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Faculty of Engineering

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Mix Design, Mechanical Analysis and Performance Modelling of Warm Mix Asphalt

By

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بِسْمِ اللَّـهِ الرَّحْمَـٰنِ الرَّحِيمِ

{ اقْرَأْ بِاسْمِ رَبِّكَ الَّذِي خَلَقَ * خَلَقَ الْإِنْسَانَ مِنْ عَلَقٍ * اقْرَأْ وَرَبُّكَ الْأَكْرَمُ * الَّذِي عَلَّمَ بِالْقَلَمِ * عَلَّمَ الْإِنْسَانَ مَا لَمْ يَعْلَمْ }

صدَقَ اللّه العَظِيم

سورة العلق: 1 - 5

In the Name of Allah, the Most Beneficent, the Most Merciful

*{Recite in the name of your Lord who created * Created man from a clinging substance * Recite, and your Lord is the most Generous * Who taught by the pen * Taught man that which he knew not}*

Allah Almighty has spoken the truth

Surat Al-Alaq (The Clot): 1-5

Abstract

The asphalt industry has been under pressure to reduce its environmental footprint and energy demands to meet current and future sustainability requirements. Warm Mix Asphalt (WMA) has been developed to reduce the environmental impact of asphalt by reducing its production temperatures from the hot range (i.e. ≥160°C) to the warm range (i.e. 100-140°C). The common idea of most WMA techniques is to use additives in order to either reduce bitumen viscosity or reduce bitumen surface tension, which enhances aggregate coating and eventually enables asphalt mixing at reduced temperatures. A robust method to determine a rational reduction in the production temperatures of asphalt, however, is not available so far; but some methods that are thought to be valid exist for certain additives such as the equi-viscous approach. Also, in the case of incorporating Reclaimed Asphalt Pavement (RAP) with WMA (WMRA), it is uncertain what production conditions are suitable for this mix and how best to design these conditions to ensure acceptable performance. Accordingly, the main aims of this research were to understand and analyse the impact of selected WMA additives on bitumen and asphalt properties, develop a rational method to design optimum production temperatures of WMA and WMRA, and deeply investigate the behaviour of these mixtures concerning performance and durability.

Different materials were involved in the study including two of the commonly used WMA additives, a chemical and a wax additive; a 50-60 penetration grade bitumen was adopted as a reference binder and used to produce control Hot Mix Asphalt (HMA); one source of RAP was used at a level of 50%. The methodology followed to achieve the aims of this study included different bitumen and mixture tests; developing an image processing technique to quantify aggregate coating and investigate additive impact on this property; develop a method to design production conditions for WMRA; investigate the durability of WMA and WMRA in terms of ageing and moisture damage resistance; modelling of these mixes to predict their mechanical performance in terms of rutting, top-down and bottom-up cracking.

The results of the bitumen investigation revealed that WMA modified binders should be characterised based on bitumen performance beyond the limits of linear viscoelasticity rather than based on the empirical or Superpave parameters.

Aggregate coating quantification results showed that WMA mixing temperatures can be decreased by about 20-30°C although additional mixing time was required to achieve full coating. However, this method must be integrated with performance results since some WMA additives, such as wax, have a significant relationship between their performance and mixing temperatures. Furthermore, based on limited compactability results, expressed as the compaction effort required by a roller compactor to compact WMA to a target density, it was concluded that the additives used both have a positive influence on enhancing asphalt compactability.

In the case of WMRA, it was discovered that the performance of WMRA significantly depends on the degree of blending (DoB) between the aged and soft binders. The DoB is found to be a function of mixing temperature and mixing time. Despite it being possible to achieve a DoB of about 96% in this study, the remaining unblended soft binder can be a source of weakness as results of WMRA revealed that this mix was susceptible to rutting at temperatures beyond the critical high temperature of the soft binder. Accordingly, the grade of the soft binder should be carefully selected, and the production conditions should be accurately designed to assure acceptable performance of this kind of asphalt.

Performance of WMA and WMRA was simulated based on the Mechanistic-Empirical Pavement Design Guide (MEPDG) principles. Rutting model parameters were determined in an innovative way from the repeated load axial test results; fatigue cracking model parameters were determined from the two-point bending results. A typical four-layer asphalt pavement was simulated; the simulation results showed that asphalt performance is controlled by two factors, the applied strain level and its mechanical response properties. The results also indicated that some mixtures can perform better than others when used as either surface or base layers. Accordingly, this analysis can be used in predicting pavement performance with a particular mix and selecting the layer in which it can perform the best.

Keywords:

Bitumen rheology; Mixing temperatures; Aggregate coating; RAP; Asphalt performance; Durability; Performance modelling.

Dedication

To the pure soles of my father, brother and sister who would be so happy to see this dream come true...

To my kind-hearted mother, and my dear brothers and sisters...

To my beloved wife and lovely children...

Acknowledgement

First of all, all praise and gratitude are to Almighty Allah for making everything possible and for giving me the strength, health, and patience during this research.

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Declaration

The research described in this PhD thesis was performed at Nottingham Transportation Engineering Centre, Department of Civil engineering, University of Nottingham between October 2015 and September 2019. I hereby declare that this work is my own and has not been submitted to any other university for a degree.

Ahmed Abed

July 2019

University of Nottingham

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Chapter 1: Introduction

1.1 Background

The increasing understanding of the negative and dangerous impacts of climatic changes, global warming, and environmental pollution has forced all industries to reconsider their production methods to minimise greenhouse gas emissions (GHG). With that in mind, the asphalt industry is required to reduce its environmental burden because it is a significant CO_2 emission contributor due to energy requirements during the production phase of this material. At present, Hot Mix Asphalt (HMA) has been the dominant type of asphalt produced. HMA production passes through several stages, including aggregate drying and heating, heating of bitumen, and mixing these ingredients at elevated temperatures. Mixing temperatures depend on the grade of the binder; for most unmodified paving grade bitumen, it is about 165°C (Vidal et al., 2013). At these elevated temperatures, a significant amount of energy is consumed, and substantial quantities of GHG and pollutants are emitted into the air (Capitão et al., 2012). The amount of CO_2 emissions depends on the source of energy used to produce asphalt; it varies between 11.7-33.3 kg/ton of asphalt, whereas the fuel consumption is between 4.7-5.18 kg/ton based on the fuel type (Almeida-Costa and Benta, 2016). In response to these drawbacks with the HMA industry, Warm Mix Asphalt (WMA) was developed.

WMA is the technical term used to describe asphalt mixtures produced at temperatures 10-40°C lower than equivalent HMA (EAPA, 2010); or it can be defined as an asphalt mixture produced between 100-140°C (Prowell et al., 2012). Both definitions refer to the main property of WMA, which is the production of asphalt at temperatures lower than that of the traditional HMA. WMA was developed in the middle of the nineties of the last century after signing the Kyoto protocol, which requires the developed countries in Europe and the USA to reduce carbon footprints (UNFCCC, 2008). WMA can be produced by three broad technologies: organic additives, chemical additives, or foaming techniques. The first method works by modifying bitumen by wax additives that reduce bitumen viscosity, which allows for reducing the production temperatures of asphalt. The second one incorporates chemical additives that reduce the bitumen surface tension and friction between binder and aggregate, which allows for lowering mixing and compaction temperatures of asphalt. The foaming techniques work by temporarily changing the bitumen state from liquid into foam during asphalt mixing, which reduces the effective viscosity of bitumen. The decreased bitumen viscosity allows for producing asphalt at reduced temperatures. It can be realised that the primary aim of all of these methods is decreasing production temperatures of asphalt to enhance the sustainability of the asphalt industry.

Reducing the environmental impact of the asphalt industry and developing a greener and a more sustainable alternative to HMA was the ultimate aim of WMA development. Since the development of this type of mixture, it has received considerable interest from the asphalt industry and highway agencies due to the expected economic and environmental benefits of this mixture over the traditional HMA. The literature documents WMA benefits thoroughly; the benefits vary from direct to indirect. The direct benefits include a reduction in the greenhouse gas emissions and fuel consumption resulting from reducing production temperatures of asphalt. The indirect benefits include the ability to pave in cooler temperatures due to the enhanced workability of asphalt, extending the paving season due to the increased capability to pave in colder weather, and less exposure to fumes at asphalt plants.

Despite the promising benefits of WMA, this mixture must exhibit comparable performance to the conventional HMA. WMA must perform equally to HMA regarding all performance measures, including permanent deformation, fatigue cracking, and durability before it can be accepted and implemented as a successful replacement for the traditional HMA.

1.2 Problem Statement

Asphalt production is a complicated thermodynamic process involves heating the raw materials, mixing, laying, and compaction. During these processes, the temperature must be maintained at certain levels to ensure full aggregate coating during mixing, and sufficient workability during asphalt laying and compaction to ensure proper compaction to the target density. For most paving grade (unmodified) bitumen, the mixing temperature is around 160±5°C, whereas the optimum compaction temperature is about 10°C lower than the mixing temperature. Several studies have stated that at these elevated temperatures, the GHG emissions are critical.

In the UK alone, the average annual production of asphalt is approximately 21 million ton/year (EAPA, 2015). The vast majority of this tremendous amount of asphalt is produced following the hot production method, which means that the annual carbon footprint of the asphalt industry in the UK could be approximately 231-693 thousand tons depending on the fuel type used in asphalt plants. This critical environmental impact must be reduced if a greener and environmentally friendlier asphalt production is to be achieved. Thus, the asphalt industry is under pressure and demand to reduce its carbon footprint and to reshape itself to satisfy sustainability requirements.

One of the possible alternatives to end this problem is the use of WMA technologies because they allow for asphalt production at temperatures lower than HMA, which can help in reducing GHG from asphalt. However, the additive mechanism to reduce the production temperatures is unclear. Although this type of mixture has been under investigation for the past 15 years, so far, there are no standards to design the production temperatures of WMA. Manufacturer's recommendations are the primary source of information to select production temperatures of WMA. Accordingly, investigating and understanding the effect of the additives on mixing and compaction temperatures is a necessity.

Furthermore, WMA performance must be proven to be comparable to that of HMA; otherwise, it cannot be accepted. But WMA may always be different from HMA due to several factors, including: differences in the production temperatures which make WMA subjected to less ageing during production, effects of the additives on bitumen rheological characteristics which may change viscoelastic properties of the bitumen, impacts of the additives on bitumen aging properties which may slow down this phenomenon leading to increased risk of permanent deformation, influences of the additives on the bitumen cohesion or aggregate-bitumen adhesion forces which may improve or weaken the resistance of asphalt to specific distress types.

Additionally, asphalt recycling has been proven to be an economical and environmentally friendly approach to enhance the sustainability of the asphalt industry. Asphalt recycling is relatively successfully performed within the hot production temperature range. If the WMA technologies fail to incorporate recycled asphalt pavement (RAP), then these technologies would probably be ignored, because RAP is a valuable material that can save natural resources of aggregate and bitumen and reduce the production cost of asphalt. Therefore, any asphalt production method that fails to incorporate RAP may not be implemented. Correspondingly, asphalt recycling within the WMA temperature production range must be investigated to verify the applicability and practicality of that kind of asphalt mixture.

Reducing production temperatures of asphalt by using WMA additives is likely to affect different aspects of an asphalt mixture, including aggregate coating, mix workability and performance. Incorporating RAP into WMA increases the complexity of the design because of the technical problems associated with asphalt recycling, including blending quality between the aged and fresh binders, and effects of RAP on mix performance. Accordingly, it is critical to study the implications of temperature reduction on these aspects. Hence this study is conducted to cover that gap in asphalt industry understanding.

