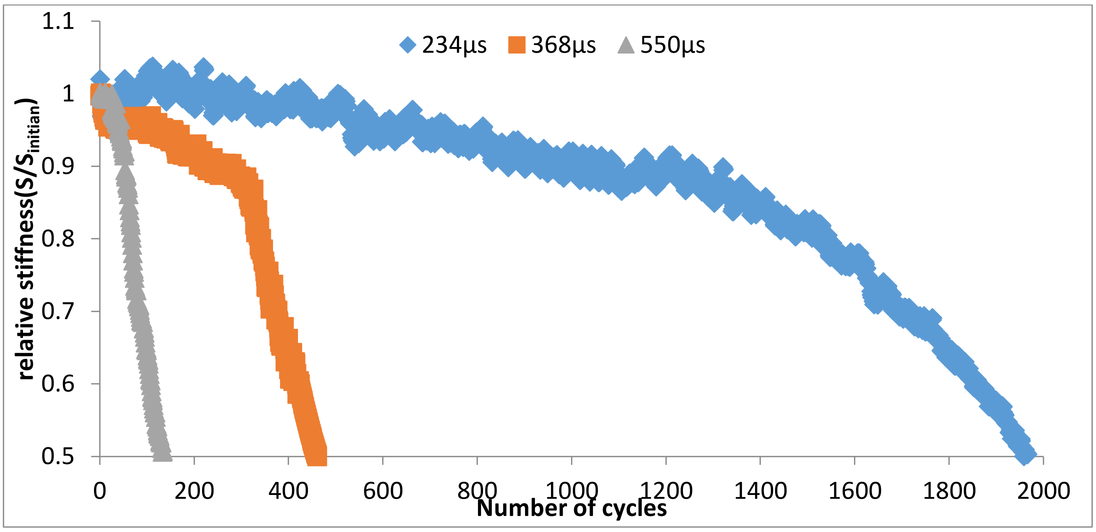


Figure 7-21 Stiffness evolution curve for DBM 90 at different initial radial strain levels

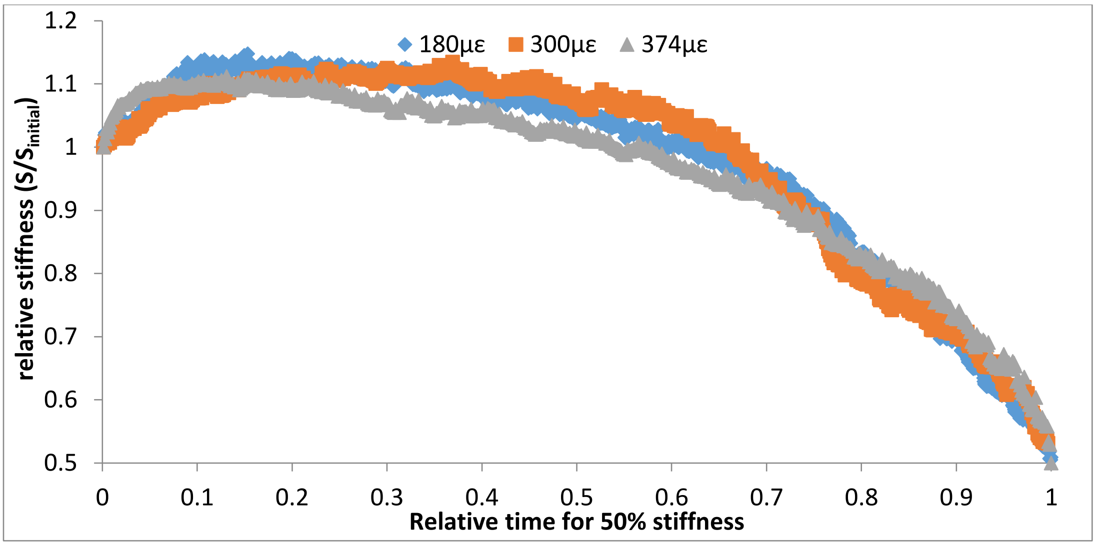


Figure 7-22 Normalised stiffness reduction curve for 50%RAP-FBM

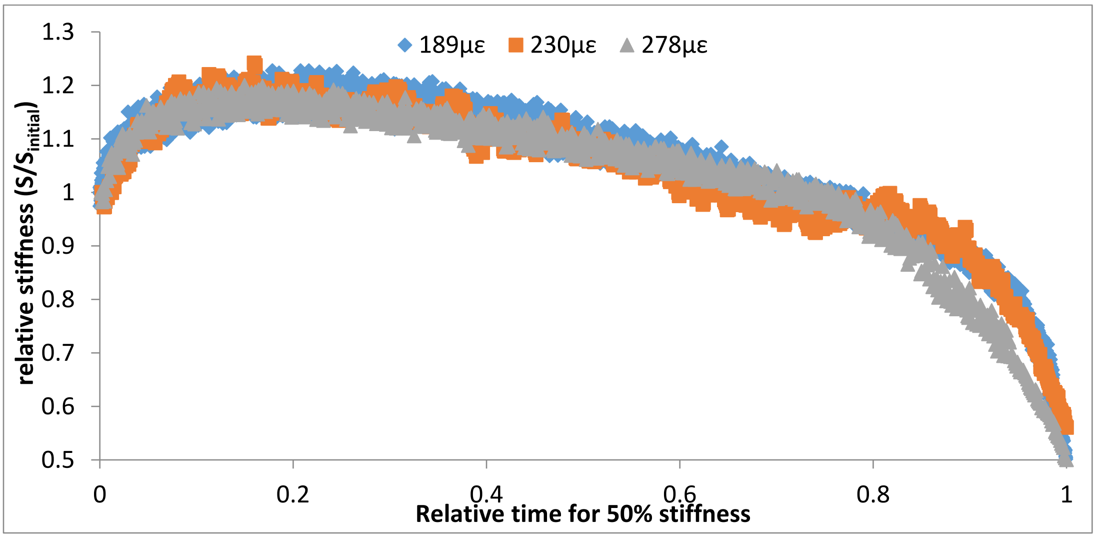


Figure 7-23 Normalised stiffness reduction curve for 50%RAP+1%Cement-FBM

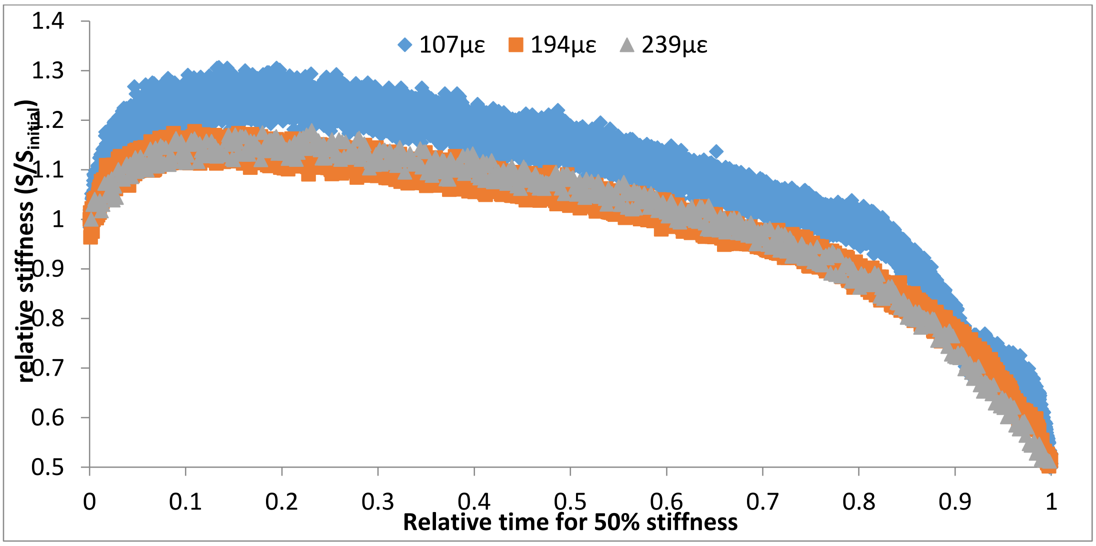


Figure 7-24 Normalised stiffness reduction curve for 75%RAP+1%Cement-FBM

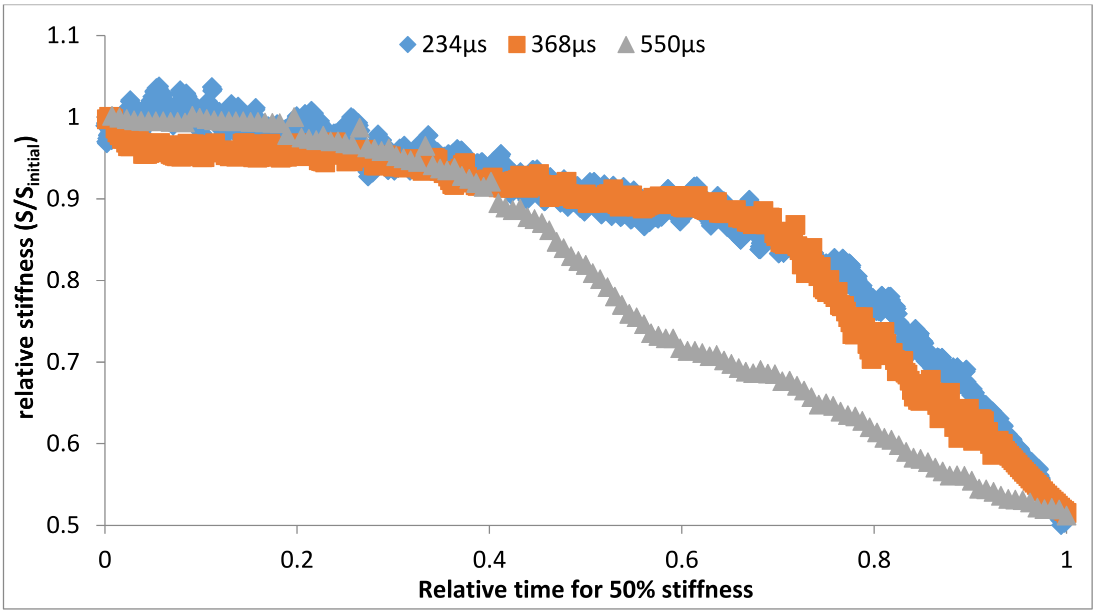


Figure 7-25 Normalised stiffness reduction curve for DBM 90

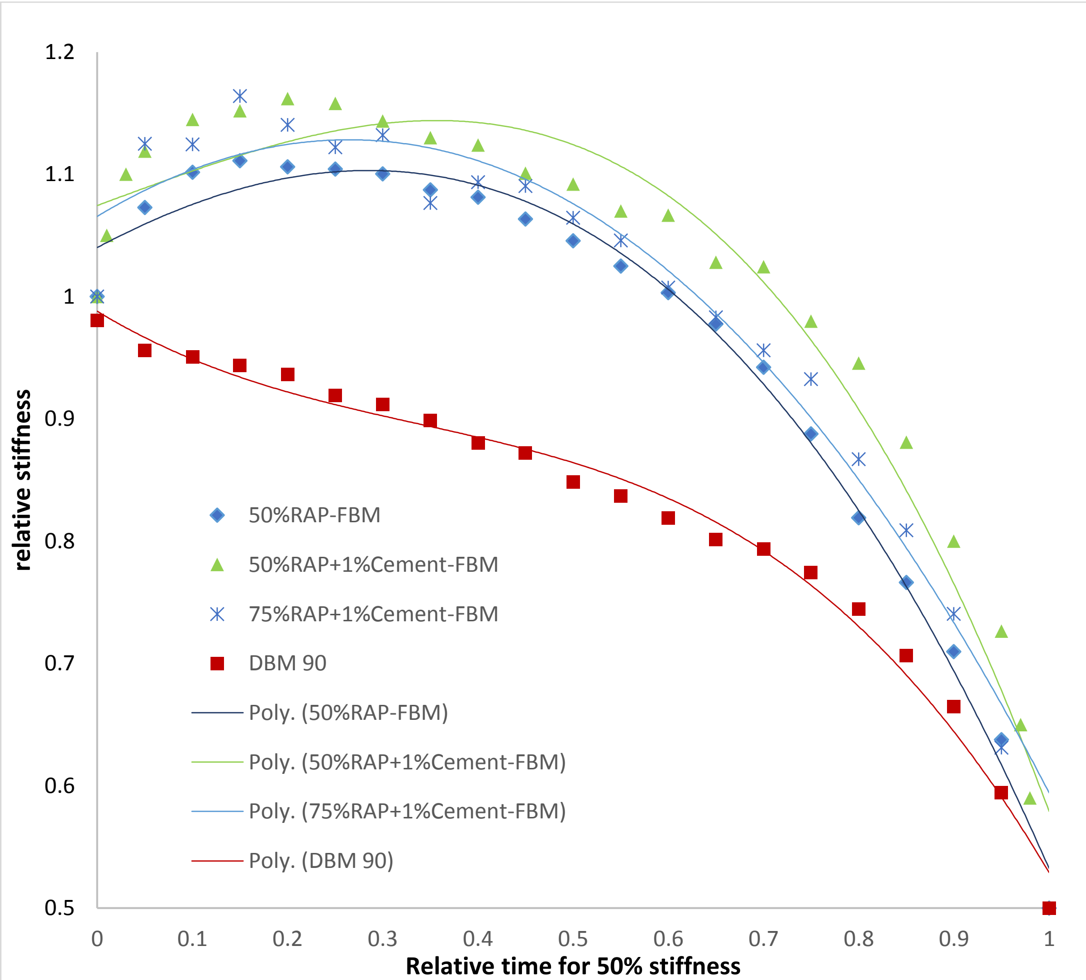


Figure 7-26 Comparison of stiffness evolution of the mixtures considered in the study