1.3 Aim and Objectives of the Study

The primary aim of this study was to investigate, design, and analyse WMA and Warm Mix Recycled Asphalt (WMRA) mixtures produced at reduced production temperatures. Thus, mechanical properties of WMA and WMRA produced at different production conditions would be analysed in comparison with reference HMA, to detect effects of the production conditions on mixture properties accompanied by scientific and fundamental reasons for such effects. To achieve this aim, several tasks must be fulfilled, as follows:

1. The first objective was to study and analyse the effect of the additives on bitumen characteristics and rheological properties. The goal of this objective was to understand additive effects on bitumen properties and utilise this understanding in explaining and analysing WMA performance. Two commonly used WMA additives, namely Sasobit and Cecabase, were used in this step.

2. Study and analyse mechanical properties of WMA and WMRA produced at different production conditions in order to monitor and understand the relationship between production temperatures and mix performance.

3. Provide a method to quantify aggregate coating during asphalt mixing. WMA additives are supposed to enhance aggregate coating when mixing temperatures are reduced. Therefore, a logical way to prove this impact is by quantifying aggregate coating.

4. Correlate the production temperature effects on mechanical properties of the studied mixtures with the aim to use this relationship to select an optimum production temperature that ensures comparable performance to the control HMA while the production temperature is kept to the minimum.

5. Assess the performance of the studied mixtures produced at the recommended temperatures determined in step four in terms of rutting, fatigue cracking, and durability to fully validate the applicability of those kinds of asphalt.

6. Build models to predict the mechanical performance of the studied mixtures. This step aims to simulate and predict rutting and fatigue cracking performance of the mixes when used in asphalt pavement construction.

1.4 Research Methodology

The methodology followed in this study involved conducting an intensive experimental program at binder and mixture levels to provide the necessary data to analyse the behaviour of the investigated asphalt mixtures and provide analysis methods that can be applied to achieve the aim and objectives of this study. The methodology is detailed as follows:

1. Perform a comprehensive literature review on WMA and WMRA production technologies to understand properties, advantages, and disadvantages of these methods; also, to identify any gaps that have not been covered or well understood in the literature. The general conclusion of this step was that there was no global method to balance production temperature reduction and WMA/ WMRA performance. Also, the literature review identified one of the main problems associated with WMRA mixture, which is the blending quality between rejuvenator binder and RAP binder. Furthermore, in many countries including the UK, these mixtures are still at the state-of-the-art stage, and there are no standards to govern the production of these mixtures, except guidelines established by highway agencies and recommendations from WMA additive manufacturers. With this in mind, the methodology was designed to cover these gaps and achieve the aim and objectives of this study.

2. Investigate and understand the effects of the selected additives on the base bitumen characteristics. Asphalt performance is affected to a significant extent by the properties of the binder, so any attempt to analyse and

understand asphalt behaviour should be accompanied by a comprehensive understanding of the properties of the base bitumen.

3. Produce WMA samples at different reduced temperatures and analyse their mechanical properties in order to study effects of reducing production temperatures on mix properties. In this study, the selected mechanical properties were Indirect Tensile Stiffness Modulus (ITSM) and Indirect Tensile Strength (ITS) since these measures provide fundamental information about the performance in a cost-effective and time-effective matter. So, by correlating these measures with the production temperatures and comparing the results with the control mix, any drop in the performance can be determined and accordingly, the production temperatures can be selected.

4. Quantify aggregate coating to detect the influence of the additives on this property. In this study, the coating was quantified by the application of image processing principles on images taken during the asphalt mixing process. This method provides a direct evaluation of WMA additive impact on aggregate coating. Accordingly, the relationship between aggregate coating versus production temperatures can be derived by analysing aggregate coating of mixtures produced under different production conditions.

5. Use the results from the aggregate coating study and mechanical properties against time study to optimise production temperatures of WMA. The optimised production temperature is the minimum mixing temperature at which WMA performs similarly to HMA in terms of mechanical properties

and aggregate coating. In other words, the lowest production temperature at which WMA shows comparable performance to HMA in terms of ITSM, ITS and aggregate coating can be selected as an optimum production temperature.

6. Investigate the blending quality between RAP binder and virgin bitumen of WMRA mixtures produced at different mixing temperatures. The Degree of Blending (DoB) between RAP and virgin binders is an important parameter that affects homogeneity and performance of that kind of asphalt. Therefore, it has to be carefully studied and the factors that have a significant influence on this property and mix performance as well, such as mixing temperature and mixing time have to be identified and optimised.

7. Study and analyse performance of WMA and WMRA samples produced at the optimised production conditions in terms of the primary performance measures; these are dynamic modulus, permanent deformation, fatigue cracking, and durability in order to fully validate the applicability of using that kind of asphalt mixture in real-life applications.

8. Derive permanent deformation and fatigue cracking models for the studied mixtures from the rutting and fatigue cracking data in order to use the models in the prediction of WMA and WMRA pavement performance.

9. Develop performance prediction models for the studied mixtures. By generating dynamic modulus master curves from the dynamic modulus testing and by implementing the derived rutting and fatigue laws generated from the rutting and fatigue data, and following the MEPDG methodology, pavement performance can be predicted.

1.5 Novelty and Contribution

This study was conducted to enhance an essential criterion of modern asphalt requirements, which is the sustainability of the asphalt industry. Several novel techniques were utilised to achieve the aims and objectives of this study. Firstly, the use of the image processing technique to quantify the aggregate coating process was a novel approach innovated in this study. This approach enables the quantification of the aggregate coating when mixing asphalt at different temperatures. Also, it enables the detection of WMA additive effects on this property, which was significantly useful in validating the advantages of the additives in that area.

The other novel method followed in this study was the correlation between mixing temperatures and WMA/ WMRA performance indicators. This step was critical to understand the relationship between performance and mixing temperatures and using this relationship in determining suitable mixing temperatures of the studied mixtures. Moreover, this step led to the detection of the significant and unique relationship between mixing temperatures of WMA produced using Sasobit and ITSM/ ITS measures. This relationship provided conclusive evidence for the selection of optimum mixing temperatures for that kind of asphalt. This step also showed the necessity to find an alternative method to determine a production temperature of WMA produced using the chemical additive because this mixture did not exhibit any dependency on mixing temperature, which led to the conclusion that a direct quantification of aggregate coating was required.

The other innovative aspect developed in this study was the design and analysis approach applied to quantify the DoB of WMRA and to detect and understand the relationship between this property and mixing temperatures, mixing time, mix homogeneity, and mix performance in terms of ITSM and ITS. The developed analysis can sufficiently determine DoB and can efficiently be utilised to optimise production conditions to maximise the DoB. Moreover, this analysis showed the applicability of recycling up to 50% RAP within the warm production temperature range by designing and optimising the production conditions of that kind of asphalt.

Moreover, the developed method to predict the performance of the studied mixtures is innovative. Linking Kenlayer to Matlab to determine pavement response then using the response together with rutting and fatigue cracking laws to predict pavement performance is a new approach that can be utilised in predicting the performance of similar unconventional mixtures based on actual material properties. Furthermore, determining the MEPDG rutting model parameters from RLAT results and using the model in the prediction of permanent deformation is a novel method that has not been found in the literature. This method appears efficient and reliable, but it still needs to be validated and calibrated before it can be implemented in pavement design problems.

The contribution of this research will have a definite impact on the sustainability of asphalt engineering. Lowering mixing temperatures without compromising mix performance decreases greenhouse gas emissions and saves production energy and combining asphalt recycling with WMA techniques significantly reduces production cost by reusing existing materials and saving natural resources of aggregate and bitumen. Accordingly, this study can be considered as a step forward towards a greener, environmentally friendlier, and more cost-effective asphalt industry.

1.6 Publications

This PhD has been published in three journal articles and two international conferences, as follows:

- Abed, A., Thom, N. and Grenfell, J., 2019. A novel approach for rational determination of warm mix asphalt production temperatures. *Construction and Building Materials*, *200*, pp.80-93.
- Abed, A., N. Thom, and D. Lo Presti, Design considerations of high RAP-content asphalt produced at reduced temperatures. *Materials and Structures*, 2018. **51**(4).
- Abed, A., N. Thom, and J. Grenfell, *Evaluation of Mixing Temperature Impact on Warm Mix Asphalt Performance*, in *Tenth International Conference on the Bearing Capacity of Roads, Railways and Airfields*. 2017: Athens, Greece.

1.7 Thesis Structure

This thesis consists of eight chapters, detailed as follows:

Chapter one presents a brief introduction to the topic of the study and explains the necessity of conducting this study. It also shows the aim and objectives of the study and the methodology followed to achieve these targets.

Chapter two provides an overview regarding the background of WMA technologies and the advantages and disadvantages of these technologies. It describes and evaluates the available methods to reduce production temperatures of WMA. It also introduces the available methods and technologies regarding asphalt recycling; technical issues and problems with the mechanical performance of that mixture are also discussed.

Chapter three introduces and describes the properties of WMA additives used in this study. It also deeply investigates the effects of these additives on bitumen empirical and rheological properties. The influences of the additives on bitumen rutting and fatigue cracking performance are also studied in this chapter.

Chapter four introduces the methodology followed to design WMA and to optimise mixing temperatures of that type of mix. It presents the image processing methodology applied to quantify aggregate coating and to detect impacts of WMA additives on this property. It also provides preliminary performance results in terms of stiffness and strength.

Chapter five presents bitumen design and mix design of WMRA. Binder blending laws are investigated and analysed, and validation of binder blending results is introduced. Then the design of WMRA based on the binder design study is presented, and the methodology followed to quantify the degree of blending between RAP and soft bitumen at mixture level is detailed.

Chapter six investigates performance and mechanical properties of the designed mixtures produced at the optimised conditions determined in chapter four and five. The properties investigated in this chapter are dynamic modulus, permanent deformation, fatigue cracking, and durability.

Chapter seven introduces mechanistic-empirical pavement performance prediction of the design mixtures using material properties acquired in this study and the mechanistic-empirical pavement design guide models.

Chapter eight focuses the light on the main conclusions drawn based on this study. It also provides recommendations to cover areas related to WMA and WMRA mixtures that need to be further investigated in order to expand the knowledge regarding that kind of asphaltic mixture.

Chapter 2: Literature Review

2.1 Introduction

Asphalt is a construction material consisting of granular aggregate and bitumen (Thom, 2014). The aggregate represents the structural skeleton of this material that resists the external traffic loading, whereas the bitumen is the binder that glues aggregate particles together. The production of this material involves: heating bitumen, drying and heating aggregate, mixing, transport, laying, and compaction (Hunter et al., 2015). Production temperatures of this material vary depending on the technology applied and the characteristics of the ingredients. Accordingly, different types of asphalt have been classified based on the production temperatures; these are Hot Mix Asphalt, Warm Mix Asphalt, Half Warm Mix Asphalt, and Cold Mix Asphalt (Nicholls et al., 2015), as shown in Figure 2-1.

HMA is a type of asphalt that is produced at elevated temperatures roughly between 150-175°C (Heneash, 2013). It is the most dominant type of asphalt and has been used for many decades. In fact, most of the current roads and airfields are constructed using this type of asphalt. However, under the demand for sustainability requirements, highway agencies are investigating alternative types of asphalt. The second type of asphalt is WMA; it refers to asphalt mixtures produced at temperatures 10-40°C lower than reference HMA (Perkins, 2009, EAPA, 2010). WMA is receiving increasing attention due to the expected environmental and economic benefits from reducing production temperatures of asphalt. Half warm mix asphalt indicates asphalt mixtures produced at temperatures between approximately 70-100°C (Nicholls et al., 2015). This mixture can be produced by using foamed or emulsified bitumen with aggregate heated to the production temperature of the mix in order to improve the aggregate coating process and accelerate water evaporation. Cold mix asphalt is an asphaltic mixture produced at ambient temperatures using emulsified or foamed bitumen. The main problem with this type is the curing time; it takes up to six months before the material builds its strength and meets the design stiffness (Thom, 2014, Needham, 1996), which limits the application of this kind of asphalt.

The common aim among these technologies is to reduce production temperatures of asphalt, either by foaming technologies, using additives, or using emulsified bitumen. However, the consequences of reducing the temperature can be reflected on critical properties of asphalt, such as aggregate coating, workability, compactability, and most importantly, the mechanical properties. Since the focus of this study is on WMA, this is described in more detail in the following sections.

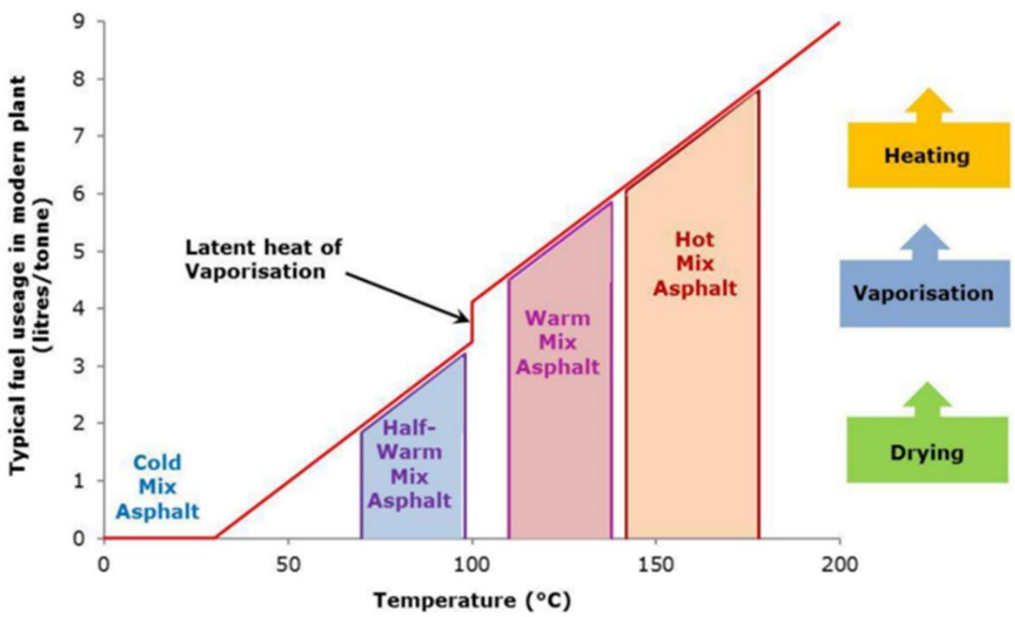


Figure 2-1. Asphalt types based on the mixing temperature (Nicholls et al., 2015)

2.2 Brief History of WMA

The concept of reducing asphalt production temperatures to achieve economic or environmental benefits is not new to the asphalt industry. Although the currently used WMA was developed in the mid-nineties of the last century, the first work of reducing bitumen mixing temperatures was carried out in 1956 when Dr Ladis from Iowa State University innovated a method to produce foamed bitumen by injecting steam into hot bitumen; the aim was to improve soil stabilisation using foamed bitumen. In 1968, Mobil Oil Australia bought the discovery rights from Dr Ladis and modified it by injecting cold water instead of steam to produce foamed bitumen (Muthen, 1998). Since then, a number of studies on implementing and improving the foamed bitumen have been conducted.

In 1970, Bowering (1970) used foamed bitumen in soil stabilisation; he used a Hveem Stabilometer to evaluate the improvement in soil strength. Based on the test results, he concluded that the foamed bitumen improved the strength of a wide range of soil materials, including sandy and gravely soils. In 1976, Bowering and Martin (1976) conducted a comprehensive review of the impacts of applying foamed bitumen on pavement performance and construction; they concluded that foamed bitumen could significantly stabilise weak soils to be used in pavement construction, and this method was proved to be very economical. In 1979 Acott (1979) used foamed bitumen to enhance the shear strength of sandy soils. He concluded that the application of a small amount of foamed bitumen to the sandy soils could significantly improve its shear resistance characteristics which leads to reducing the vertical strain at the top of the subgrade and improving pavement integrity. In 1982, Wijk and Wood (1982) used foamed bitumen with RAP to construct a base course layer by mixing 4% of foamed bitumen. The researchers reported that the initial performance was satisfactory. In 1988, Akeroyd and Hicks (1988) applied foamed bitumen in stabilising base and sub-base pavement layers of road sections located in the UK. They used Marshall stability and the Nottingham repetitive load apparatus to evaluate material mechanical properties; they reported that comparable stiffness to that of HMA could be achieved by using RAP and foamed bitumen. In 1993, Maccarrone et al. (1993) used foamed bitumen in stabilising a base course layer instead of cement to improve the fatigue resistance of this layer and reduce curing time. They stated that the foamed bitumen could significantly reduce construction cost and improve the strength and durability of pavement materials.

However, traditional WMA was developed in 1996 in Europe. The principal aims of developing this technology were reducing greenhouse gas emissions and saving energy (Sargand et al., 2009). The first pavement constructed using WMA was in 1997 in Germany, and Sasobit was used in that project (Prowell et al., 2012). Since then, this technology has been gaining global popularity due to the expected economic and environmental benefits. In 2000, a group of European asphalt companies published one of the earliest reports about laboratory evaluation and real pavement section trials constructed using WMA technologies in different countries including Norway, UK, and Netherlands (Koenders et al., 2000). The WMA was used to construct wearing courses with different thickness and under mostly medium traffic volumes. The report demonstrated that there was no discernible difference between HMA and WMA in terms of volumetric properties, stability, adhesion, and permanent deformation. The study, however, recommended further work to validate the long-term performance of WMA and its performance when RAP is incorporated.

In the United States, WMA technology became familiar early in this century. In 2007 a group of American experts in pavement materials visited several European countries in order to investigate the benefits and drawbacks of this technology (D'angelo et al., 2008). Lower CO_2 emissions, reduced fuel consumption, better compaction and longer hauling distance are the main advantages of WMA that the group had realised. The experts expected that this technology would be applied soon in the US, and there were no obstacles to its implementation. Since then, WMA has gained wide popularity in the US; it represented about 6% of the total asphalt production in 2009, and 19% in 2011 (NAPA, 2013). In 2015, it increased to 33% of the total asphalt production (NAPA, 2015), and the latest report of the National Asphalt Pavement Association stated that in 2017 WMA represented 38.9% of the total asphalt production (NAPA, 2018).

2.3 WMA Production Technologies

Since the development of WMA, a significant number of WMA technologies have been developed. Currently, this research highlights twenty-six technologies that are currently being followed in the market to produce WMA, as shown in Table 2-1. These technologies have commonly been classified based on the nature of the additive used into three main categories: technologies that utilise organic additives, technologies that apply chemical additives, and bitumen foaming technologies (D'angelo et al., 2008, Zaumanis, 2010). The following sections briefly describe some of the commonly used technologies.

Table 2-1. Currently in practice WMA technologies

Additive	Manufacturer	Technology	Ref
Sasobit®	Sasol	Organic	(Sasol)
Asphaltan A, B, Bit and 117	ROMONTA	Organic	(Romonta Gmbh)
Licomont® BS 100	Clariant	Organic	(Clariant)
Leadcap	KUMHO	Organic	(Kumho Petrochemical Company)
SMC	Not stated	Organic	(Ai et al., 2015)
3E LT or Ecoflex	Colas	Organic	(Rubio et al., 2012)
Montan Wax	Romonta GmbH	Organic	(Rubio et al., 2012)
EvothermTM DAT and EvothermTM 3G	Ingevity	Chemical	(Ingevity, 2018)
Rediset ® WMX-8017A and Rediset LQ	AkzoNobel	Chemical	(Akzonobel)
Cecabase RT	Arkema Group	Chemical	(Arkema Group, 2016)
Thiopave	Shell	Chemical	(Shell)
Revix	Mathy-Ergon	Chemical	(Rubio et al., 2012)
SonneWarmixTM and SonneWarmix RJTM	Sonneborn	Chemical	(Sonneborn)
ITERLOW T	IterChimica	Chemical	(Iterchimica, 2018)
Aspha-min®	GmbH	Foaming mineral	(Gmbh, 2018)

Advera® WMA	PQ Corporation	Foaming mineral	(Pq Corporation, 2018)
Double Barrel Green	Astce	Water-based foaming	(Astec Dillman, 2018)
Ultrafoam GX,	Gencor Industries	Water-based foaming	(Gencor, 2018)
Aquablack WMA	MAXAM Equipment	Water-based foaming	(Maxam Equipment, 2018)
Warm Asphalt Mix (WAM) Foam	Kolo Veidekke and Shell	Water-based foaming	(Larsen et al., 2004)
Low Energy Asphalt	LEACO	Water-based foaming	(Rubio et al., 2012)
Low Emission Asphalt (LEA)	McConnaugheay Technologies	Water-based foaming and foaming mineral	(Perkins, 2009)
LT Asphalt	Nynas	Water-based foaming	(D'angelo et al., 2008)
LEAB	Royal Bam Group	Water-based foaming	(Bam, 2018)

2.3.1 Organic Additives

Organic additives are used to reduce bitumen viscosity at asphalt production temperatures. The basic idea of applying these additives to asphalt mixtures is to reduce bitumen viscosity at mixing temperatures, which allows for producing asphalt at reduced temperatures. These additives usually consist of wax or fatty amides, and they can be applied to decrease bitumen viscosity at temperatures above their melting temperature (Almeida-Costa and Benta, 2016, Prowell et al., 2012). Caution must be taken when selecting an organic additive since the melting point of the additive should be higher than the high critical temperatures of the in-service pavements otherwise the additive may melt, and the pavement may suffer from permanent deformation (EAPA, 2010). There are several marketed organic additives described as follows:

2.3.1.1 Sasobit®

Sasobit is an organic WMA additive produced by Sasol Wax Company. Figure 2-2 shows two typical forms of this additive. This additive has two distinct impacts on bitumen. First, it works as a viscosity reducer above its melting temperature which is about 100°C. Hence it is used to reduce production temperatures of asphalt, enhance its workability, and improve the asphalt compaction process (Sasol). Second, it creates a lattice structure that stiffens the bitumen at temperatures below its melting point (Choi, 2007). Chemically, it consists of long polymethylene hydrocarbon chain lengths between C45-C100 (Sasol, 2016b), which increases the melting point of bitumen since the natural wax in bitumen melts at temperatures around 70°C due to its relatively low hydrocarbon chain length of C22-C45 (Damm and Hinrichsen, 2003).

Depending on the required results, Sasol recommends adding 1.5-4% of Sasobit by weight of bitumen (Sasol, 2016a). Dosages up to 3% are mainly used to reduce production temperatures, improve asphalt workability, reduce CO_2 emissions, reduce bitumen ageing, and improve compaction during poor weather conditions. Higher dosages up to 4% are used to improve deformation resistance, allow faster opening of roads to traffic, improve rutting resistance at bus stops, and permanent deformation performance asphalt pavement roads under heavy traffic volumes. Table 2-2 provides a more comprehensive idea about the common Sasobit dosages used in previous studies and the nature of these projects (Jamshidi et al., 2013). This table indicates that for airport applications, 3-4% was applied, whereas 1.5-3% was applied for roads.

The effect of this additive on bitumen characteristics has been extensively investigated in the literature. The most consistent effect that was reported by several studies is bitumen stiffening, as it increased softening point and decreased penetration (Abed et al., 2017, Rodríguez-Alloza et al., 2013, Silva et al., 2010b). On bitumen rheology, it increased complex shear modulus, reduced phase angle, and increased Superpave critical high temperature which improved the permanent deformation resistance (Julaganti et al., 2017, Silva et al., 2010b, Zhang et al., 2015, Kim et al., 2011a). Also, it retarded bitumen ageing (Hurley and Prowell, 2005, Banerjee et al., 2012). At asphalt mixing temperatures, it reduced bitumen viscosity, which allowed for asphalt mixing at reduced temperatures (Hurley and Prowell, 2005, Rodríguez-Alloza et al., 2013).