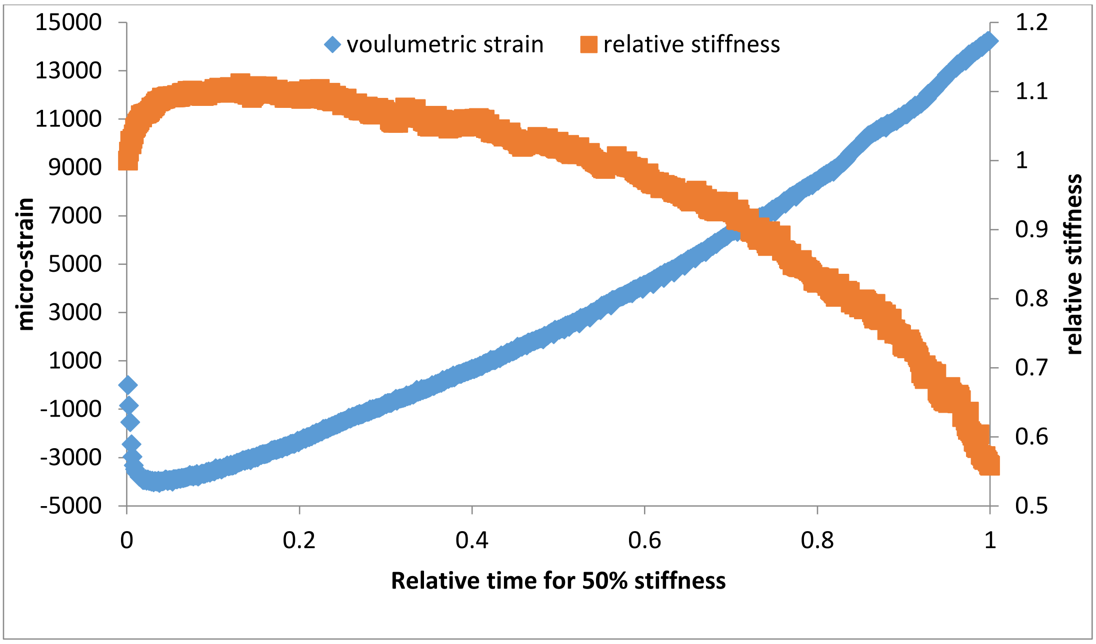


Figure 7-27 Volumetric strain of 50%RAP-FBM test at initial radial strain 300με

7.3.2.4 Hopman et al. curve

Though analysing the stiffness evolution curves is useful in terms of understanding the development of fatigue damage they give little information regarding the actual points of fatigue failure. This was found to be particularly true with regards to FBMs as they did not follow the traditional three phase response which has two inflection points, as for DBM 90 in Figure 7-26. The first point defines a decreasing rate of stiffness loss associated with micro cracking in the specimen while the second inflection point reflects a change in mechanical behaviour which could be correlated to macro-cracking. Therefore, the method proposed by Hopman et.al[144], which involves plotting the product of the ratio of stiffness of the specimen (S) to the initial stiffness (S_{inital}) and the number of cycles (n)[$n*S/ S_{inital}$] was used. In this method there exists a transition point, the peak value of the curve, represents the shift from micro-cracking to macro-cracking. This method of identifying the fatigue failure point also validates the testing methodology used in the study.

The $n*S/S_{inital}$ against n plots as a means of defining the fatigue failure points are presented in Figure 7-28 to Figure 7-31. The fatigue response in the figure shows that there exist transition points for all the mixtures considered in the study and at all the initial radial strain levels. The transitions points are represented in the figures by vertical dashed lines. It is important to note that for all the tests considered in the study the transition points exists within the 50% stiffness reduction. Therefore, the failure criterion considered in this study, which is the fatigue life associated with a 50% stiffness reduction, is not too conservative.

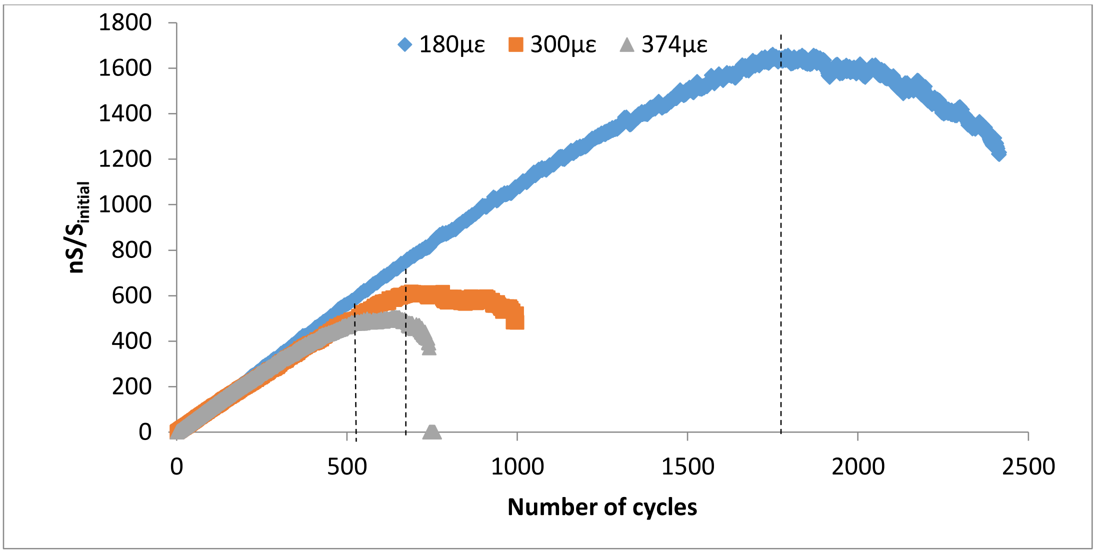


Figure 7-28 Fatigue transition points for 50%RAP-FBM

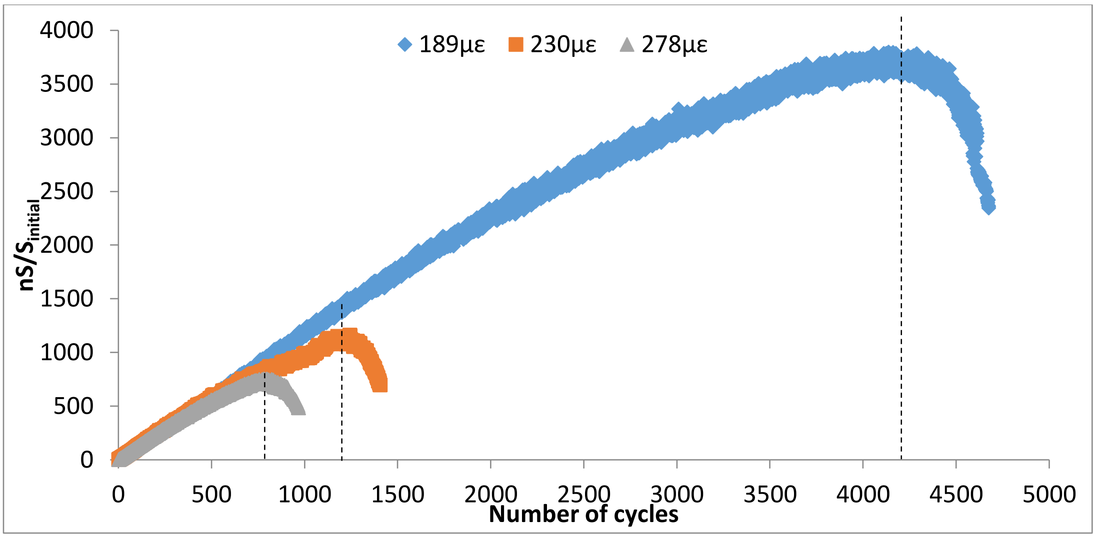


Figure 7-29 Fatigue transition points for 50%RAP + 1%Ceemnt-FBM

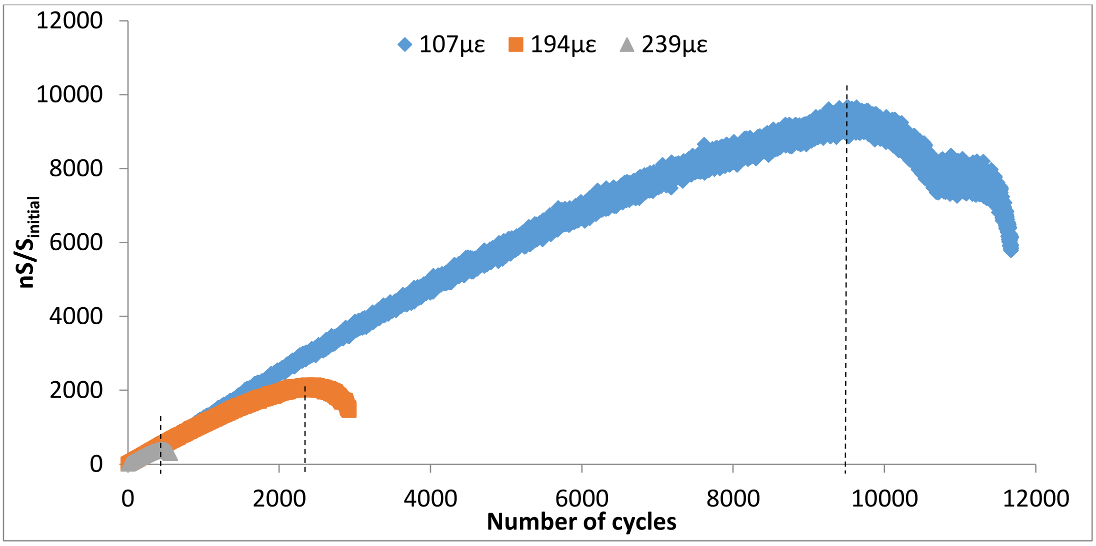


Figure 7-30 Fatigue transition points for 75%RAP+1%Cement-FBM

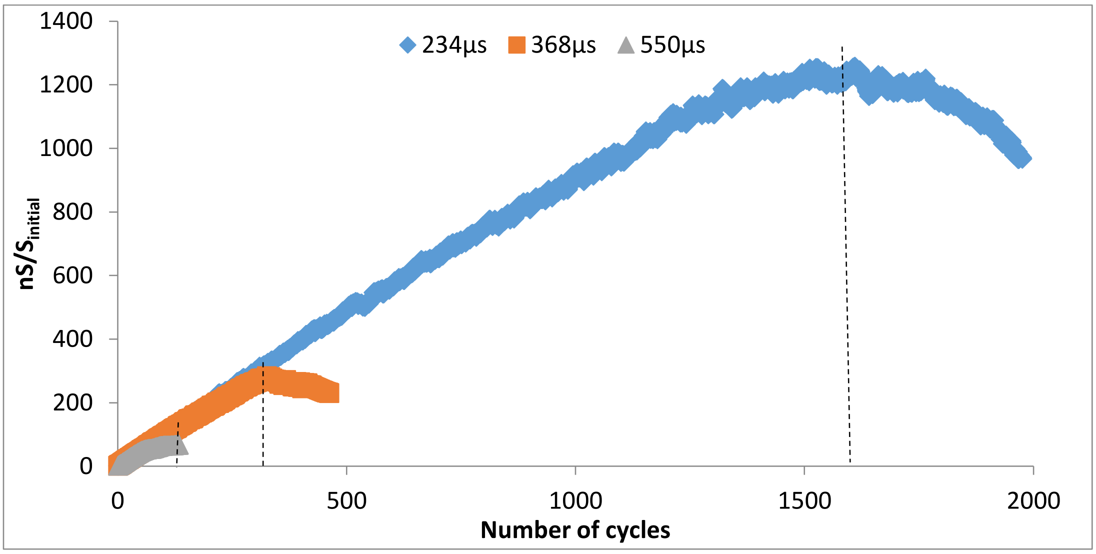


Figure 7-31 Fatigue transition points for DBM 90

7.3.2.5 Vertical (axial) strain

Analysis of the axial dynamic creep data facilitates understanding of the permanent deformation behaviour during the test. The axial strain, along with strain rate per cycle, is plotted against number of cycles in Figure 7-32, Figure 7-34 and Figure 7-36. The total number of cycles for each test was the number required to reach 50% stiffness. From the figures it is evident that both FBMs and DBM 90 behaved similarly in terms of permanent deformation. This behaviour is similar to that of bitumen and traditional asphalt mixtures in which the creep curve can be divided into three regions: a primary creep region where the strain rate decreases, a secondary creep region where the strain rate is almost constant, known as steady-state strain rate, and a tertiary creep region where the strain rate increases rapidly as the specimen approaches failure. This behaviour is clearly demonstrated by the strain rate plots in the Figure 7-32, Figure 7-34 and Figure 7-36. As expected the strain rate was significantly influenced by the applied stress (initial radial strain) of the test. The tests that were carried out at high stress levels (high initial strain) exhibited higher strain rates and vice versa.

It is clear from Figure 7-32, Figure 7-34 and Figure 7-36 that lower stress levels (initial strains) lead to higher permanent vertical strain at 50% stiffness. The vertical strain data was normalised in a similar way to stiffness so as to analyse the vertical strain without strain level dependency. The data points were normalised in relation to the number of cycles needed to reach failure, which is to reach 50% stiffness in the present case. Thus the relative time was calculated for each data point by dividing the number of cycles by the number of cycles to half the initial stiffness. The normalised vertical strain (permanent) curves can be found in Figure 7-33, Figure 7-35 and Figure 7-37. The total vertical strain in the plots gives the permanent strain of the specimen when it reaches 50% initial stiffness. As can be seen, at all relative time points, the tests with low stress values were found to have higher strain values. Therefore it can be interpreted from these plots that higher stress application causes less permanent deformation than lower stress levels by the time specimens reach 50% stiffness.