On the asphalt performance level, researchers reported inconsistent results. In terms of rutting, Hurley and Prowell (2005) demonstrated that due to the binder ageing retardation property of Sasobit and the reduced production temperatures of WMA, the mix became more prone to permanent deformation. Other studies, however, mentioned that despite the reduced production temperatures of WMA, Sasobit improved rutting resistance to different extents (Jalali, 2016, Wang et al., 2013).

In terms of fatigue cracking, based on Jamshidi et al. (2013)'s literature review, some researchers have concluded that this additive could reduce fatigue cracking resistance because they noticed that this additive reduced the tensile strength in their studies. A recent study by Kim et al. (2017) also showed that this additive could reduce fatigue cracking resistance. However, other studies indicated that this additive resulted in similar fatigue cracking resistance to a control mix (Sanchez-Alonso et al., 2013) or even improved fatigue life (Jalali, 2016).

Figure 2-2. Two typical types of Sasobit, flakes to the left and prills to the right. (Hurley and Prowell, 2005)

Table 2-2. Sasobit dosages used in different European countries

Year	Country	Name	Utility	Area (m2)	Sa-sobit (%)	Binder type	Mix type
2007	Russia	Gelendzhik	Runway and Taxiway	200,000	3	Pen 60/90	AC 0/85, AC 0/16S
2006	Norway	Svalbard	Runway and Taxiway	170,000	3	Pen 490	AC 0/11
2005	Austria	Linz–Hörsching	Runway	50000	3	PMB 60/90	AC 0/16, BT 0/22
2005	Ger-many	Frankfurt	Runway	250,000	4	PMB 45, PMB 25 and Pen 30/45	SMA 0/11 S, AC 0/22, AC 0/32 CS
2005	Slove-nia	Spodnjibrnik–Moste	Road	1800	3	AC 50/70	BT 0/16 S, BT 0/22 S, AB 0/11 S
2005	Slove-nia	Laze	Road	1000	3	AC 50/70, Olexobit 45	Abi 0/22, SMA 0/8
2005	Serbia	Belgrad	Runway	20,000	3	PMB 60/90	AC 0/8, AC 0/16
2005	Ger-many	Eurogate (Hamburg)	Container Terminal	50,000	4 and 2	PEN 50/70, PEN25 PMB	0/22 incl. 50%RAP, 0/8 incl. 20%RAP
2004	Ger-many	Tollerort (Humburg)	Container Terminal	70,000	4	PEN 25 PMB	0/16S and SMA 0/16

2004	Ger-many	Airbus Terminal Finkenwerder (Hamburg)	Runway	96,000	2.5	PMB Carbit 45, PEN 25 PMB	AC 0/11S
2004	Ger-many	Airport Berlin Shonefeld	Runway	135,000	3	PEN 45 PMB	0/16S
2003	Ger-many	Munich	Apron	60,000	3	PMB 45	AC 0/11, AC 0/16
2003	Sweden	Sturoman	Apron and runway	110,000	3	Pen 120/160	AC 0/11
2003	Den-mark	M23, Oslo–Drammen	Motorway	100,000	3	B 85	SMA 15
2003	Sweden	Umea (near Arctic Circle)	Military Airport	110,000	3	B 120/160	ABTS 11
2003	Den-mark	M30, Rodby (Heavy traffic side)	Motorway	30,000	2	AC 40/60	ABB
2003	Den-mark	M11, Holbaek (Heavy traffic side)	Motorway	15,000	2	AC 40/60	ABB
2003	Den-mark	M70, Alborg South	Motorway	81,000	1.5, 3	SMA 11, ABB	AB 60
2003	Ger-many	A25 Hamburg	Motorway	25,000	1.5	SMA 0/8 S	0/16S, Caribit 25 RC, Caribit 45
2003	Czech	Brno	Road	3000	3	PMB 45	SMA 0/11 S
2003	Ger-many	Fraport (Frankfurt)	Runway	100,000	4	0/22 S, Carbit 25 RC, Carbit 25/45	SMA 0/11 S, ATS CS 0/32
2002	Norway	Drammen–Westfall	Road	10,000	3	B 85	SMA 11 + 15
2002	Norway	Oslo–Drammen	Motorway	100,000	3	B 85	SMA 15
2002	Ger-many	Rendsburg	Bridge Pavement	16,500	3	Olexobit 45	Gussasphalt 0/11 S
2001	UK	Cambridge	Apron	3000	3	PMB 65	AC 0/16
2001	Italy	Piemont, Turin	Road	4000	3	AC50/70	AC 0/16
2001	Hungry	Szekesfehervar	Road	3000	3	AC 50/70	SMA 0/11
2001	Ger-many	Veddeler Damm–Hamburg	Industrial road in the port of Homburg	5000	3	PEN 45 PMB	0/16HS, SMA 0/8
2001	Ger-many	Hamburg Airport	Runway	60,000	3	AC 50/70	SMA 0/11
2000	Ger-many	A1, Maschen – Harburg	Motorway	25000	3	SMA 0/5	Caribit 45

2000	Ger-many	B 83, Bad Eilsen (A-Road)	Road	25000	3	SMA 0/5	Caribit 45
2000	Den-mark	Aarhus	Road	22,000	3	SMA 0/8	AC50/70
2000	France	Misc. streets	Road	20,000	3	Gussas-phalt	AC 30/50
2000	Nether-land	Docking sta-tion Schoi-tema,Woerden	Road	15,000	2 and 6	Gussas-phalt	AC 20/30
2000	Ger-many	Neuhöfer Straße, (Ham-burg)	Waste dis-posal site	80,000	4	AC 50/70 0/16	SMA 0/11

2.3.1.2 Asphaltan

This additive is a product of ROMONTA Company located in Germany. There are four types of it: A, B, Bit, and 117. All of these additives can be applied to reduce bitumen viscosity at temperatures above their melting point. Hence they improve aggregate coating, allow for asphalt production at reduced temperatures, and enhance asphalt compactability (Romonta Gmbh). They also decrease bitumen penetration and increase softening point, indicating a stiffening effect and offering improved permanent deformation resistance (Rodríguez-Alloza et al., 2013). At low temperatures, these additives slightly increase bitumen stiffness, which may indicate a minor detrimental impact on low temperature cracking performance (Das et al., 2012).

2.3.1.3 Licomont® BS 100

This is also a viscosity reducer additive produced by CLARIANT chemical company. When applied to bitumen, this additive can reduce its viscosity, which enhances asphalt workability and offers asphalt production at reduced temperatures (Clariant). The manufacturer claims that this additive can prolong pavement service life, increase its durability, and enhance asphalt resistance to rutting which makes it suitable for sections under heavy traffic loading such as airfields, intersections, and bus stops. Rodríguez-Alloza et al. (2013) investigated the effect of this additive on bitumen characteristics. They used 50/70 dmm penetration grade bitumen in their study and found that this additive significantly increased the softening point and can reduce asphalt production temperatures by about 17°C if 4% of the additive included.

2.3.1.4 Leadcap

Leadcap is a WMA additive designed by KUMHO Petrochemical Company exclusively to produce WMA. This additive can reduce mixing and compaction temperatures by about 20 to 30°C when applied with a dosage 2-3% by weight of bitumen, the main advantages of this product are better low temperature cracking resistance, improved rutting resistance and improved moisture resistance since it contains an adhesion promoter (Kumho Petrochemical Company). Kim et al. (2014) studied the effect of modifying a crumb rubber modified binder with this additive and reported that this additive reduced bitumen viscosity and improved its high-temperature rheological properties. They also stated that this additive improved low temperature cracking resistance.

2.3.1.5 SMC

SMC is an organic bitumen modifier developed recently in China. This modifier offers a significant increase in asphalt strength when it is tested in terms of Marshall Stability. However, by investigating its impact on the moisture damage and low temperature cracking resistance, it was concluded that this additive has a relatively negative impact on these measures (Ai et al., 2015). Therefore, it was recommended to apply this additive in regions that are subjected to little rainfall and have a relatively warm climate.

2.3.2 Chemical Additives

This technology involves modifying bitumen with a "surfactant" that reduces the binder surface tension and enhances aggregate coating and adhesion when mixing asphalt at reduced temperatures. At the microscale level, chemical additives reduce the frictional forces at the interface of the binder and aggregates, which offers approximately 20-30°C reduction in asphalt production temperatures (EAPA, 2010). The chemical additives consist typically of different materials such as surface tension reducers, emulsification agents, and aggregate-bitumen adhesion enhancement agents (Zaumanis, 2010). Since these additives are liquids at ambient temperature and do not have a specific melting point, they mitigate the limitation of choosing the additive based on the meting point, which is the case with the organic additives. Currently, several types of chemical additives are available in the market; some of the frequently used types are described as follows:

2.3.2.1 Evotherm^{TM}

This additive comes under three versions, Evotherm^{ET} (emulsion technology), Evotherm^{DAT} (Dispersed Asphalt Technology), and Evotherm 3G (third generation). Basically, it is an emulsion that consists of a bitumen and chemical package that is designed to enhance aggregate coating, binder-aggregate adhesion, and mixture workability. The idea behind this additive is that most of the water contained in the emulsion is designed to evaporate as soon as it contacts hot aggregates, which enables the production of asphalt at temperatures 85-115°C (D'angelo et al., 2008).

2.3.2.2 Rediset®LQ

Rediset® is a chemical WMA additive produced by AkzoNobel Company. The manufacturer states that this additive provides: significantly enhanced workability and compactability when producing asphalt at reduced temperatures, improved moisture damage resistance because it contains an adhesion promoter, enhanced aggregate coating, and reduced gas emissions and fuel consumption (Akzonobel). This additive works by decreasing binder surface tension, which promotes its aggregate coating ability, which reduces asphalt production temperatures by about 33°C (Prowell et al., 2012).

2.3.2.3 Cecabase RT®

Cecabase RT is a liquid surfactant produced by ARKEMA Group; it allows for the asphalt to be produced and compacted up to 40°C lower than conventional HMA (Arkema Group, 2016). This additive decreases bitumen surface tension, which reduces friction forces at the interface between the binder and aggregate and allows for coating and compaction at reduced temperatures (Prowell et al., 2012). There are three types of this additive, Cecabase RT® BIO 10, Cecabase RT® 945, and Cecabase RT® 2N1. All of them are used to produce WMA with improved workability and compactability; the main difference is that the second type contains an antistripping agent whereas the third one offers enhanced moisture resistance, which makes each type suitable for particular applications.

2.3.2.4 Thiopave

Thiopave is a chemical additive innovated by the Shell company. It consists of sulphur and special additives designed to reduce production temperatures of asphalt and prolong its service life. This additive can be used to mix asphalt at 135°C while the mixture is still compactable at temperatures down to 60°C (Shell). This additive is added as a solid pellet directly to asphalt; when it contacts hot aggregate, it immediately melts and behaves similar to a binder which replaces about 30-40% of the base binder. This additive can enhance Marshall stability, improve rutting resistance, increase asphalt stiffness, and it may slightly reduce water resistance and increase low-temperature cracking potential (Nicholls, 2009).

2.3.2.5 SonneWarmix

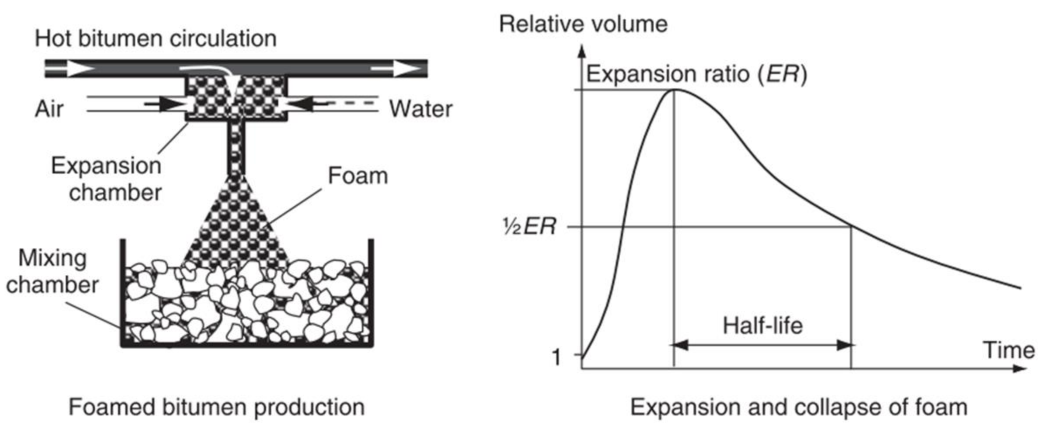
This additive is a high melting point paraffinic hydrocarbon compound produced by Sonneborn AR. It improves asphalt workability and compactability, which helps in reducing production temperatures by about 10°C (Sonneborn). Its typical dosage is between 0.5-1.5% by weight of bitumen; at this level of modification, this additive may not change the base binder grade (West et al., 2014a). There are two types of this additive, SonneWarmixTM and SonneWarmix RJTM, the main difference between them being that the second one is designed to incorporate RAP (up to 60%) in the mix because it works as a binder rejuvenator.

2.3.3 Foaming Technologies

Foaming technologies follow an entirely different approach from the previous WMA technologies because they depend on changing bitumen physical status rather than modifying its characteristics. In these technologies, the bitumen is transformed from liquid status into foam, which reduces its “effective” viscosity and allows for its application at reduced temperatures. Figure 2-3 (a) illustrates the process of foaming bitumen. In this process the bitumen is pumped hot usually between 160-180°C, then cold water at a rate of 2-5% of the bitumen weight and air are injected into the bitumen; when the water contacts the bitumen it immediately evaporates and encapsulates in the bitumen creating foam (Thom, 2014).

Foam bitumen can be characterised by two main properties, expansion ratio and half-life (Kuna et al., 2014). The first one is the ratio of the maximum volume of foamed bitumen to its liquid state volume. The second property is the time calculated from the second that the foamed bitumen attains its maximum volume until it collapses to half of the maximum volume; both characteristics are illustrated in Figure 2-3 (b). Both of these parameters are critical in the WMA production process; the expansion ratio represents the binder volume during mixing, which means that a higher expansion ratio leads to lower viscosity bitumen, whereas the half-life indicates the time that is available to mix asphalt while bitumen is still foam.

In order to improve these characteristics, different methods have been developed, and foaming additives have been designed. Generally, these methods have been classified into two broad techniques, water-based and water-bearing (Perkins, 2009, Zaumanis, 2010), described as follows:



a b

Figure 2-3. Bitumen foaming principles, (a) foaming process, (b) characteristics of foamed bitumen (Thom, 2014)

2.3.3.1 Water Based Bitumen Foaming Process

This technique can be considered as a direct method to produce foamed bitumen by injecting a pre-calculated amount of water into hot bitumen. The water evaporates and is entrapped in the bitumen as bubbles; this process temporarily increases bitumen volume which leads to reduced bitumen viscosity and improved coating and compaction at reduced temperatures, approximately 20-30°C lower than a reference mixture (EAPA, 2010). There are several available technologies that implement this concept in WMA production; some of the common ones are described as follows:

2.3.3.1.1 Warm Asphalt Mix (WAM) Foam

WAM-Foam is an asphalt production process developed after a joint effort between the Kolo Veidekke and Shell companies in 1995. This production process consists of two stages, in the first stage, very soft bitumen, representing 20-30% of the total bitumen, is introduced at 100°C to provide initial aggregate coating; in the second stage a hard binder is foamed by injecting cold water and added to the mix (Prowell et al., 2012). The final asphalt discharge temperature is between 100-120°C, which can provide about 30% energy saving and an equivalent reduction in carbon footprint (Larsen et al., 2004).

2.3.3.1.2 Low Emission Asphalt

LEA is a complicated WMA production process consisting of two stages. In the first stage, the coarse aggregate and part of the fine aggregate are mixed with bitumen that contains a coating and adhesion promoter at conventional HMA temperatures. In the second stage, the rest of the aggregate is introduced cold and wet with about 3-4% moisture; the moisture is turned into steam when it contacts the hot mix which creates foam within the mix that enhances mix workability and compactability (Bonaquist, 2011a). The equilibrium (asphalt discharge) temperature of this process is about 100°C (D'angelo et al., 2008).

2.3.3.1.3 Double Barrel Green, Ultrafoam GX, Aquablack WMA, and Warm Mix Technologies

These technologies have been developed by different companies in the US. Each technology utilises specific production equipment developed by the individual manufacturer. In these technologies, the same concept of injecting cold water to foam the bitumen is applied, a small amount of water is added to a hot binder that evaporates and creates steam encapsulated in the binder as microscopic bubbles which form foamed bitumen (Zaumanis, 2010).

2.3.3.2 Water Containing Bitumen Foaming

This type of foaming technology can be considered as an indirect method to produce foamed bitumen, because the water used to foam the bitumen is crystalline water contained in a hydrophilic mineral, usually zeolite. The scientific reason for using zeolite as a foaming additive is that its crystalline structure contains some voids that can be utilised to store water. Also, it is a hydrophilic material which makes it able to absorb and store water without having any adverse effect on its structure (Asphalt Scientist, 2018). The amount of water that can be stored in the zeolite structure is approximately 20%; this water is released as a mist when heated above 100°C (EAPA, 2010) which makes it a very suitable additive to foam bitumen at WMA production temperatures.

Currently, there are two well-known WMA additives that are used in foaming techniques, Advera and Aspha-Min. The first one is a WMA additive produced by Eurovia Services GmbH. It consists of very fine synthetic Zeolite (particle size <0.075mm) that contains about 20% of crystalline water that is released when heated above 100°C (Prowell et al., 2012). When the water is released, it creates steam that is entrapped in the bitumen, increasing its volume, which reduces bitumen viscosity and enhances mixture workability. It is recommended to incorporate 0.1-0.3% of this additive by weight of the total mix; lower doses are suitable for enhancing mix compactability whereas higher dosages are suggested for mixtures with binder content more than 7% (Prowell et al., 2012).

The second additive is also a synthetic Zeolite powder but has a relatively larger particle size than Advera. It also contains 20% of crystalline water that is released during asphalt mixing creating bubbles in the mix, the bubbles increase binder volume and reduce its viscosity which improves mix workability and compactability (D'angelo et al., 2008). This additive has the ability to gradually release water which is used to prolong the foam status of bitumen up to seven hours as long as the mix temperature is above 100°C (Prowell et al., 2012). The recommended dosage of this additive is about 0.3% by weight of the total mix, and it offers approximately 20-30°C reduction in the production temperatures of asphalt (D'angelo et al., 2008).

2.4 Advantages of WMA Technologies

All WMA methods share one ultimate aim, which is reducing production temperatures of asphalt to achieve environmental and economic benefits. Logically, decreasing mixing temperatures means reducing the amount of energy/ fuel required to heat up aggregate and bitumen, it also means decreasing CO_2 emissions equivalent to the saved amount of fuel. It also helps in retarding bitumen ageing which can make asphalt less susceptible to cracking and more durable than HMA. Accordingly, the more significant the reduction in the production temperatures, the greater the benefits of WMA technologies. In this regard, several researchers have investigated the benefits of implementing WMA.

Larsen et al. (2004) used Warm Asphalt Mix foaming technology to produce asphalt at temperatures between 100-120°C. They reported that this technology offered about 30% reduction in fuel consumption; they also observed a decreased amount of CO_2 emissions and fumes at asphalt plants. Frank et al. (2011) investigated the economic and environmental benefits of eight WMA technologies including organic, chemical and foaming methods. They measured fuel consumption at six asphalt plants and carbon emission at three plants for technical reasons. They stated that these technologies offered about 10-15% less greenhouse gas emissions and 8-35% decreased fuel consumption. However, they pointed out that these measurements depend on several factors including selected WMA technology, reduction in the production temperature, fuel type, and moisture content of the aggregate.

Another field study was conducted by Prowell et al. (2014) to evaluate impacts of multiple WMA technologies in comparison with reference HMA. The average reduction in production temperatures of asphalt was about 10°C, which led to 22.1% saving in fuel consumption and a 20% decrease in CO_2 emissions. However, the study mentioned that these measurements were affected by other factors such as the condition and maintenance of asphalt plants, the burner design of the plants, and tuning of the burners which appeared to have a substantial effect on fuel consumption and CO_2 emissions. Almeida-Costa and Benta (2016) deployed heat transfer and water evaporation principles to calculate the energy consumption of two WMA additives namely Sasobit® and Rediset™, and they converted the calculated energies to equivalent CO_2 fumes using emission factors suggested by UK Department of Energy and Climate Change. They concluded that the WMA technologies lowered energy consumption between 8.6-14.4% in comparison with reference HMA. Their results also indicated that these technologies reduced CO_2 emissions by about 8%. However, the emission reduction of the study is relatively conservative, because most environmental evaluation studies of WMA technologies have reported a reduction between 15-40%. For instance, D’angelo et al. (2008) reported that CO_2 emissions were reduced by about 31% in Norway, 30-40% in Italy, 15-30% in Netherlands, and 23% in France, whereas Prowell et al. (2012) stated that emission reduction depends on several factors including type of fuel, asphalt plant design, aggregate moisture content, and level of production temperature reduction.

The above advantages of WMA can be considered as direct benefits because they are relatively quantifiable. However, indirect benefits have been reported in the literature, as follows (D’angelo et al., 2008, Button et al., 2007, Zaumanis, 2010, Prowell et al., 2012).

1. Improved asphalt workability and compactability.
2. Possibility to compact asphalt in colder weathers.
3. Ability to transport asphalt more considerable distances without any drop in workability or compactability of the mix.
4. Capability to compact thicker asphalt layers than usual lifts
5. Reduced compaction effort in case of compacting at the same condition as the reference HMA.
6. Extended pavement construction season due to the ability to compact in colder weathers.

7. Improved compaction in the case of RAP or Reclaimed Asphalt Shingles (RAS) incorporation, because these mixtures usually contain a stiff binder which is hard to compact. Therefore, the presence of WMA can improve the compactability of these mixtures.

8. Ability to incorporate higher levels or RAP due to the reduced binder ageing resulting from the decreased production temperatures.

9. Healthier working environment for the workers at asphalt plants and construction sites as well due to the reduced amount of emissions and fumes.

10. Friendlier to the public close to construction sites because of the reduced pollution.

11. Ability to pave in nonattainment areas where it is restricted to perform construction works because of the inadequate air quality, and that is due to the improved construction environment of WMA.

12. Some WMA products offer a quicker opening of roads to traffic, which may be an essential factor at specific sites such as airports.

2.5 Drawbacks of WMA Technologies

Despite the promising benefits of WMA technologies, there are some disadvantages and general uncertainty regarding cost-effectiveness and performance of these technologies. On the one hand, if it is assumed that the WMA performance is generally accepted, there is still some concern related to the initial cost of the additives which may make the production cost of the WMA higher than HMA. Furthermore, costs of any required modification to asphalt plants in order to adopt WMA technologies is another concern that should be considered. Kristjansdottir

(2006) conducted production cost estimation analysis for three of the most widely used WMA technologies namely WAM foam, Aspha-min, and Sasobit in comparison with control HMA; the results are presented in Table 2-3. It can be seen that the WAM Foam has the highest modification cost ($45000-55000) since foaming technologies require significant modification to asphalt plants. However, it has the lowest cost ($0.3 per ton), since in this technology the foaming additive is water only. On the other hand, the other technologies have a lower plant modification cost but higher production cost due to the cost of the additives. This means that WMA technologies are likely to increase the initial production cost of asphalt, which may hinder the implementation of these technologies.