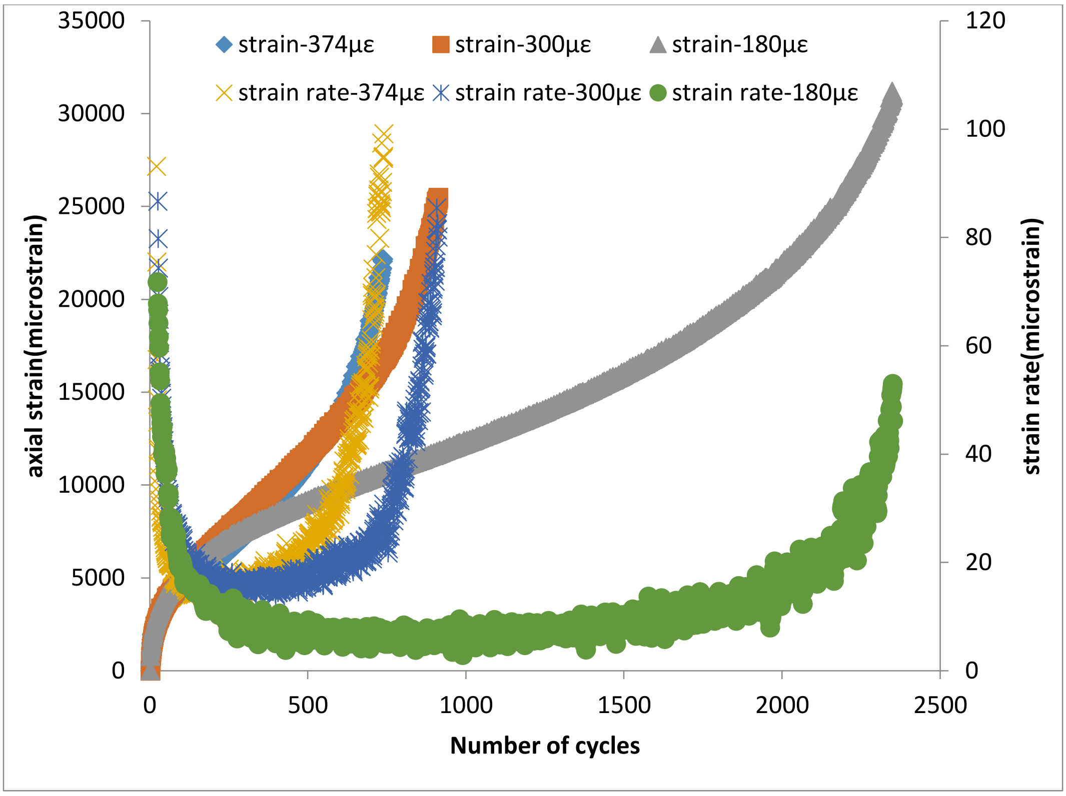


Figure 7-32 Dynamic creep and strain rate curves of 50%RAP-FBM at different stress (initial strain) levels

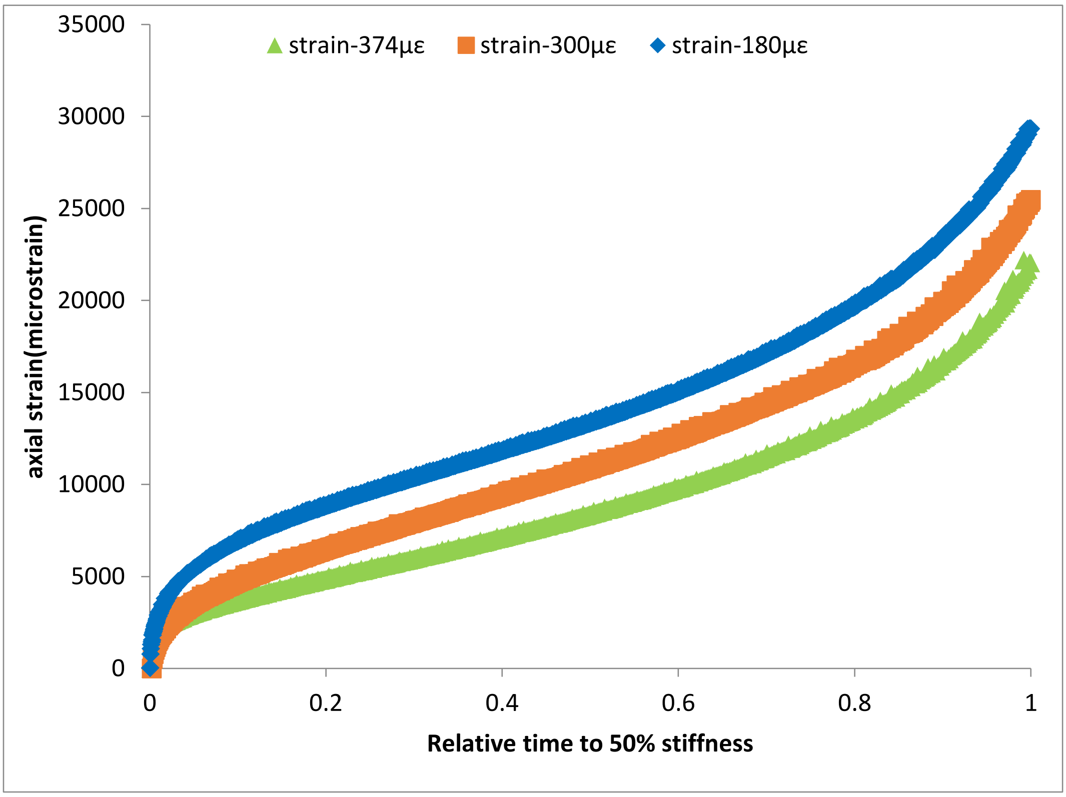


Figure 7-33 Normalised dynamic creep curves for 50%RAP-FB

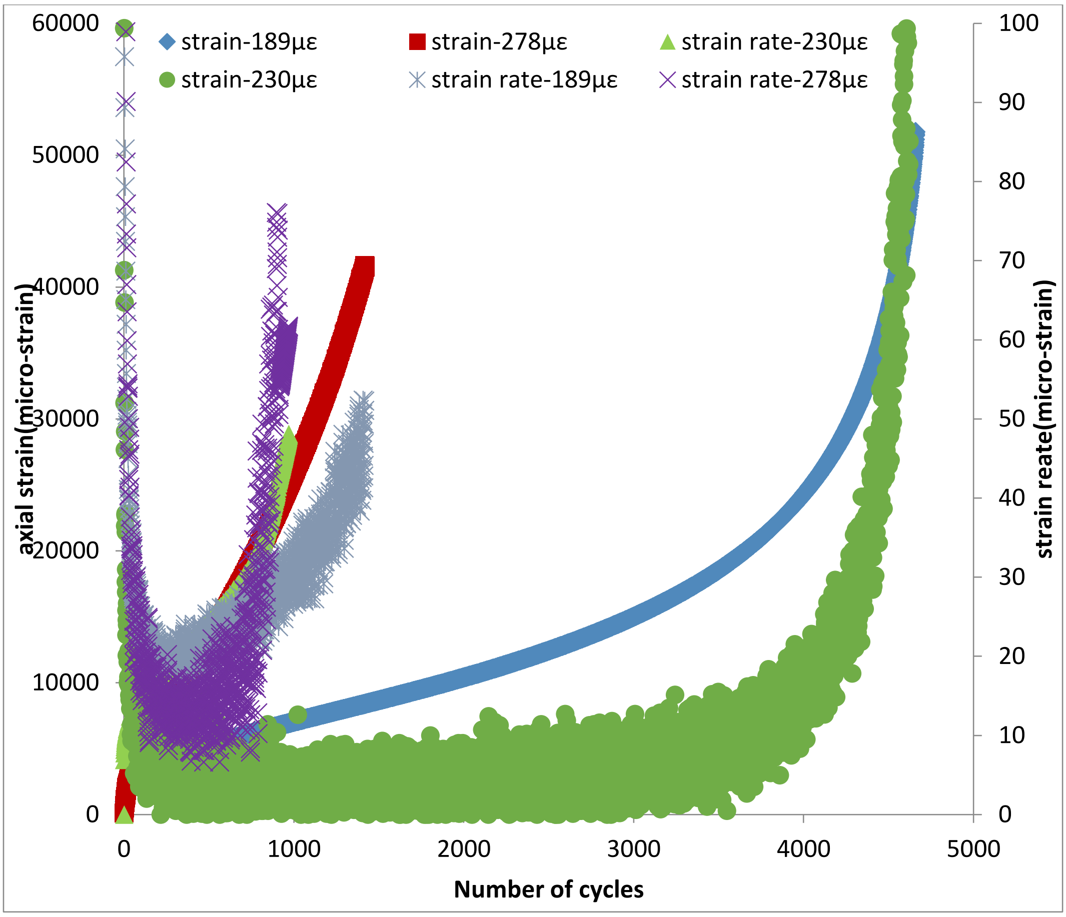


Figure 7-34 Dynamic creep and strain rate curves of 50%RAP+1%Cement-FBM at different stress (initial strain) levels

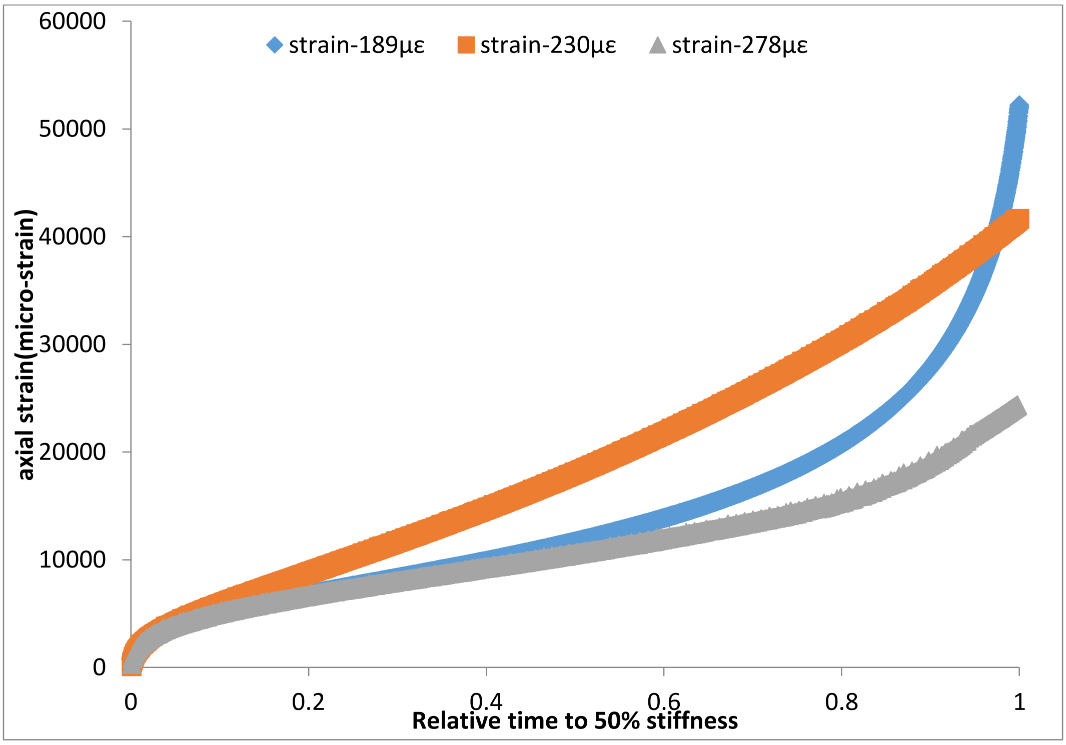


Figure 7-35 Normalised dynamic creep curves for 50%RAP+1%Cement-FBM

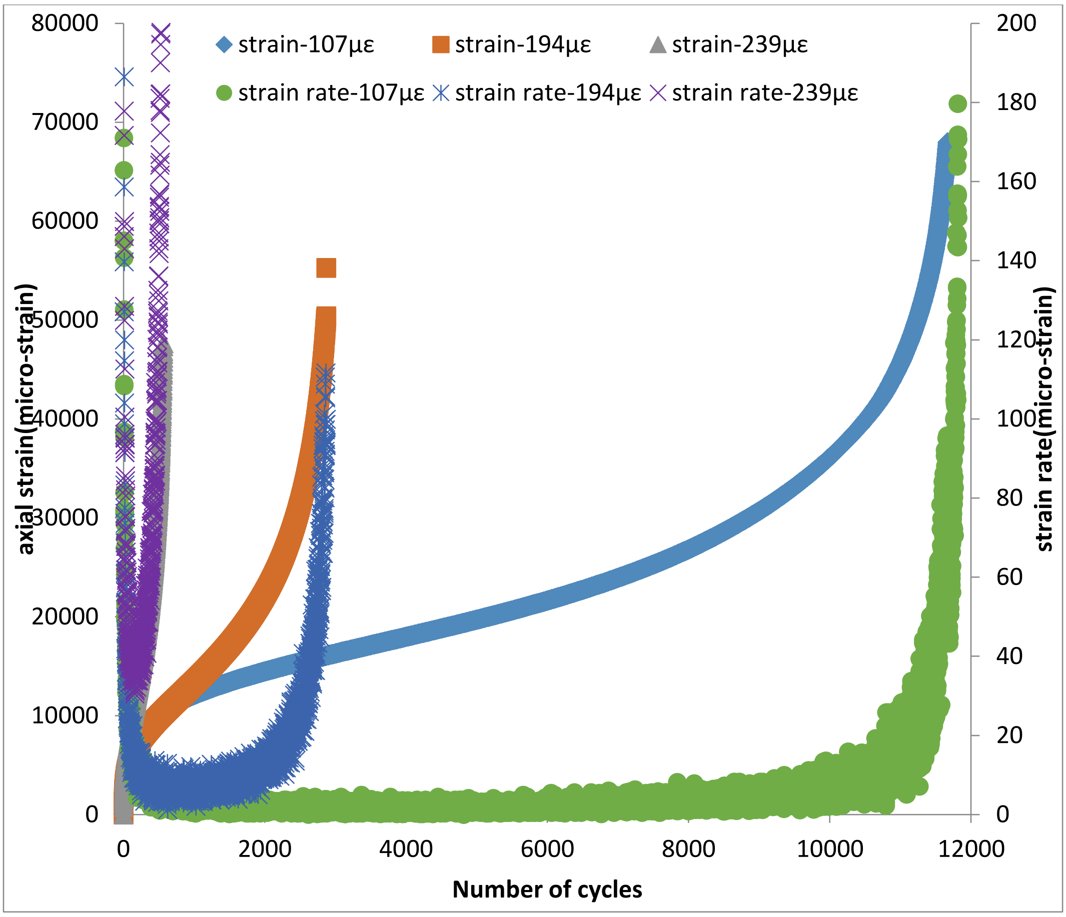


Figure 7-36 Dynamic creep and strain rate curves of 75%RAP+1%Cement-FBM at different stress (initial strain) levels

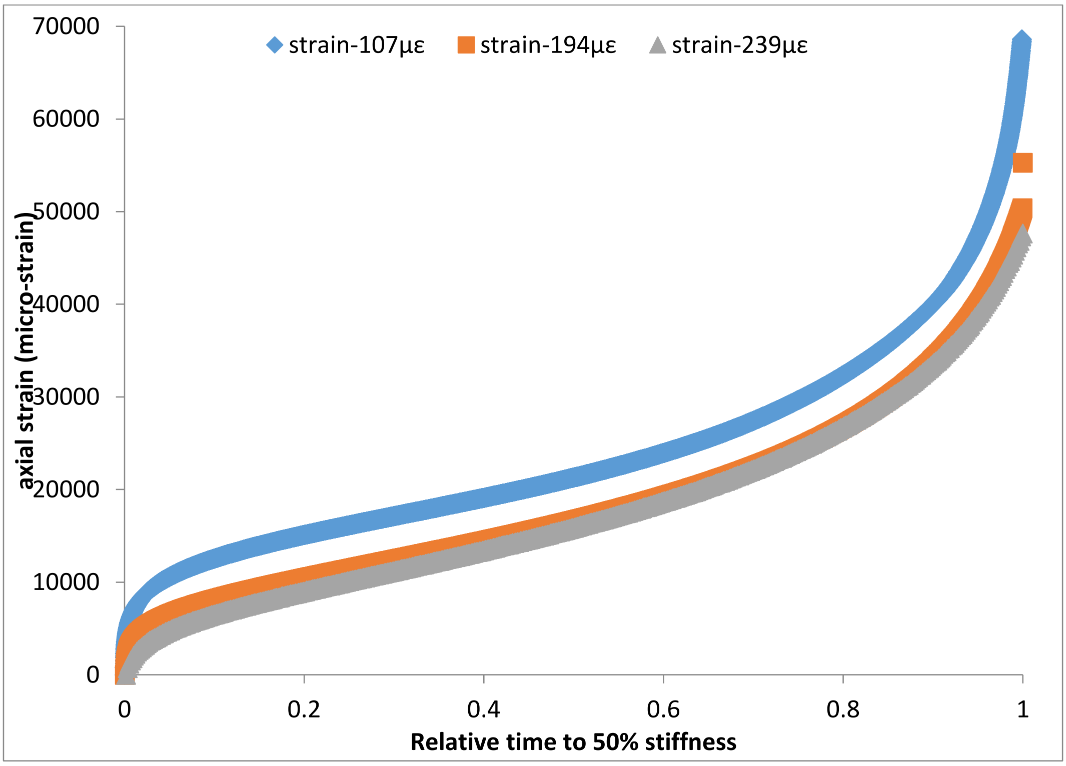


Figure 7-37 Normalised dynamic creep curves for 75%RAP+1%Cement-FBM

7.4 Summary

The primary objective of the present study is to evaluate the permanent deformation resistance and fatigue resistance of FBM when compared with conventional HMA.

The permanent deformation resistance was assessed by performing RLAT tests on cylindrical specimens compacted by gyratory compactor. By evaluating the permanent deformation characteristics the temperature sensitivity of HMA in terms of permanent deformation and influence of stress on temperature sensitivity were also assessed. The RLAT test results indicate that both test temperature and stress level have significant influence on permanent deformation characteristics as expected. The effect of stress on permanent deformation was increased with increase in test temperature. It was also found that from limited tests and mixture combinations, RAP content has only a slight influence on permanent deformation of FBMs. However, the presence of cement greatly improved the permanent deformation resistance of FBMs. Nevertheless the effect of cement decreased with increase in test temperature. FBMs were also found to be less temperature susceptible than HMA in terms of permanent deformation and, within FBMs, mixtures with cement were found to be more sensitive than FBMs without any cement.

For assessing the fatigue performance of FBMs, the ITFT was initially considered to investigate the effect of cement on the fatigue life. The ITFT tests results showed that the FBMs without cement (50%RAP-FBM) have lower fatigue life than HMA (DBM90) at any initial strain level. Nevertheless, similar to permanent deformation, the fatigue life was improved with the addition of 1% cement to FBMs. A mixture (FBM) with 3% of cement was also tested in the ITFT to validate the suggested effect of high cement content available in the literature. The results indicate that 3% cement addition greatly reduced the flexibility of FBMs and thus the mixtures showed relatively lower fatigue life at a given strain level. However, the above discussion was not found to be completely valid when uniaxial tests were carried out. In stress controlled uniaxial tests, a sinusoidal load of 1Hz frequency was applied axially to induce tensile strain in the radial direction. The failure criterion considered in the study was the number of cycles to reach 50% stiffness and this was plotted against the measured initial strain values. Results indicated that there was not much difference in fatigue life among different mixtures (omitting the 3% cement case) and also between FBM and HMA. However, stiffness evolution curves showed that FBMs fail in a different pattern compared to HMA. Unlike HMA, which showed a three stage evolution process, for FBMs the stiffness increased rapidly to reach a maximum and decreased at a slower rate until failure. It was also found that by plotting curves according to Hopman et al., which identify the fatigue failure transition point, use of the 50% stiffness criterion for fatigue life evaluation is not a conservative approach.

Uniaxial tests also revealed that, although in fatigue the FBMs were found to behave differently from HMA, in terms of permanent deformation, FBMs behave similarly to HMA in that a steady state strain rate was achieved.

8 STRUCTURAL DESIGN OF PAVEMENTS INCORPORATING FBMS

8.1 Introduction

The structural design of pavements is required to make sure that the structure serves its function structurally and functionally in an economically viable manner for the designed life period. However, this task of designing a pavement structure is not as straightforward as for some other engineering structures. Moreover, there is never a unique design for any combination of design inputs [142]. The major issue regarding the design of pavement structures is developing a useful relation between the laboratory and field performance of the materials used in the pavement. The Mechanistic – Empirical method which is based on the mechanics of the materials and relates an input such as wheel load to an output (pavement layer response) such as stress or strain is a possible option. This chapter will apply this Mechanistic–Empirical approach to design of pavements with FBMs. In the present study design charts have been developed for pavements incorporating FBMs as base course material. The scope of the study was limited to design based on the fatigue criterion only.

The present chapter examines the effect of the FBMs that have been considered in this study on the pavement response. 16 pavement structures (Table 8-1) have been chosen for sensitivity analysis. These were selected such that they represent common practice in the industry. Design charts were developed for pavements incorporating base courses comprising FBMs. Traditional bituminous mixtures were also considered as base course in the same calculations for a direct comparison. Non-linear elastic analysis was considered to provide mechanistic response (stresses and strains) analysis of the both FBM base and the granular sub-base of the pavement structures considered. Two different design methodologies were tested to assess the implications of applying the analysed pavement design methods on pavement layers with FBMs. The first is a traditional approach for conventional flexible pavements which is based on pavement life as a function of computed tensile strain in the material and interpreting fatigue data in a conventional way. The second approach is believed to be more appropriate for FBMs because of their characteristic behaviour noticed in the previous chapter (Chapter 7). This new method is an iterative approach which considers the actual stiffness evolution of the mixture.

Table 8-1 Pavement structures considered in the present study

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

Note: Number in parentheses indicates the number of sub layers for non-linear analysis

8.2 Flexible pavements design principles

In order to design any civil engineering structure it is important to understand how it fails in service. A pavement structure doesn't fail suddenly, rather it gradually deteriorates with time. This deterioration is primarily caused by traffic loads, a function of the number and the magnitude of the wheel loads[145]. The two principal modes of failure are cracking of bound layers and rutting in the wheel tracks with contributions from all pavement layers.

Figure 8-2 illustrates a typical flexible pavement. The main structural element of the pavement structure is the base. This is usually a bituminous mixture in flexible pavements overlaid by a relatively thin (generally 30-100 mm) surfacing layer. The role of the surfacing layer is to provide a safe surface for traffic by providing adequate skidding resistance. This layer is generally not considered as a structural layer. The sub-base and subgrade together are considered as the pavement foundation. For designing the pavements a standard wheel load of 40kN was considered and the actual load spectrum is converted into an equivalent number of standard loads.

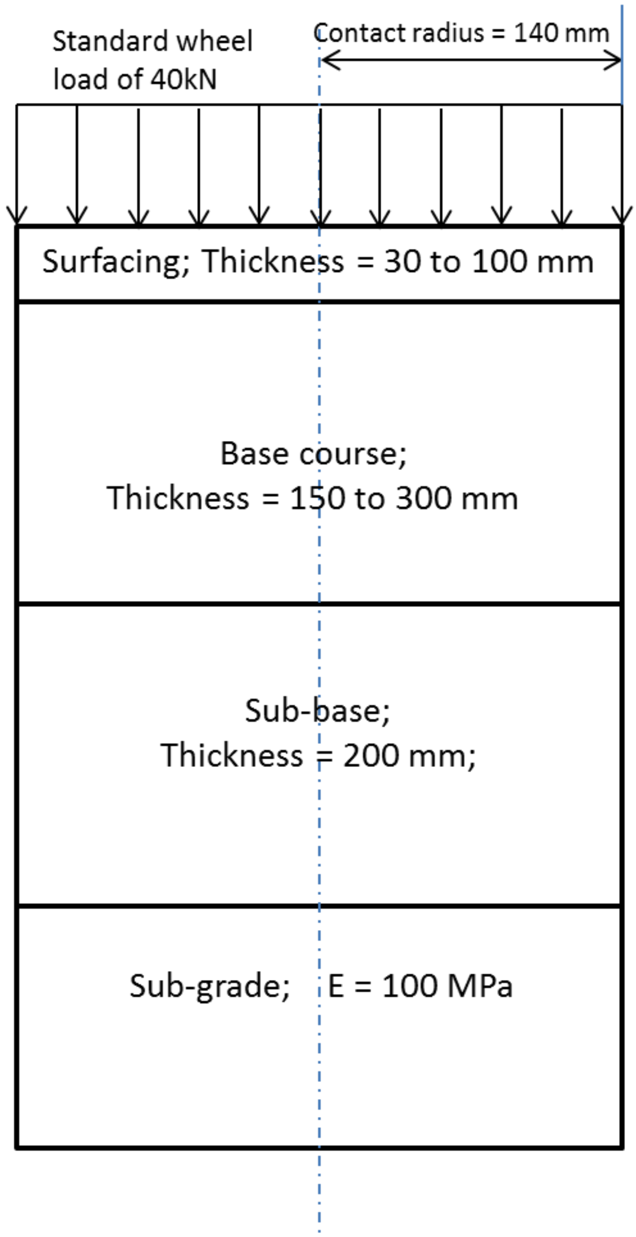


Figure 8-1 Pavement structure considered for analysis

8.3 Failure criteria

In the present study fatigue designs for the pavements were based on laboratory tests obtained in the previous chapter (Chapter 7). The fatigue equation and regression coefficients are presented in Eq. 7-3 and Table 8-2. These laboratory tests as discussed in Chapter 7 employed sinusoidal loading of 1Hz (stress controlled). In practice these loadings are distributed across the pavement and there are also rest periods between wheel loadings. To accommodate these effects, a correction factor called a shift factor (or transfer function) is usually applied to the fatigue equation. However estimating this shift factor correctly is not easy as it depends on many factors such as type of loading, frequency of loading, test temperature etc. For flexible pavements the shift factor ranges from 2 to 440 in different research findings [146]. In the present project fatigue tests as discussed in

Chapter 7 were conducted at 1Hz frequency at 20°C which are considered as very conservative testing conditions. Therefore, a shift factor of 400 has been employed.

$$N_f = C * (\frac{1}{\varepsilon_{x,max}})^m \qquad \text{Equation 7-3}$$

Table 8-2 Regression coefficients for fatigue equations from uniaxial test

Mixture type	C	m	R^2
50% RAP - FBM	2.00E+10	3.114	0.98
50% RAP+1% Cement - FBM	1.00E+12	3.774	0.98
75% RAP+1% Cement - FBM	9.00E+11	3.825	0.97
DBM 90	2.00E+10	2.968	0.99

8.4 Analytical approach

The Mechanistic (Analytical) – Empirical approach to design flexible pavement has two steps. In the first step, the response (stresses and strains) of the pavement layers to the wheel loads is calculated using analytical tools (models). From this data critical points are identified. In the second step these critical responses are compared to the permissible stresses and strains identified from the laboratory tests. For design purposes, generally only two locations in the pavement structure are considered for critical stresses and strains. These are the bottom of the asphalt base layer and the top of the subgrade where the maximum values of key parameters develop in most cases. Since in the present study design is based on fatigue criteria, tensile strain at the bottom of the base layer was used as the design parameter. This is because fatigue cracking of the bituminous layer under repeated wheel loads is initiated by the magnitude of tensile strain rather than the stress and therefore this parameter is generally used to characterise the fatigue property of a bituminous material such as in Eq. 7-3.

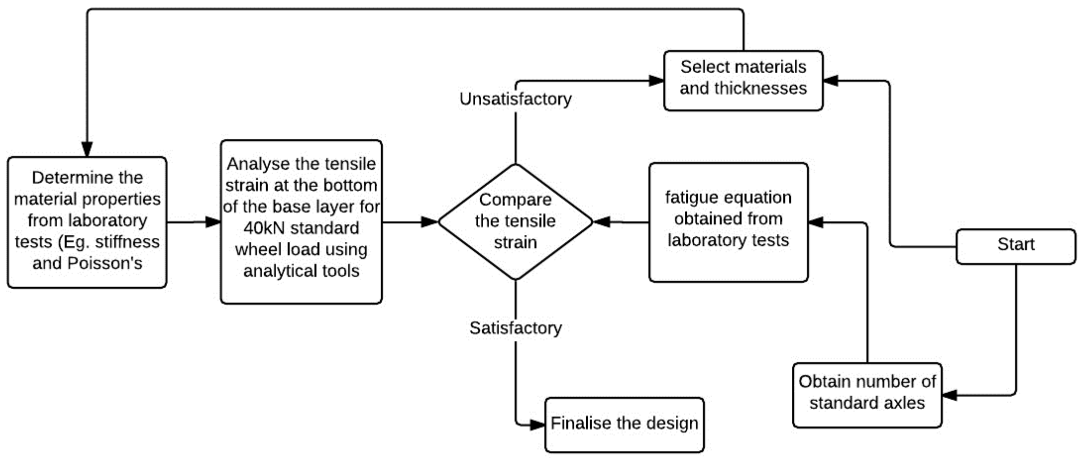


Figure 8-2 Flow chart for design of flexible pavement against fatigue cracking

Figure 8-2 shows a flow chart for Mechanistic-Empirical pavement design against fatigue cracking using the fatigue relationships that were developed in the previous chapter. The design process involves the selection of available materials and thicknesses and to ensure that the fatigue criterion is satisfied. For the number of load applications expected during the design life of the pavement, the maximum allowable value of each strain criterion has to be determined. As can be seen in the flow chart, the key parameters for the flexible pavement design are cumulative number of standard axle loads for the design life, the design temperature, properties of the materials used in each layer and the fatigue characteristics of the base layer. As discussed the standard wheel load of 40kN is used for design purpose and the load spectrum during the design life is converted to an equivalent number of standard loads using a fourth-power law.

The traffic (wheel load) induced response of pavement layers (stresses and strains) depends upon the interaction of layer thicknesses and material properties. Elastic properties are characteristics of importance as they influence the stresses and strains and thus failure of the pavement layer. However, these elastic properties are sensitive to temperature and rate of loading. Although across the UK the mean annual temperature at low altitudes varies from about 8°C to 11°C, pavement design is recommended to be carried out at a reference temperature of 20°C (Highways Agency). In addition to that the Highways Agency also suggests testing the stiffness of the material at a reference loading rate of 5Hz. In the present study fatigue tests were also carried out 20°C from which the fatigue equations were derived.