On the other hand, it is not generally accepted that the WMA technologies can assure an acceptable performance as good as the HMA performance in the field. Several researchers have stated that WMA performance must be proven to be equivalent to the conventional HMA before it can be fully implemented. Prowell et al. (2014) stated that despite the promising benefits of WMA technologies, their performance should be as good as HMA; otherwise, they will not receive attention. Other studies (Capitão et al., 2012, Jamshidi et al., 2013) have also reported that WMA performance should be at least comparable or even better than traditional HMA in order to be accepted as a satisfactory alternative. Poor WMA performance results in shorter pavement service life than HMA. In this case, WMA needs more maintenance which also causes further traffic delay and can be considered less sustainable than HMA. Accordingly, WMA performance must be at least equivalent to HMA and must be analysed and considered before judging the sustainability of these mixtures.

In this regard, different studies have been conducted to understand and evaluate the mechanical performance of WMA. Zaumanis (2010) mentioned that the reduced production temperatures of WMA lead to decreased binder ageing, which may cause premature permanent deformation in the early life of pavements.

Table 2-3. Costs ($ per ton) of some WMA technologies (Kristjansdottir, 2006)

Cost type	HMA	WAM Foam	Aspha-min zeolite	Sasobit wax
Additional (additive) cost	-	0.3	4.0	3.5
Cost of energy (oil + electricity) consumption	6.5	4.9	4.9	4.9
Reduction in energy cost	-	1.6	1.6	1.6
Calculation for total cost	-	89+0.3-1.6	89+4.0-1.6	89+3.5-1.6
Total cost	89	87.7	91.4	90.9
% increase or decrease	-	-1.5	+2.7	+2.1
Installation cost and royalty	-	45000-55000 +5000 annually	-	-
Example: A 3000-ton production in the beginning year	2670000	2677000	2742000	2727000
Example: A 3000-ton production per year (after 1st year)	2670000	2631000	2742000	2727000

Mo et al. (2012) studied the rutting performance of WMA produced by using chemical additives by conducting immersed wheel tracking tests; they concluded that the WMA did not show comparable performance in comparison with control HMA samples. They also stated that rutting resistance and stripping were significant defects in the applied WMA technology. Benta et al. (2015) investigated the performance of WMA produced using Rediset^{TM}; their results indicated that the rutting resistance and moisture damage resistance of the studied WMA were decreased. These results were attributed to the relatively high air voids of the tested WMA samples. However, this explanation contradicts one of the repeatedly reported benefits of WMA technologies, which is the improved mix workability and compactability. Wang et al. (2013) also investigated the performance of different WMA technologies. They observed that Sasobit improved rutting resistance, but it increased the susceptibility of WMA to low-temperature cracking, whereas the second technology, Aspha-min, negatively affected rutting and moisture damage resistance. Das et al. (2012) also mentioned that wax additive slightly decreased low temperature cracking resistance.

With respect to the moisture damage, it has been reported that the reduced production temperature of WMA can lead to an incomplete drying of aggregate which may increase the potential of that kind of failure (West et al., 2014b). However, it seems that this measure is WMA technology dependent. Caro et al. (2012) investigated the moisture susceptibility of three WMA methods using dynamic mechanical analysis. They found that Aspha-min technology was the most susceptible mix to moisture damage, Evotherm performance was acceptable, whereas Sasobit was also prone to moisture damage but not as critical as the Aspha-min performance. Other studies (Khodaii et al., 2012, Mohd Hasan et al., 2015) also reported negative performance of WMA regarding moisture damage resistance, but they demonstrated that using hydrated lime is a successful method to improve WMA performance against moisture damage.

The other point that has to be mentioned is that WMA technologies are still relatively new. The current experience of highway agencies and the asphalt industry with WMA performance is mainly based on trial sections constructed in the past ten years. Thus the long-term performance of WMA technologies can be considered as a source of uncertainty because, apart from premature rutting, most pavement distress is a function of traffic loading and environmental effects which in turn are functions of time. Moreover, it can be seen that most of the cited studies did not report any concern regarding WMA performance in terms of fatigue cracking. However, as stated earlier, the fatigue cracking problem appears after asphalt suffers from a certain ageing level, usually long-term ageing. This means that it is too early to judge the performance of WMA regarding fatigue cracking as this measure should be evaluated after long-term ageing. However, a recent study by Kim et al. (2017) indicated that the presence of WMA additives (Sasobit and Advera) caused a drop in fatigue life which means that this kind of failure may be a point of weakness of that kind of asphalt.

2.6 Mix Design of WMA

Mix design is a primary step in producing a successful asphalt mixture. It involves the determination and calculation of several essential factors including bitumen grade, optimum bitumen content, aggregate gradation, volumetric properties and performance analysis. Every factor has a significant effect on the mechanical response of the designed mix; binder grade must be selected based on the local climatic conditions; if a softer binder is used, then the mix may suffer from rutting. If harder bitumen is used, then the mix may exhibit fatigue cracking or low temperature cracking. On the other hand, binder rheological properties have a critical effect on the response of the mix, because they can significantly influence essential characteristics of the mix such as cohesive and adhesive strength. Aggregate gradation is another critical factor; it represents the skeleton of the mix that resists applied stress. Balancing aggregate sizes is a cornerstone to ensure optimum performance. The other critical parameter is the volumetric properties of the mix; they must be carefully designed; otherwise, mix response will be adversely affected. For instance, if the air voids are low, then this probably will increase stiffness of the mix, which leads to improved performance. If air voids are high, however, then this will reduce mix stiffness, which leads to higher strain development in the mix and eventually higher distress levels. Also, it can make asphalt susceptible to moisture damage as the water intrudes into the mix and affects adhesion strength between binder and aggregate.

For traditional HMA, asphalt mix design has been a standard practice for a couple of decades. Marshall and Superpave mix designs are two of the commonly applied mix design methods. For WMA, however, yet there isn't a standard mix design method, but there are guidelines developed by individual highway agencies based on their experience in the field. This could be attributed to the vast number of WMA additives and technologies, and the fundamental difference in the characteristics and effects of each technology on the design mix. Intrinsically, the main difference between HMA and WMA is the production temperatures. This means that some mix design measures can remain the same, such as binder content, aggregate gradation, or volumetric properties. However, the additive effect on bitumen characteristics and mechanical performance of the mix is technology dependent. Therefore, it should be individually evaluated.

One of the primary studies conducted to prepare a design method for WMA was the "Mix design Practice for Warm Mix Asphalt" performed by the National Strategic Highway Research Programme (NCHRP) (Bonaquist, 2011a). The main finding of this research was that WMA design does not require a new design method, but it can be designed as a special case of HMA. This finding was based on extensive laboratory and field studies. Another NCHRP report, number 9-43 (Bonaquist, 2011b), highlights the main modifications and changes required to design and adopt WMA technologies. The first modification is the selection of WMA technology; this step should be decided based on several factors including, reliability of the technology, available data regarding the performance of the technology, additive cost, costs of plant modifications if required, reduction in asphalt production temperatures. The second modification is the selection of binder grade. Generally, binder grade should be the same with both methods, HMA and WMA. However, it was suggested to raise the critical high temperature by one grade to meet rutting requirement as the reduced production temperatures of WMA retard binder ageing during production. The main differences between HMA and WMA are the selection of mixing temperatures, selection of compaction temperatures, and performance evaluation. These differences are discussed as follows.

2.6.1 Determination of WMA Mixing Temperature

One of the main differences between HMA and WMA is mixing temperatures. For conventional HMA, mixing temperatures can be designed based on bitumen viscosity measurements (Yildirim et al., 2000), as shown in Figure 2-4. The temperature at which bitumen exhibits viscosity equals to 0.17±0.02 Pa.s is selected as the design mixing temperature. This range of viscosity is to ensure full aggregate coating in a reasonable mixing time. Accordingly, some researchers used the viscosity criterion to determine the mixing temperature of WMA since some additives such as Sasobit depend on reducing bitumen viscosity to reduce mixing temperatures of asphalt.

In the literature, this method is called the equi-viscous approach. This method involves the determination of viscosity profile at different temperatures; the temperature at which bitumen exhibits the required mixing viscosity is selected as an optimum mixing temperature. Hurley and Prowell (2005) applied this concept to determine the mixing temperature of 64-22 binder; their results are illustrated in Figure 2-4. The figure shows that the WMA additive shifted viscosity profile vertically at temperatures above 105°C. The reduction in mixing temperatures, in this case, is about 20°C. Cao et al. (2011) also applied this concept, but they used rubberised bitumen rather than an unmodified one, they reported that the reduction in mixing temperature was about 18°C.

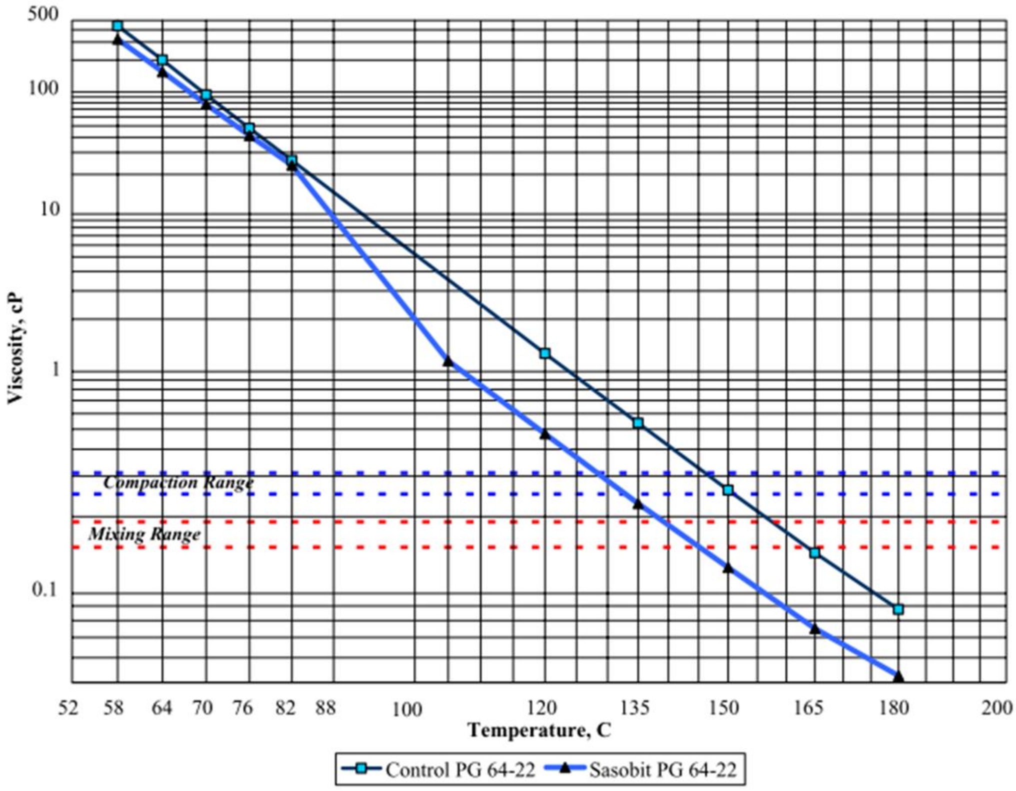


Figure 2-4. Mixing and compaction temperature determination based on binder viscosity (Hurley and Prowell, 2005)

However, this concept is valid only for additives that reduce viscosity. This means that the equi-viscous method cannot be utilised to determine the mixing temperatures of chemical additives since they do not alter bitumen viscosity, or of foaming methods since the viscosity of foamed bitumen cannot be accurately measured. Furthermore, some studies have mentioned that determining mixing temperature based on this approach can be misleading because it has been discovered that the reduction in mixing temperature is often quite insignificant, around 5°C in most cases. For instance, Silva et al. (2010a) applied this approach and reported that the Sasobit (which is a viscosity reducer) offered 3-8°C reduction in mixing temperature, as shown in Figure 2-5. Another study conducted by Silva et al. (2010b) also applied this concept to determine mixing temperatures; this study demonstrated that the wax additive (Sasobit) allowed for approximately 7°C reduction. Therefore, the researchers suggested to use a blend of soft binders (160-220 dmm) and wax to maximise the reduction in mixing temperatures; the idea was to implement the stiffening effect of the wax additive to improve soft binder performance at high temperatures. Other studies (Buss, 2010, Wasiuddin et al., 2007) reported similar conclusions. Accordingly, it can be concluded that the equi-viscous approach may not be a valid method to determine WMA mixing temperatures.