Given the non-linear elastic characteristics of FBMs as identified in previous chapters, this means that strains are non-linear functions of the stress condition. In the present chapter analysis was carried out using KENLAYER, an analytical tool which is capable of both linear and non-linear analysis on similar materials. KENLAYER is an analytical tool for flexible pavements in the KENPAVE computer program developed by Huang (2004). Huang stated that 3 methods have been incorporated into KENLAYER for non-linear analysis. Method 1, the most accurate but time consuming and in which the stress dependent layer is subdivided, was used in this work. He reported that using a 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. Therefore the non-linear elastic approach was adopted in this exercise. The other 2 methods and detailed description of the capabilities of the program are documented by Huang (2004).

8.5 Conventional fatigue life approach

For non-linear elastic analysis using KENLAYER the parameters of the traditional K-θ model (k1 and k2) are considered. For obtaining these parameters triaxial tests were carried out on FBMs at confining stress of (σ_c) 50–200 kPa and deviator stress (σ_d) 50–240 kPa; these stress levels having been first determined in preliminary KENLAYER runs. The testing conditions were temperature of 20°C and loading frequency of 5Hz. The control mixture, which is a DBM90, was considered as linear for analysis with a stiffness value of 3200 MPa (ITSM@20C). For other layers in the pavement structure hypothetical material properties were assumed as those materials for those layers in question were not studied in the present work. The relevant material properties of the pavement layers are detailed in Table 8-3. From the uni-axial compression tests that were carried out in Chapter 7 it was found that the Poission's ratio of the FBMs considered in the study was found to be in the range of 0.2 to 0.45 independent of the mixture type. The preliminary structural analysis using Poission's ratio values in this range revealed that the Poission's ratio has limited influence on the response parameters such as stresses and strains. Therefore, an average value of 0.35 was considered for the structural analysis. The material properties of the surfacing layer, sub-base and subgrade were made constant for all cases considered. The other required information such as loads and layer thicknesses are as stated in Figure 8-2.

Table 8-3 Pavement Layer Material Properties

Material	Layer	Density (kg/m^3)	Poission's ratio	Elastic Behaviour	ITSM/k1 and k2 (MPa)
HRA(40/60)	Surfacing	2400	0.35	Linear	3100
DBM90	Base	2400	0.35	Linear	3200
50%RAP	Base	2180	0.35	Non-linear	424 and 0.413
50%RAP+1%Cement	Base	2180	0.35	Non-linear	678 and 0.351
75%RAP+1%Cement	Base	2140	0.35	Non-linear	707 and 0.372
Unbound	Sub-base	2000	0.35	Non-linear	50 and 0.55
Unbound	Subgrade	1800	0.4	Linear	100

8.5.1 Design approach

A total of 16 cases (Table 8-1) were studied for multilayer elastic analysis to obtain the response of each structure to a 40kN standard wheel load. As seen in Table 8-3 non-linear elastic behaviour was considered for base layers with FBMs and for sub-base in all cases. For non-linear analysis the corresponding layer was divided into sub-layers of 50 mm thickness. As in KENLAYER the non-linear analysis is an iterative approach, a seeding stiffness value is required for the analysis. Seeding stiffness values of 500 to 1200 MPa were considered depending on the number of sub-layers. For each iteration a constant stiffness value is calculated by KENLAYER based on the magnitude of stress obtained in the previous iteration. For analysis the stresses at the mid-depth of each sub-layer are used to determine the stiffness. After stresses are calculated the stiffness of each sub-layer is recalculated and a new set of stiffnesses is allocated to each sub-layer. This is repeated by the KENLAYER program until the stiffness converges to a specific tolerance limit. The outputs from the KENLAYER analysis are vertical, horizontal and shear stresses and strains and vertical displacement values. The output file also includes the stiffness values for each of the sub-layers.

The significance of the non-linear analysis approach for FBMs can be identified in Figure 8-3. The figure shows the distribution of horizontal strain through the depth of the pavement. Analysis was carried out considering 50%RAP-FBM first as non-linear and then linear for Case 6 in Table 8-1. For the linear analysis case the ITSM value at 20°C which was found to 2550 MPa was used. The non-linear parameters used were as in Table 8-3. As can be seen from Figure 8-3 within the surface layer (50 mm thick) the horizontal strain distribution is not very different. However, in the base course and sub-base the horizontal strain distribution is considerably different using non-linear analysis when compared with linear analysis. The higher values of strain for non-linear analysis are because of stress dependent stiffness values in FBMs. At lower stress levels these materials tend to have low stiffness because of their non-linearity (similar to granular material).

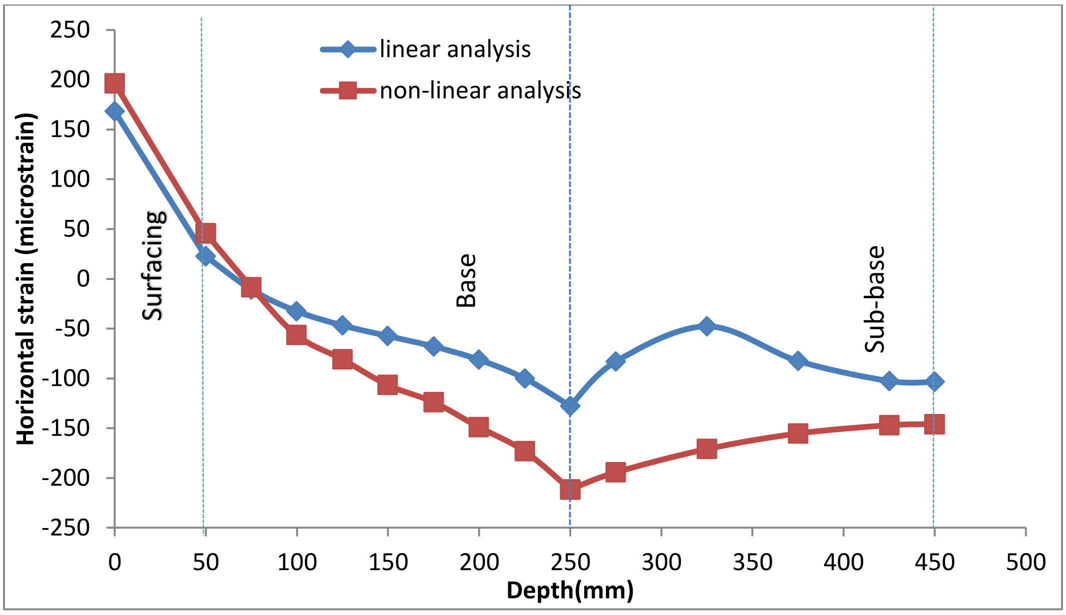


Figure 8-3 Horizontal Strain Distribution through Pavement Structure (Case 6) for 50%RAP – FBM

8.5.2 Stress and strain distribution

Using pavement structure case number 6 from Table 8-1, i.e. surfacing layer thickness of 50mm, base layer thickness of 200mm (divided into 4 equal sub-layers) and a sub-base layer of 200mm, the stress and strain distributions through pavements containing 50%RAP-FBM, 50%RAP+1%Cement-FBM, 75%RAP+1%Cement-FBM and DBM90 as road base layers were analysed using KENLAYER. Figure 8-4 shows the vertical stress distribution in the pavements. The results of the analyses indicate that vertical stress distribution in the FBMs is not significantly different for all the pavement types considered. It can be seen from the figure that vertical stress (compressive) reduces with depth. Figure 8-5 shows the vertical strain distribution in the pavements. Figure 8-6 presents the horizontal stress distribution in the pavement structure. The figure shows that the horizontal stress gradually changed with depth from a relatively high compressive stress (negative value) at the top of the surfacing layer to a tensile stress (positive value) on top of the subgrade. The figure also indicates significant differences in stresses for the base layers and that the maximum horizontal tensile stress and vertical compressive strains are at the bottom of the base layer (Figure 8-5 and Figure 8-6).

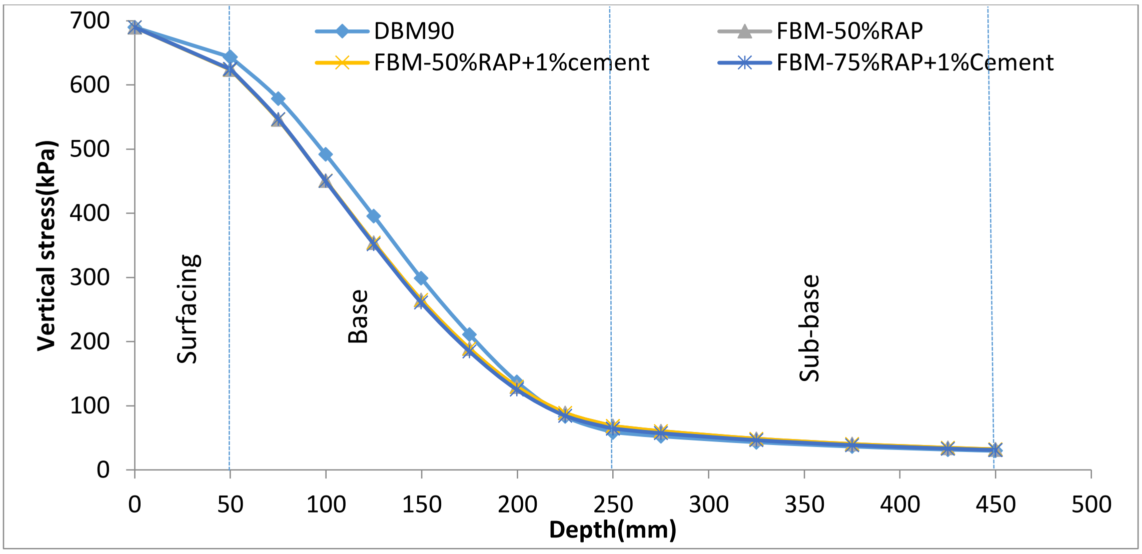


Figure 8-4 Vertical Stress Distribution through Pavement Structure (Case 6)

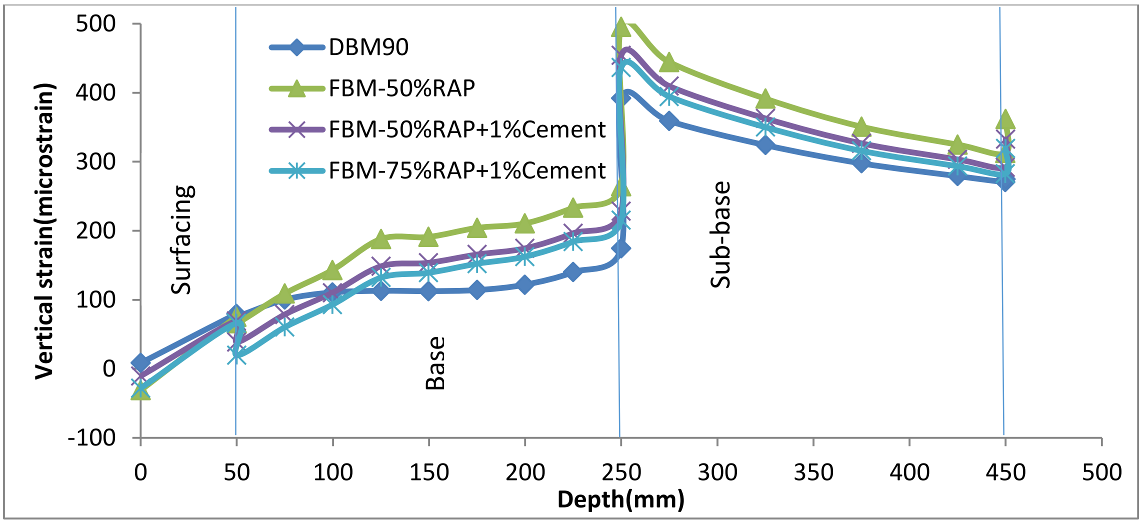


Figure 8-5 Vertical Strain Distribution through Pavement Structure (Case 6)

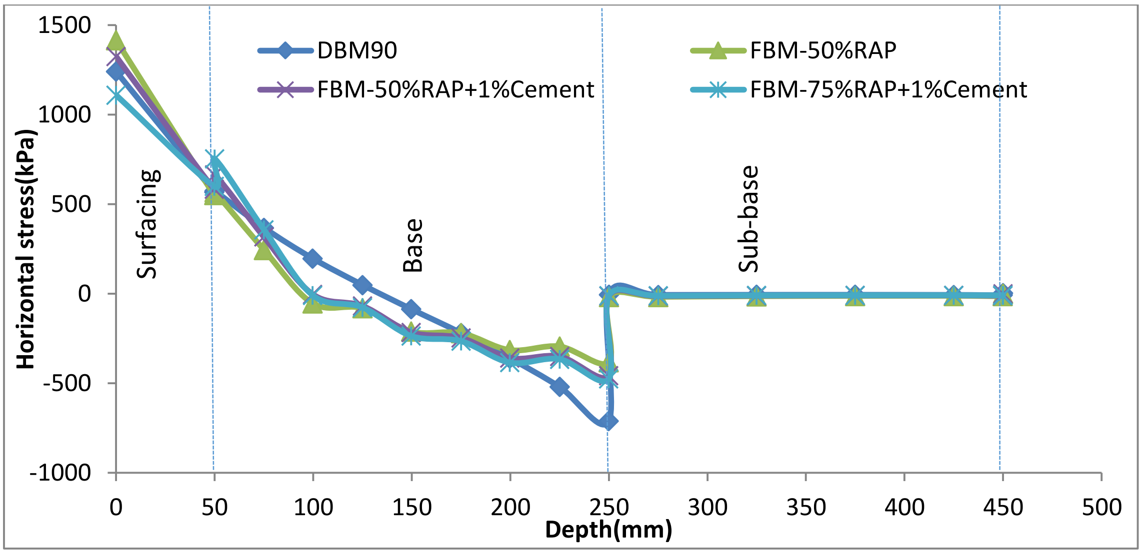


Figure 8-6 Horizontal Stress Distribution through Pavement Structure (Case 6)

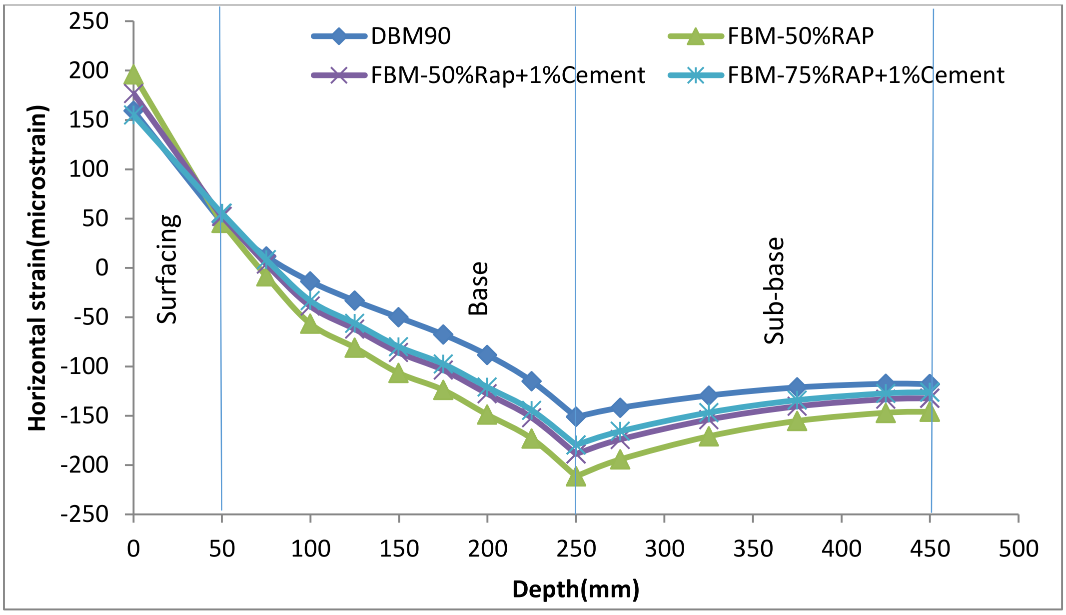


Figure 8-7 Horizontal Strain Distribution through Pavement Structure (Case 6)

8.5.3 Design charts

The results of the analysis discussed in the previous section using KENLAYER are used for generating the design charts. As seen in Figure 8-7, the maximum horizontal strain in the FBM base layer was found at the bottom of the layer. This maximum strain value is used for design purposes. The fatigue life of the structural layer was derived the fatigue equation obtained from the laboratory results (Table 8-3) and multiplying by the shift factor (400). The design charts are presented in Figure 8-8 to Figure 8-11. As can be seen in the figures a longer life was noted for pavement structures with DBM 90 base layer when compared with pavement structures with FBM base course of similar thickness. As expected, with an increase in base course thickness and surfacing thickness the fatigue life increased. However, with an increase in base course and surfacing thickness the difference between fatigue lives of pavements with DBM90 and FBMs with cement decreased. This trend could be attributed to the better performance of FBMs with cement (50%RAP+1%Cement and 75%RAP+1%Cement) at lower strain levels (Chapter 7).

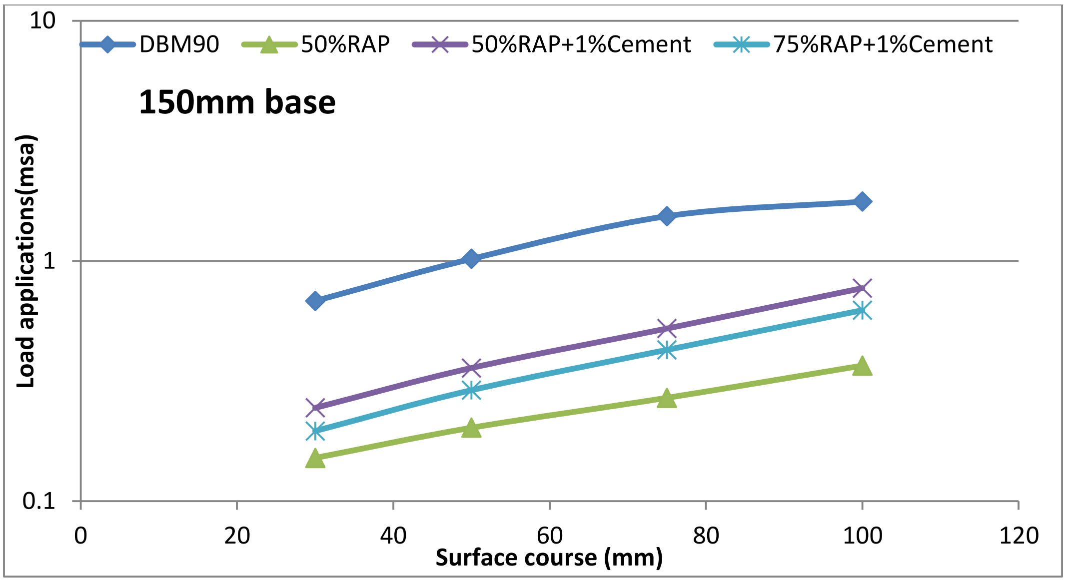


Figure 8-8 Design chart for 150mm base course pavement structure

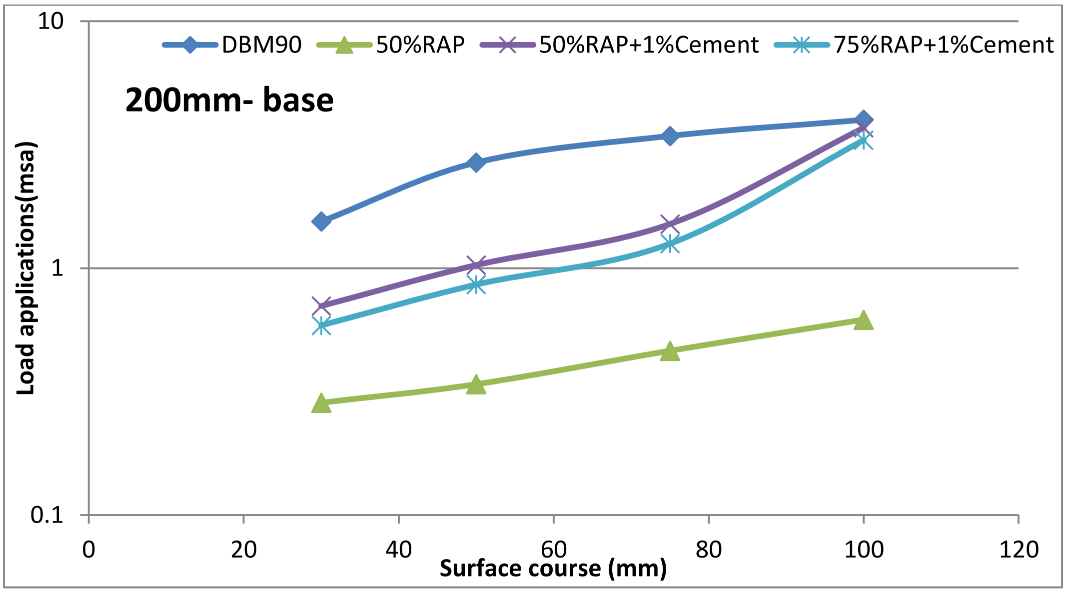


Figure 8-9 Design chart for 200mm base course pavement structure

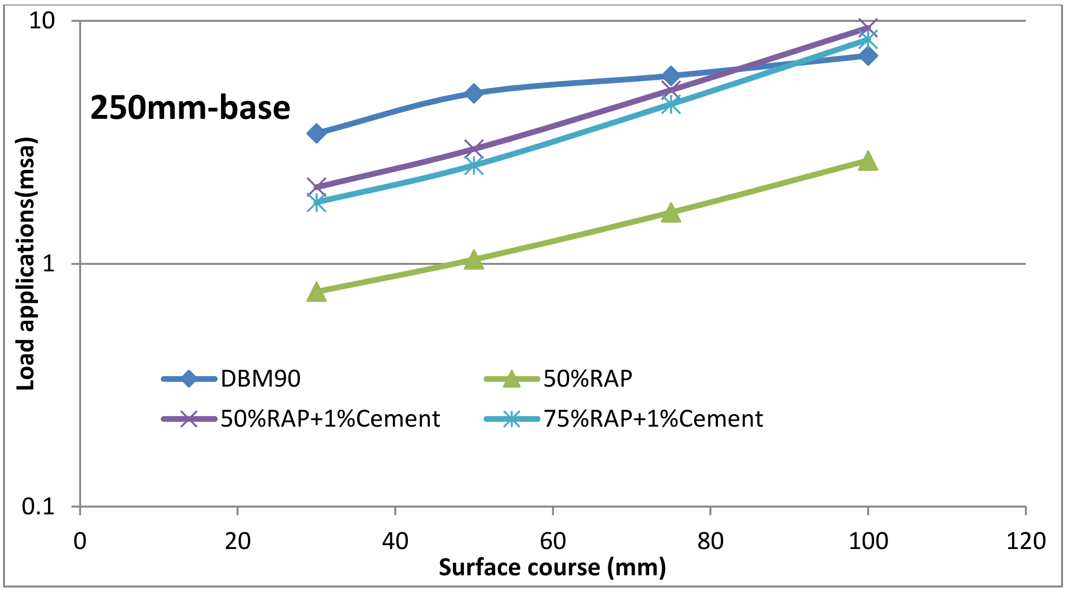


Figure 8-10 Design chart for 250mm base course pavement structure

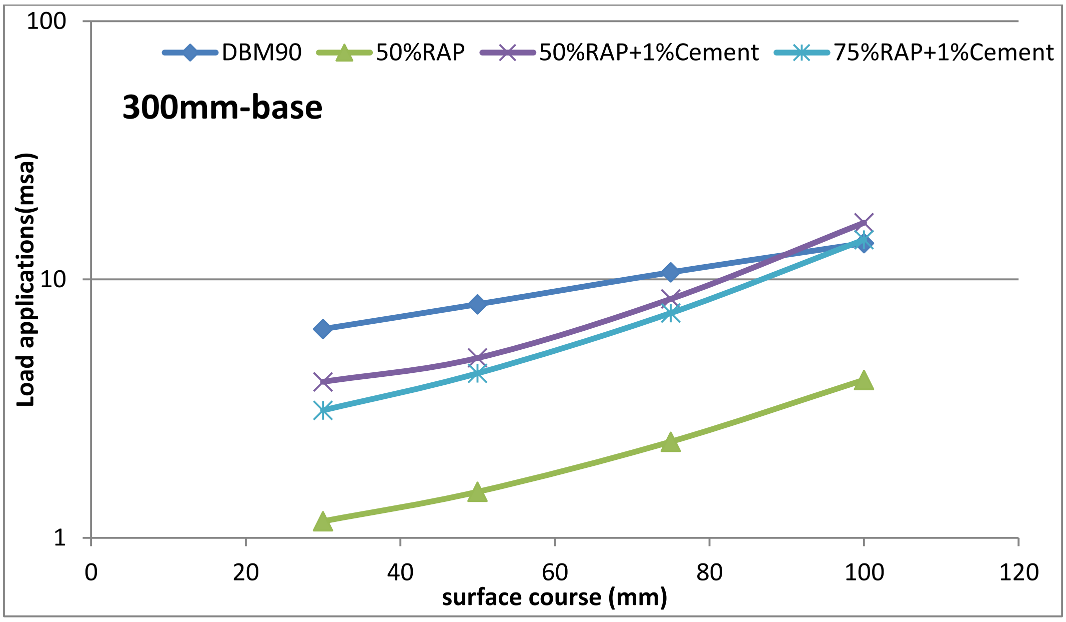


Figure 8-11 Design chart for 300mm base course pavement structure

8.6 Cumulative fatigue life approach

In the traditional pavement design approach, the fatigue life of a pavement layer is defined as the number of standard wheel loads it can sustain at a certain resulting strain level. The magnitude of strain is considered as the maximum permissible strain and is established by means of laboratory tests by which fatigue lines are established. For obtaining the design thickness of the pavement layers, layer thicknesses are adjusted until the strain criterion is satisfied. However if DBM90 was considered as the base course, the strain level would increase as stiffness decreases with load applications (Figure 8-12). Furthermore, it has to be noted that stiffness evolution is different if FBMs are considered as base course. In the traditional design approach the stiffness evolution is not taken into account. However such an assumption is not correct.

To account for the stiffness evolution an iterative approach recommended by Oliveira (2008) was considered to accommodate layer stiffness evolution with number of load applications in the design. For this the stiffness evolution curve (Figure 8-12) was divided into equal time intervals. Within each interval the average stiffness value was used as the input value for determining the strain level in the pavement which was assumed constant during that interval. This procedure is straightforward if linear elastic behaviour is assumed as stiffness values are inputted directly in to KENLAYER. However, since in this study non-linear elastic behaviour was assumed for FBMs and KENLAYER requires k1 and k2 as inputs, seeding stiffness evolution was carried out differently. For this, k1 was assumed to follow the stiffness evolution trend and k2 was assumed as constant throughout. For each strain level the corresponding fatigue life was estimated from the fatigue equation of each mixture. It has to be noted that a constant strain was assumed during the interval. However, since only a portion of the fatigue life is consumed during each interval, the cumulative fatigue life was calculated by adding the lives consumed during all the intervals. This iterative procedure approach is illustrated in the flow chart in Figure 8-13. Figure 8-14 presents the results of the analysis using KENLAYER. Figure 8-15 shows a comparison of design lives obtained by the two different design approaches.