Workability of asphalt mix is another method that has been used to determine rational mixing and compaction temperatures. Workability can be defined as a measure that depicts the ability to place, work, move by hand, and compact asphalt mixtures (Wasiuddin et al., 2007). Traditionally, this method has been used as an alternative to the equi-viscous method to determine production temperatures of rubberised or polymer-modified bitumen. Because these kinds of binders exhibit dependence on the shear rate (non-Newtonian fluids) during viscosity measurements due to the presence of the modifiers, this makes it challenging to use viscosity to determine production temperatures of those kinds of asphalt. Accordingly, different studies have attempted to utilise the workability property to determine production temperatures of WMA technologies.

The workability can be used to determine production temperatures by a method called equi-torque. Theoretically, the workability of reference HMA has to be determined first, then the workability of the selected WMA technology at different reduced mixing temperatures is measured. The temperature at which WMA shows similar workability to HMA is considered as an optimum mixing temperature. Wang et al. (2013) applied this method to determine production temperatures of three WMA additives. They developed a workability device can measure the torque during mixing and at different temperatures. According to the results of this study, the developed device was sensitive to mixing temperature, even when the reduction was about 15°C.

NCHRP report 691 (Bonaquist, 2011a) has included the evaluation of four existing workability devices designed to measure the torque or force applied to spin a blade in an asphalt sample for potential implementation in determination of WMA production temperatures, two of them are shown in Figure 2-6, A and B. The main concern with these devices was their sensitivity to temperature changes. The report demonstrated that these devices could only be sensitive to mixing temperatures if the difference in mixing temperature is high, around 30°C. Accordingly, the study recommended that measuring workability is not required; instead, the aggregate coating should be assessed at the planned mixing temperatures of WMA to evaluate the suitability of the selected mixing temperature.

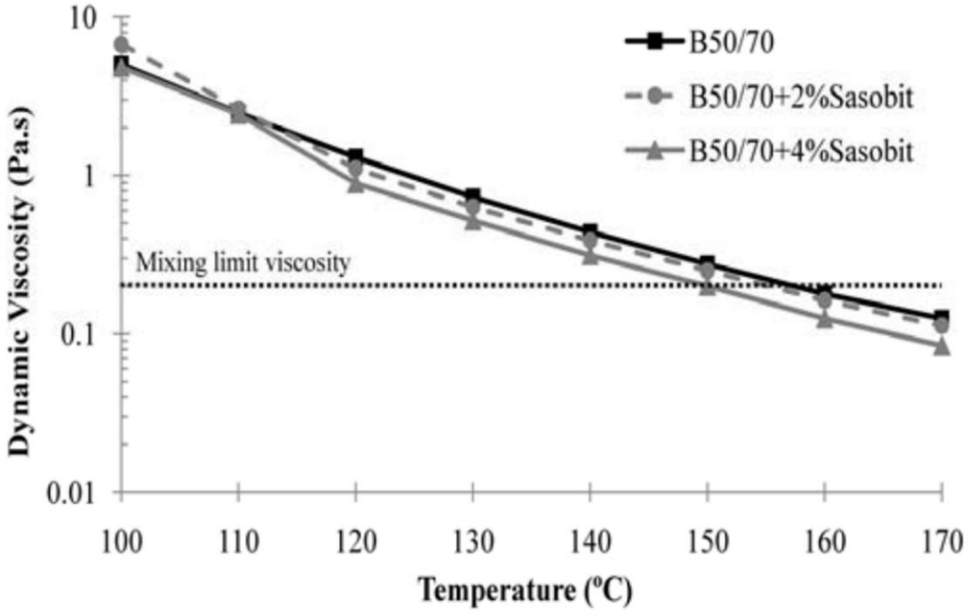


Figure 2-5. Mixing temperature reduction based on the equi-viscous method (Silva et al., 2010a)

A

B

Figure 2-6. Workability quantification devices. A University of Hampshire device. B gyratory compactor with shear stress measuring cell

2.6.2 Determination of WMA Compaction Temperature

On the other hand, the compaction temperatures of asphalt should be carefully designed. Compacting at temperatures higher than the optimum can make the asphalt creep in front of the compactor as bitumen viscosity is still low, which may affect the thickness of the asphalt layer. However, compacting at temperatures lower than optimum may lead to a decreased density as bitumen viscosity is high at this stage, which may weaken asphalt performance due to the increased air voids content. Accordingly, the compaction temperature of asphalt must be carefully designed.

Determination of compaction temperature of HMA is relatively a straightforward practice; the temperature that corresponds to (unmodified) bitumen viscosity of 0.28 Pa.s is the optimum compaction temperatures. For WMA technologies, however, some alternative methods have been developed in the literature since viscosity measurements have been reported invalid, as explained earlier. Since achieving a specific density is the ultimate aim of the compaction process, then this idea has been used to determine optimum compaction temperatures of WMA; this process is called equi-volumetric. This process involves compacting WMA samples at different reduced temperatures under the same compaction effort as a reference HMA; the temperature at which WMA exhibits similar volumetric properties to the HMA is considered as an optimum compaction temperature for the WMA. This method was suggested by the German Asphalt Paving Association (2009); Figure 2-7 illustrates the determination of compaction temperatures using a Marshall hammer compactor based on this method. The figure indicates that the additive used in this study reduced compaction temperature by about 17°C. Li et al. (2016) also applied this method to determine compaction temperatures of WMA produced using Sasobit. Their study indicated that this additive reduced compaction temperature of the WMA by about 10°C.

However, depending on the volumetric properties alone in determining WMA compaction temperatures may not guarantee optimum asphalt performance. Hurley and Prowell (2005) realised the importance of integrating asphalt performance into the process of determining compaction temperatures of WMA using the equi-volumetric method. They demonstrated that reducing compaction temperatures led to higher rutting for both HMA and WMA mixtures, and the increased rutting was attributed to the reduced binder ageing due to the reduced compaction temperature rather than changes in air voids content. This means that following an equi-volumetric method alone to determine the compaction temperatures of WMA may not guarantee adequate asphalt performance.

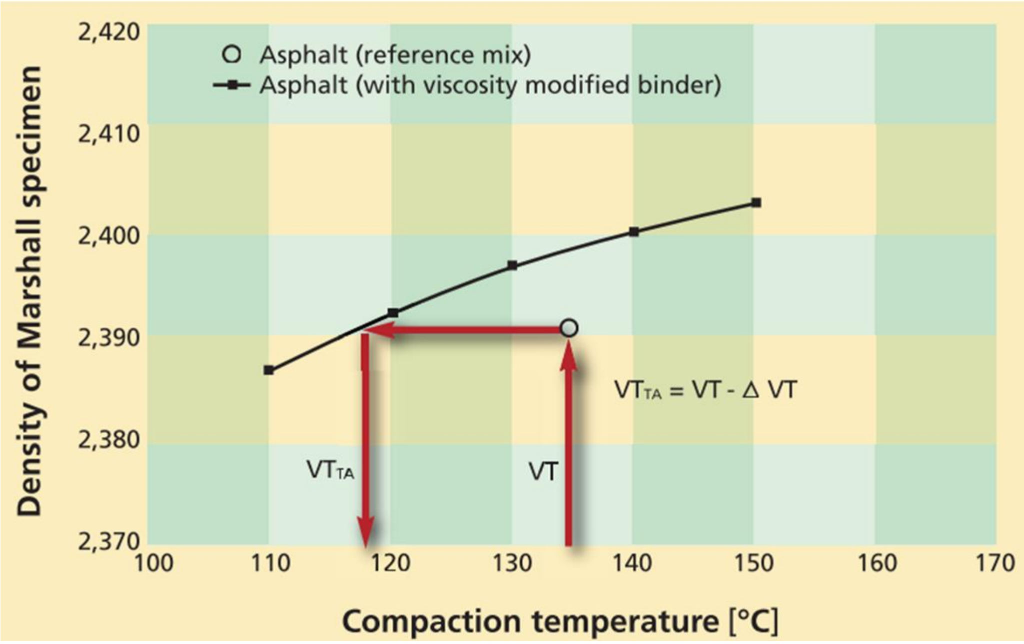


Figure 2-7. Equi-volumetric compaction temperature determination (German Asphalt Paving Association, 2009)

2.7 Reclaimed Asphalt Pavement

Reclaimed Asphalt Pavement (RAP) is the portion of asphalt that is removed at the end of its service life. It consists of aggregate and usually significantly aged bitumen. Recycling of this material has taken place since 1915; however, in 1970 it started receiving more attention due to the inflation of oil prices caused by the Arab oil ban (Kandhal and Mallick, 1997). The aim was to recycle existing asphalt to reduce the construction cost of asphalt pavements. RAP can be considered as a valuable alternative source of bitumen and aggregate because recycling this material in asphalt mixtures leads to reducing the use of virgin bitumen and aggregate (Copeland, 2011). RAP has several valuable economic and environmental benefits, including (Kandhal and Mallick, 1997):

1. Reducing the production cost of asphalt. In the HMA industry, material cost comprises 70% of the total production cost of asphalt (Copeland, 2011).

When RAP is recycled, however, a minimum of 50% of the material cost is saved when 100% RAP is recycled as shown in Figure 2-8 (Zaumanis et al., 2014), due to reusing old bitumen and aggregate in the new asphalt mixtures.

2. Conservation of the raw aggregate and bitumen. When RAP is recycled, RAP binder and aggregate reduce the amount of virgin binder and aggregate required for asphalt. This means that raw bitumen and aggregate are saved and can be used in the future.

3. Maintaining of the in-service road geometrics. When asphalt is milled and recycled, it allows for laying a new asphalt pavement layer without significantly altering the profile of the pavement. This point is particularly important when maintaining existing pavement geometry is required, such as for clearance under bridges.

4. Protection of the environment. Old asphalt is a solid waste; unless it is recycled using the best available practices, it will be dumped in landfill. Moreover, recycling RAP means significantly reducing the negative environmental impact resulting from producing raw bitumen and aggregate. This is because, during the production phase of the raw materials, a massive amount of pollutants is emitted to the air as a result of burning fuel required to operate machines and equipment used to produce these materials. Lee et al. (2010) stated that a reduction of 20% in CO_2 emissions is achieved when using recycled materials as a result of reducing emissions resulting from the production phase of these materials, whereas Zaumanis et al. (2016) stated

that CO_2 reduction could be increased up to 35% when 100% RAP is recycled.

5. Saving of energy. Recycling RAP means reducing the amount of virgin materials required in new asphalt mixtures, which in turn means saving the energy required to produce the raw materials. This means that considerable quantities of fuel that are required to produce raw materials will be saved, and this point constitutes a significant positive economic impact for the asphalt industry. Lee et al. (2010) stated that 16% less energy is required when RAP is recycled.