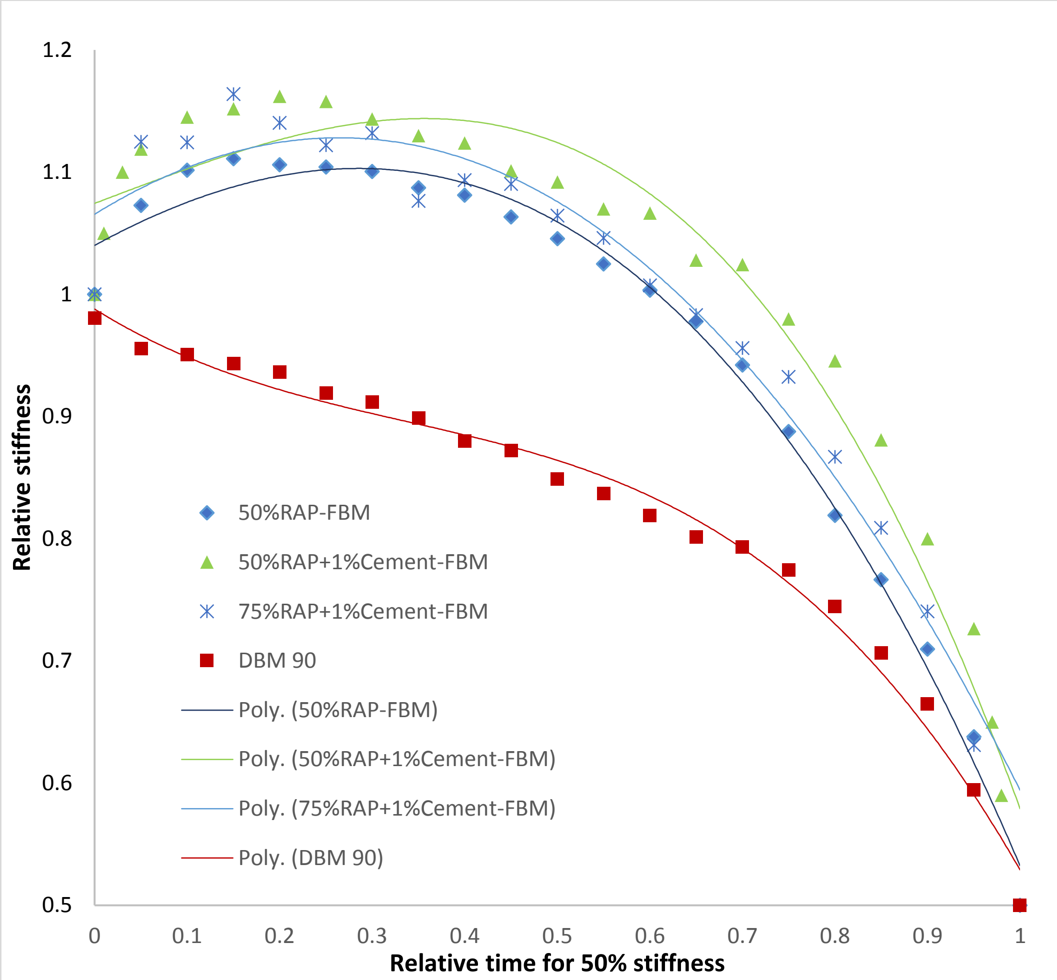


Figure 8-12 Stiffness evolution of the mixtures considered in the study (Form Chapter 7)

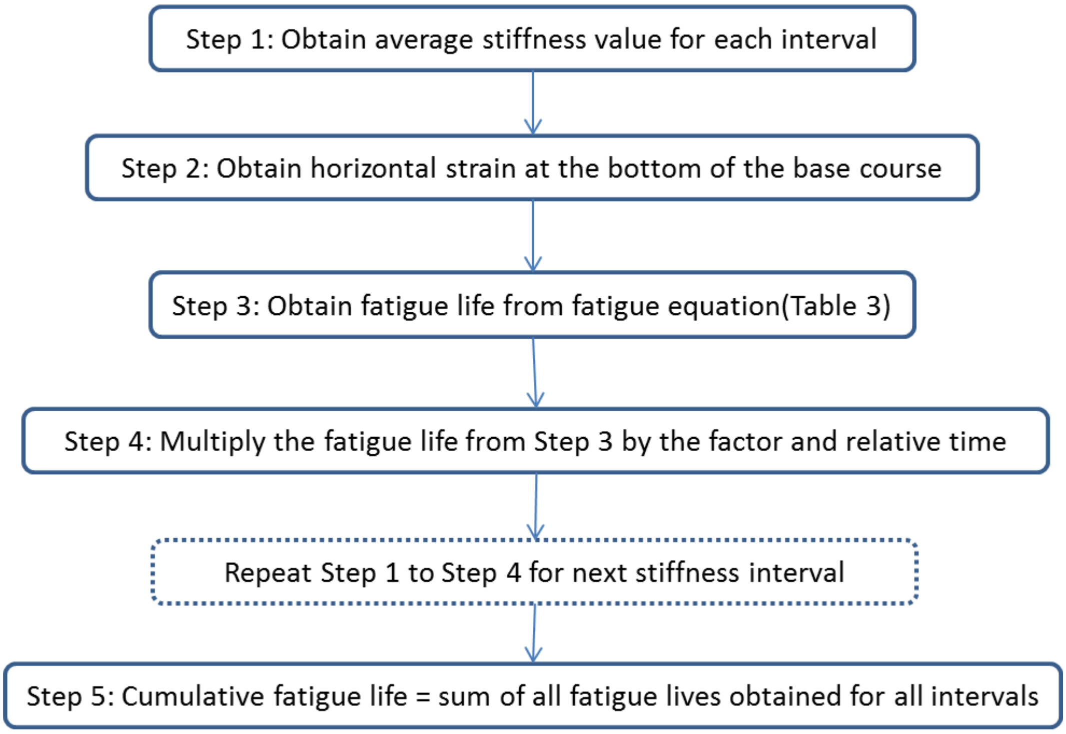


Figure 8-13 Flow chart for cumulative fatigue life approach (design)

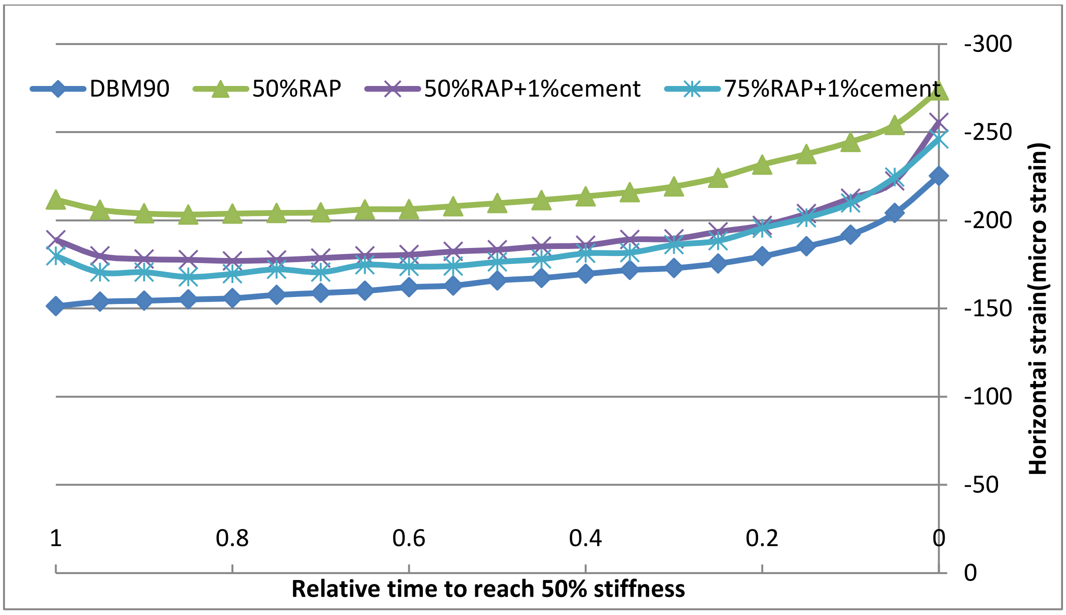


Figure 8-14 Horizontal strains (Tensile) obtained at the bottom of the base layer (Case 6)

Figure 8-15 shows a comparison of the traditional and iterative approaches carried out on Case 6 from Table 8-1. As the new iterative approach considers the stiffness evolution and stiffness of DBM 90 decreases with time, the procedure gives shorter fatigue lives than estimations made based on traditional approach. The traditional fatigue design calculations overestimate the fatigue life in the case of DBM 90. However, as seen in Figure 8-12 the stiffness evolution trend of FBMs does not always decrease as in the case of DBM 90. Therefore, in the case of FBMs, the new fatigue life approach does not have to result in shorter life than the traditional approach as in the case of DBM 90. Moreover, in the case of 50%RAP+1%Cement-FBM the new approach resulted in a longer life than the traditional approach, while the other FBMs were found to have nearly the same lives (Figure 8-15). Figure 8-16 shows the cumulative number of cycles obtained for the Case 6 pavement structure with different mixtures considered until 50% stiffness loss occurs in the base layer. Figure 8-17 shows the change in horizontal tensile strain at the bottom of the base layer with cumulative number of load applications. As expected stiffness and tensile strain decreased with increase in load applications for DBM90 mixture.

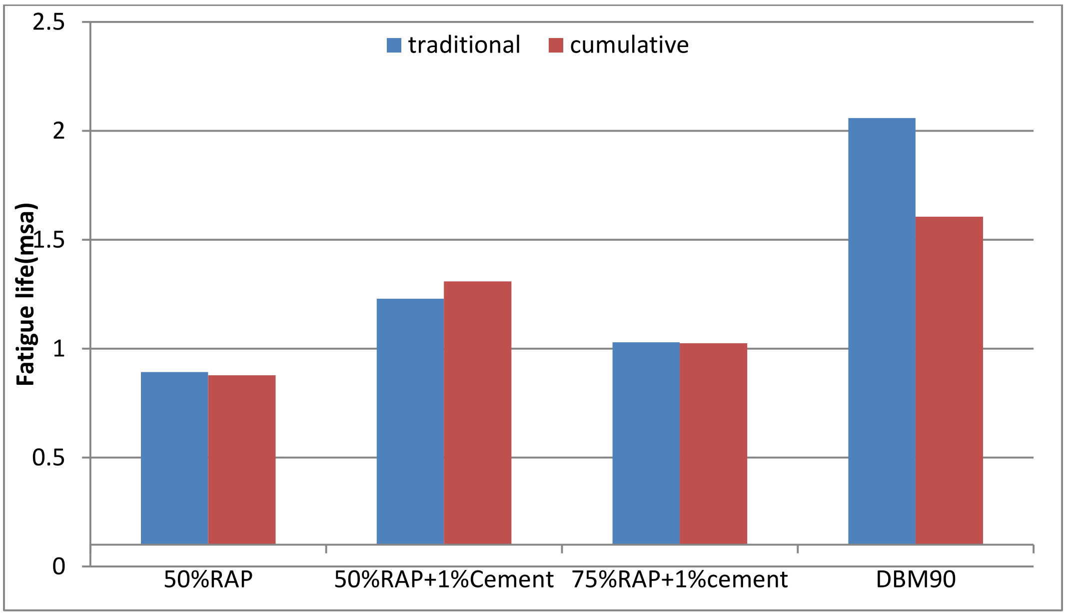


Figure 8-15 Traditional and cumulative fatigue life of mixtures considered in the study (Case 6)

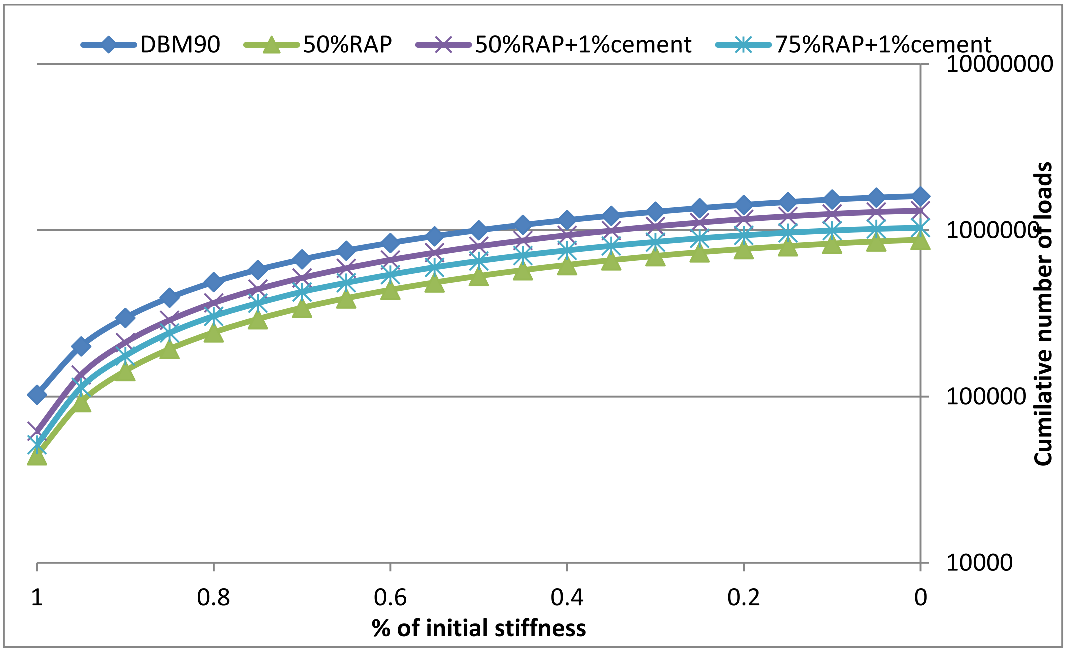


Figure 8-16 Cumulative fatigue life corresponding to stiffness evolution (Case 6)

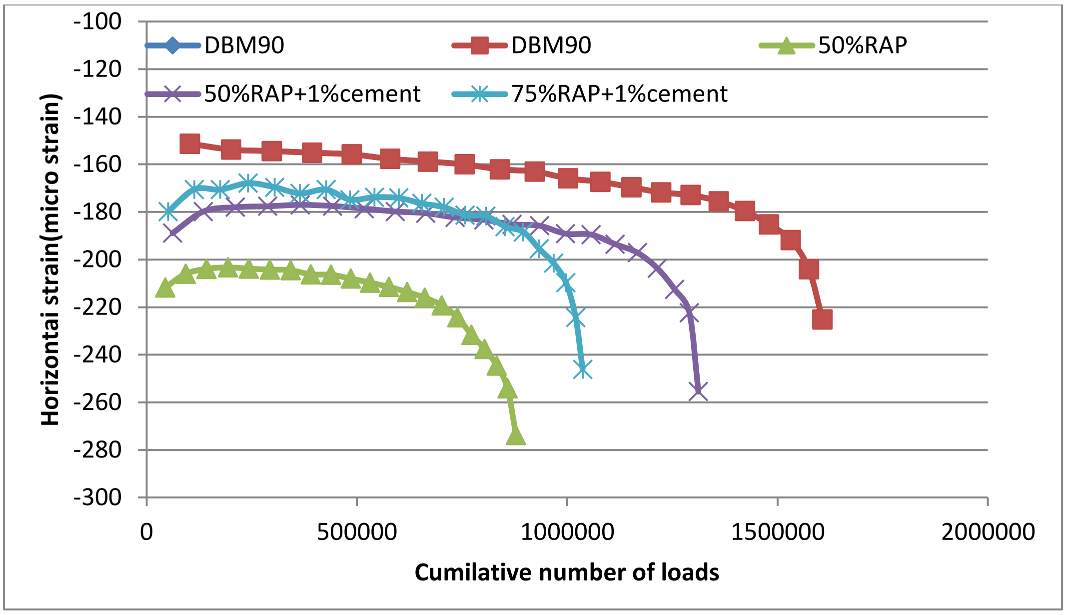


Figure 8-17 Horizontal tensile strain developed under base course with cumulative number of load cycles (Case 6)

8.7 Summary

A comparative analysis has been carried out in the present study using laboratory results obtained on both FBMs and conventional bituminous mixtures in order to evaluate the suitability of the laboratory models in the Mechanistic-Empirical design of pavements incorporating FBMs. Two different analytical approaches were considered for design based on fatigue criteria. The first was a traditional approach in which fatigue life of the base layer was calculated by computing horizontal tensile strain at the bottom of the layer and interpreting fatigue data in conventional way. Design charts were developed using this conventional approach for 16 different cases. The second approach, which is believed to be appropriate for FBMs because of their characteristic stiffness evolution, considers the actual stiffness evolution from the test data. This iterative approach was carried out on a purely comparative basis using a single pavement structure, and different mixtures were studied for that case. The results showed that the traditional design approach is satisfactory or even conservative for pavements with FBMs.

9 CONCLUSIONS AND RECOMMENDATIONS

In this chapter, a summary of the main issues discussed in each of the previous chapters is presented, with particular attention paid to the main conclusions obtained from the present project. Some recommendations for future research are also suggested at the end of the chapter.

9.1 Conclusions

The major outcome of the present research study was development of guidelines for mix design of FBMs in terms following parameters; Foamed Bitumen Content (FBC), Mixing Water Content (MWC), curing regime and compaction effort (gyratory compaction). The study also evaluated the importance of cement (or cementitious material) addition to FBMs. The presence of cement, as low as 1%, provides early stage stiffness to FBMs which is important for pavement that are trafficked immediately after placement of these mixtures. In addition to this, cement addition also reduce susceptibility of mechanical properties of FBMs to water. Another significant finding of the current research is applicability of maturity method to FBMs to assess the in-situ stiffness from the temperature history of the pavements with FBM layers. A detailed review of these findings is presented below.

A review of the mix design methods followed by different research and practicing agencies was discussed in Chapter 3. The review revealed the following conclusions.

- Except for the Asphalt Academy and Wirtgen, other mix design procedures propose a high limit for HL which seems to be difficult to achieve as half-life is very sensitive to the FWC. A half-life of 6 to 10 appears reasonable.
- Most of the agencies have not provided any recommendations regarding aggregate angularity. Though the Asphalt Academy mentioned aggregate angularity as an important factor it has not provided any recommendations.
- The maximum application rate of cement is usually less than that of the hydrated lime as recommended by the agencies considered in the present study. This is in view of the fact that though cement addition increases the initial stiffness, in the long term high cement addition reduces the flexibility of the FBM layers and thus the benefit of bitumen addition is rarely realised.
- Regarding the compaction method employed mixtures without added bitumen are recommended to be compacted using the Proctor (standard or modified) method whereas mixtures with added bitumen are recommended to be compacted with Marshall, gyratory or vibratory compaction.
- Most of the methods recommend a similar curing regime, which is conditioning unsealed specimens at 40°C for 3 days.

The physical properties of the materials used within this study are discussed in Chapter 4. Alongside bitumen and virgin aggregate, particular attention was given to reclaimed asphalt pavement (RAP) characterization.

- The tests on recovered aggregate and bitumen revealed that RAP is well within the homogeneity limits recommended by different agencies.
- A cohesion test on RAP revealed that the RAP used in the study can be classified as active which means it should not be treated as black rock.

The primary objective of the work discussed in Chapter 5 is to propose a practical and consistent mix design procedure for FBMs with the main focus being on the use of the gyratory compaction method in the proposed methodology. To attain this objective, mix design parameters such as Mixing Water Content (MWC) and compaction effort have been optimised. This mix design parametric study was initially carried out on FBMs with virgin limestone aggregate (VA) without Reclaimed Asphalt Pavement (RAP) material and a mix design procedure was proposed. The proposed methodology was later validated on FBMs with 50% RAP and 75% RAP. Efforts were also made to optimise the Foamed Bitumen (FB) content in FBMs with and without RAP. The following conclusion can be drawn from the Mix Design study discussed in the chapter.

- A rational range of 75-85% of Optimum Water Content (OWC) obtained by the modified Proctor test was found to be the optimum range of MWC that gives optimum mechanical properties for FBMs.
- OWC was also found to be slightly higher in the case of gyratory compaction when compared with modified Proctor compaction. This difference was thought to be due to the significant difference in the way the two compaction processes operate.
- It was found that a unique design number of gyrations, N_{design} (mixture specific) which is independent of the FB content can be established. N_{design} for the virgin mixture was found to be 140, while N_{design} for the mixtures with 50% of RAP and 75% of RAP was 110 and 100 respectively.
- Analysis of volumetric properties of FBMs revealed that the total fluid content, which has been successfully used in bitumen emulsion mix design is not a valid parameter in FBM mix design.
- It was also found that the presence of RAP influenced the design FB content, which means that treating RAP as black rock in FBM mix design is not appropriate.
- It was also found from limited study that the temperature of the aggregate during the mixing phase of the mix design influences significantly the quality of FBM. High temperature aggregates resulted in lower air voids in the resulting specimens. These higher densities (low air voids) could be associated with better compactability of the mixture at higher temperatures.

The study discussed in Chapter 6 was intended to better understand the curing mechanism of FBMs and to lessen the gap between laboratory curing and field evolution of these mixtures. This was achieved by evaluating different curing regimes that are being followed by different agencies and researchers, as well as identifying important parameters that affect curing. This curing regime study provided a valid investigation into the behaviour of FBM taking into account the effect of temperature, curing conditioning (Sealed or Unsealed), curing duration and the influence of cement with different curing regimes. The following inferences were drawn based on the experimental study performed.

- Curing temperature has a significant effect on the rate of curing and the 28-day strength and stiffness values. The results also suggest that 1% cement addition is an appropriate means of avoiding the effects of adverse weather during curing.
- The study also found that specifying in-situ water content as a criterion is not an appropriate approach as the same water content levels might give different stiffness values if cured at different temperatures. This leads to the suggestion that specifying in-situ stiffness could be a rational approach and a tool for estimating in-situ stiffness was developed using the maturity method.

The study in Chapter 6 also aimed to understand the curing mechanism and to interpret the level of impact that factors such as temperature, time, presence of RAP and active filler have on the curing of FBMs. The effect of these factors on curing was evaluated with reference to stiffness gain and water loss of specimens made of these mixtures. The following observations were made.

- The temperature is as important a parameter as time, as temperature has a significant influence on curing rate and also on bitumen properties.
- Higher curing temperatures resulted in a higher rate of stiffness gain. This trend is not only because of rapid water loss but also implies an increase in binder stiffness at higher curing temperatures.
- Though the presence of RAP improved the early stage stiffness of FBMs, it slowed down the rate of water loss from the specimens which resulted in smaller stiffness values at a later stage.
- The cement addition had no influence on water loss trends, but improved the stiffness significantly during all stages of curing.
- The results of curing cycle study indicate that reintroduction of water did not damage the bonds formed during curing cycle 1. Moreover, during cycle 2, the stiffness was found to be less sensitive to water content. This is particularly true in FBMs with cement in them.

The curing study in Chapter 6 also evaluated the applicability of the maturity method as a tool to assess the in-situ characteristic of FBM layers in the pavement. It was found that replacing the time term with an equivalent age term in the maturity function aided in estimating stiffness rather than relative stiffness. This was possible because of the characteristic curing of FBM in which the limiting stiffness that these mixtures reach strongly depends on the curing temperature at least for the length of curing stages considered in the present study. A strong correlation was found between maturity and the stiffness values obtained from the laboratory tests which resulted in development of maturity-stiffness relationships. The application of the method to assess the in-situ stiffness was presented using three hypothetical pavement sections. The results showed the influence of ambient temperature and the importance of cement addition to FBMs.

Chapter 7 discussed the performance characteristics of FBMs in terms of their permanent deformation and fatigue life. The performance indicators that were determined using the laboratory tests on FBM were also compared with conventional Hot Mix Asphalt (HMA) characteristics.

- The RLAT test results indicate that both test temperature and stress level have significant influence on permanent deformation characteristics as expected. The effect of stress on permanent deformation was increased with increase in test temperature.
- It was also found that from limited tests and mixture combinations, RAP content has only a slight influence on permanent deformation of FBMs. However, the presence of cement greatly improved the permanent deformation resistance of FBMs. Nevertheless the effect of cement decreased with increase in test temperature.
- FBMs were also found to be less temperature susceptible than HMA in terms of permanent deformation and, within FBMs, mixtures with cement were found to be more sensitive than FBMs without any cement.

For assessing the fatigue performance of FBMs, alongside the ITFT, the present study has also evaluated fatigue behaviour in terms of a uniaxial test in which compressive load is applied in the axial direction and indirect tensile strain was monitored in the radial direction.

- The ITFT test results showed that the FBMs without cement have lower fatigue life than conventional HMA at any initial strain level. Nevertheless, similar to permanent deformation, the fatigue life was improved with the addition of 1% cement to FBMs.
- The results indicate that 3% cement addition greatly reduced the flexibility of FBMs and thus the mixtures showed relatively lower fatigue life at a given strain level.

- From uniaxial tests, stiffness evolution curves showed that FBMs fail in a different pattern compared to HMA. Unlike HMA, which showed a three stage evolution process, for FBMs the stiffness increased initially to reach a maximum and decreased at a slower rate until failure. It was also found that by plotting curves according to Hopman et al., which identifies the fatigue failure transition point, use of the 50% stiffness criterion for fatigue life evaluation is not a conservative approach.

- Uniaxial tests also revealed that, although in fatigue the FBMs were found to behave differently from HMA, in terms of permanent deformation, FBMs behave similarly to HMA in that a steady state strain rate was achieved.

Chapter 8 examines the effect of the FBMs considered in this study on pavement response. 16 pavement structures chosen for sensitivity analysis. Non-linear elastic analysis was used to compute the mechanistic response (stresses and strains) analysis of both the FBM base and the granular sub-base of the pavement structures considered. Two different design methodologies were tested to assess the implications of applying the analysed pavement design methods on pavement layers with FBMs. The first is a traditional approach for conventional flexible pavements which is based on pavement life as a function of computed tensile strain in the material and interpreting fatigue data in a conventional way. The second approach is believed to be more appropriate for FBMs because of their characteristic behaviour noticed in Chapter 7. This new method is an iterative approach which considers the actual stiffness evolution of the mixture.

9.2 Recommendations for Future Research

In the present study, an investigation into some of the issues of FBMs has been conducted. However, FBMs are complex materials because of the diversity of the parameters that need to be accounted for. There is plenty of scope to fully understand the behaviour of FBMs and to predict the performance as accurately as possible. In this section along with scope of the present study recommendations for future study were made.

- Although it was identified in the present and past studies that many parameters influence the performance of FBMs, however, the scope of the present study was limited to a single aggregate source, single aggregate gradation, one type of bitumen and mixer. Acknowledging that past literature is available which studied the influence of these parameters on mechanical properties, it is recommended to further study the level of impact these parameters have on the performance characteristics.

- Preliminary studies also revealed that binder distribution across the aggregate phase depends upon the aggregate size, becoming less homogenous when larger aggregate size is used. It is also believed that there is a strong relation between aggregate size and amount fines required in the mixture. Therefore, it is recommended in future study to explore this relation.

- As seen in Chapter 5, the high temperature aggregates resulted in lower air voids in the resulting specimens and better mechanical properties. This supports the concept of half-warm mixes using foamed bitumen in which mixtures are prepared with half-warm (well below 100°C) aggregates. Therefore, it is recommended in future study to study the mechanical properties and performance characteristics of half-warm mixtures.

- Most of the present study was carried out on mixtures with 1% of cement. As discussed in Chapter 5, 6 and 7 this 1% cement addition to FBMs improved mechanical properties and performance characteristics. It was also shown in Chapter 7 that addition of 3% cement significantly reduced the flexibility offered by bitumen and decreased the fatigue life. It will be interesting to add in future work to determine whether there exists an optimum cement content to be added such that flexibility of FBMs is still acceptable and which results in optimal performance characteristics, particularly fatigue life.

- The scope of the present study was limited to use of cement as active filler in the FBMs considered in the study. It is recommended to carry out a study with other active fillers such as hydrated lime.

- In the present study it was assumed that cement addition does not influence the optimum bitumen content. It is worth verifying whether the presence of cement in FBMs affects the optimum bitumen content. The other major assumptions while optimising the bitumen content were that curing time and curing temperature do not influence the optimum bitumen content. Therefore, it is advised to study the effect of curing temperature and curing period on the optimisation of mechanical properties with reference to bitumen content.

- In addition to the temperature and time of curing, relative humidity is an important parameter that influences the curing. In the present study the effect of humidity has usually been neglected. However this assumption is not valid as high humidity could slow down the curing process significantly. Therefore it is recommended to consider adding relative humidity as a parameter in the curing regime. Moreover, the maturity functions that were developed in the study did not integrate the effect of relative humidity on the stiffness evolution. Therefore, it is recommended to incorporate the relative humidity parameter in such functions in future study.

- In determining the effect of temperature on bitumen properties so as to estimate its influence on curing, bitumen was extracted from the FBMs. From limited data it was shown that curing temperature has considerable effect on the extracted bitumen properties. However, more tests might be needed to confirm this. Micro-structural observation of these mixtures using image analysis is the best available option.

- A major difficulty with fatigue testing is developing a relevant relationship between the results of laboratory tests and field performance. Laboratory tests usually 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. For these to be considered in structural analysis and design multipliers known as shift factors (or transfer functions) should be applied. These shift factors were sensibly assumed in the present study as determination of these shift factors is not in the scope of the study. Therefore, it will be useful to include experimental work to determine correct shift factors in future study.

- In Chapter 7 structural design was carried assuming the FBMs as non-linear elastic, although these mixtures are viscoelastic in nature because of the presence of bitumen in them. However, simulating non-linear viscoelastic behaviour is complex and requires sophisticated tools such as Finite Element Methods. An analysis of the response of these materials more accurately using these tools is recommended in further research.
- In the present study only fatigue criteria was considered for structural design. However, it is important to consider permanent deformation criteria in the design as the pavements with FBM layer are usually susceptible to rutting. Therefore, it is critical to consider permanent deformation criteria in future structural design.

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