RAP recycling benefits have multiple positive economic and environmental benefits. However, there are also some factors and technical issues that hinder or reduce the incorporation of this material in new asphalt mixtures, including (Lo Presti et al., 2016, Baghaee Moghaddam and Baaj, 2016, Zaumanis et al., 2014):

1. Variability of RAP properties among different sources and even within the same source of RAP.
2. Mistrust and uncertainty regarding the performance of asphalt mixtures containing RAP, especially those that contain high levels of incorporation.
3. The complexity of the mix design when RAP is incorporated.
4. Technical issues regarding RAP incorporation and capacity of asphalt plants, which limits RAP recycling to a maximum of 50%.

The mistrust in the performance of asphalt mixtures containing RAP is a critical point that has been stressed in the literature and has led to the limiting of RAP incorporation level. On fatigue cracking performance, several researchers have pointed out that RAP incorporation makes asphalt mixtures susceptible to that kind of failure (Mogawer et al., 2015, Willis et al., 2012). The reason for this drop in fatigue performance has been attributed to ageing and stiffness RAP binder, the high stiffness of RAP binder, making it lose its elastic recovery properties, which eventually reflects on fatigue resistance performance. On low temperature cracking performance, the presence of RAP binder changes the failure mode from ductile to brittle; it also increases the critical low temperature cracking (Mensching et al., 2014, Mcdaniel et al., 2000). This means asphalt pavements containing RAP are prone to exhibit low temperature cracking at temperatures higher than asphalt pavements that do not incorporate RAP.

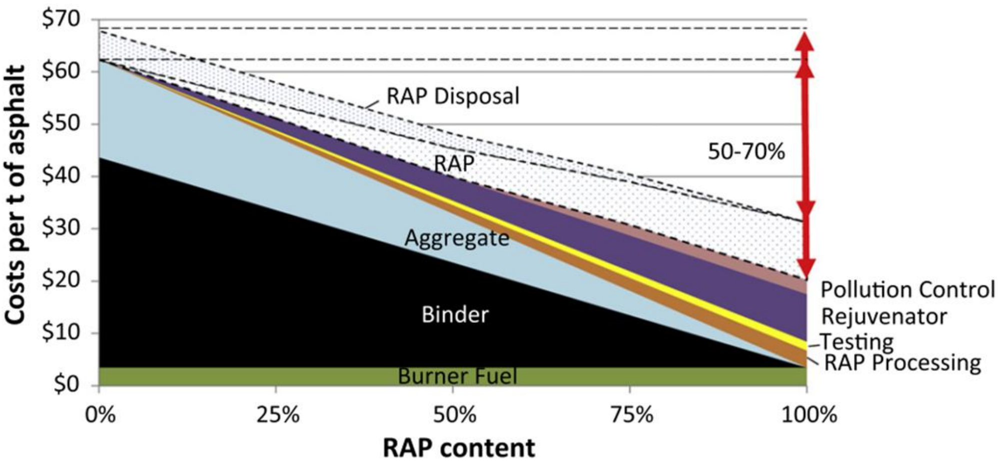


Figure 2-8. Asphalt costs and RAP economic benefits (Zaumanis et al., 2014)

2.7.1 RAP Characterisation

RAP can be incorporated into new asphalt mixtures to decrease the amount of raw binder and aggregate. Before incorporating RAP, however, it is essential to identify its engineering properties and include them in the mix design process and to assure a comparable performance to conventional HMA. Those include RAP binder content, RAP aggregate particle size distribution, RAP aggregate consensus properties such as the angularity of the coarse and fine particles, RAP binder properties (West, 2015). RAP binder content is a direct input to the mix design; based on this content and the assumed level of RAP binder participation, the rest of the binder can be calculated, and the percentage of each binder can be determined, which can then be used to establish binder blending charts. The other importance of RAP binder content is that it gives an indicator of the participation level of the binder. For instance, in Japan, a minimum of 3.8% RAP binder content is required in order to allow for incorporation of RAP in asphalt (West and Copeland, 2015). This is probably because lower than that level, the participation of RAP binder in the total mix will be lower than the expected or designed participation, which means the resulting asphalt mix will probably be dry. RAP aggregate gradation is also a direct input in the asphalt mix design process. To comply with a particular standard aggregate gradation, the particle size distribution of RAP aggregate is required, so when combining RAP aggregate with the virgin aggregate, the overall grading meets the grading requirements of the mix design standard. The properties of RAP aggregate particles such as crushed aggregate content, aggregate angularity, and dust proportion are essential indicators about the effect of RAP aggregate on critical measures such as mix volumetric properties and performance (Mcdaniel et al., 2000). However, these properties should be evaluated after blending RAP aggregate with virgin aggregate to meet the required properties of combined aggregate rather than RAP aggregate alone (Mcdaniel and Anderson, 2001).

One of the critical properties of RAP is its bitumen characteristics with particular attention paid to the ageing of RAP binder. Ageing of bitumen is a complex physicochemical process takes place over time, starting from the stage of heating and mixing asphalt at a plant until the end of asphalt life. Researchers have identified different mechanisms for bitumen ageing such as oxidation, evaporation, and absorption (Mastrofini and Scarsella, 1999, Thom, 2014). The overall effect of bitumen ageing is an increase in binder stiffness and loss of bitumen recovery properties which makes asphalt susceptible to particular distress types such as fatigue cracking and low temperature cracking (Barco Carrión et al., 2015, Baghaee Moghaddam and Baaj, 2016). This means RAP bitumen must be rejuvenated to recover its initial properties if comparable performance to conventional HMA is to be achieved. One of the widely applied methods to rejuvenate RAP bitumen is to add a softer bitumen. When the binders blend, the soft binder rejuvenates the RAP binder, while the aged binder stiffens the soft binder (Mcdaniel et al., 2000). The blending ratio of the two binders depends to a high extent on the properties of RAP binder. Accordingly, it is essential to characterise RAP binder before commencing any mix design. Other RAP binder characteristics include bitumen penetration grade, softening point, rheological properties, and low-temperature behaviour, as shown in Figure 2-9. This methodology was suggested by Lo Presti et al. (2016) to fully characterise RAP bitumen and the design binder blend in the mix. Based on RAP incorporation level, all or some of these properties are required to design the bitumen blend of the RAP and virgin binders.

2.7.2 RAP Incorporation Level and Mix Design

RAP incorporation level differs from one country to another based on the legislation and local experience with RAP recycling methods. It also depends on the structural layer that RAP is recycled into. In surface layers, recycling levels are still low at around 15%, because these layers are in direct contact with traffic loading; therefore, they require high distress resistance (West et al., 2016). In the lower layers, more RAP may be incorporated, up to 50% (Copeland, 2011). Nevertheless, RAP recycling in asphaltic mixtures has generally been limited to 10-30%. Until 2006, RAP incorporation in asphalt mixtures was limited to approximately 15% because at this level of RAP no binder grade change was required which reduces the costs of binder testing to characterise RAP binder properties (Copeland, 2011). However, due to the economic and environmental benefits of RAP recycling and the increasing costs of virgin bitumen and aggregate, highway agencies have become more willing to increase the level of RAP incorporation. In 2014, the average RAP recycling level in the US was about 20% (Hansen and Copeland, 2015). More recently, different studies have demonstrated the feasibility and capability of incorporating 100% RAP in asphalt mixtures (Zaumanis et al., 2014, Lo Presti et al., 2016, Dinis-Almeida et al., 2016).

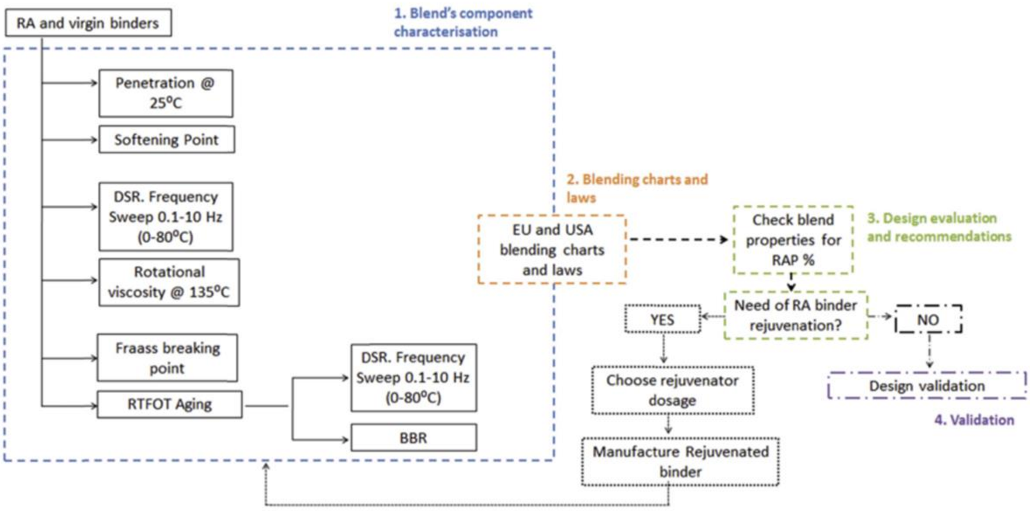


Figure 2-9. RAP binder characterisation methodology (Lo Presti et al., 2016)

From a binder blending point of view, the level of incorporation is directly related to the percentage of RAP, which determines the percentage of the hard binder in the mix. The presence of hard bitumen can change the mechanical properties of the designed mix due to its effect on the properties of the soft binder when the two binders are blended. Accordingly, highway agencies have set some guidelines to adjust mix design in case of RAP incorporation. According to the Superpave mix design specifications for RAP incorporation, three levels of RAP were determined: less than 15%, 15-25%, and more than 25% (Mcdaniel et al., 2000). The low RAP dosage requires no virgin binder grade change as the RAP binder content is quite low in this case and will minorly affect the virgin binder properties. The intermediate RAP incorporation level involves reducing the grade of the virgin binder by one increment (6°C based on the PG grading system) at both the high and low-performance grades. However, at higher RAP dosages, blending charts are recommended as the effect of RAP binder becomes significant due to its increased percentage in the mix. Later, the Superpave specifications were slightly modified, and the low PG of the RAP binder was taken into account when designing virgin binder grade (Mcdaniel and Anderson, 2001), as shown in Table 2-4. The critical low temperature of the RAP binder was found to be an important factor in limiting RAP percentage; the higher the low critical temperature, the stiffer the RAP binder, and the higher the susceptibility of the designed mix to low temperature cracking. However, Al-Qadi et al. (2009) conducted a study to determine the amount of active RAP binder when different percentages of RAP (0, 20, and 40%) are recycled; they concluded that incorporating up to 20% of RAP did not require adding a softer bitumen, and up to 40% of RAP required two PG levels softer bitumen. More recently, Mcdaniel et al. (2012) showed that recycling up to 25% RAP did not require a change in the virgin binder grade. They also stated that this method of recycling did not adversely affect the low-temperature performance of that mix. In Europe, BS EN standard 13108-1-2006 (BSI, 2006) determines the level of RAP recycling based on the structural layer that will be constructed. For surface courses, 10% of RAP is allowed without any design modification, whereas 20% is allowed for binder courses and base courses.

Table 2-4. Superpave specifications for RAP mix design

Recommended virgin binder grade	Recovered RAP binder low critical temperature °C		
	-22	-16	-10
	RAP %		
No binder grade change	<20%	<15%	<10%
One PG grade softer	20-30%	15-25%	10-15%
Use blending charts	>30%	>25%	>15%

From a mix design point of view, there are two ways to design asphalt mixtures containing RAP. The first is to design based on a known RAP percentage; the second is to design based on a known virgin binder grade (Al-Qadi et al., 2007). The first method allows the designer to select RAP incorporation level based on the desired criteria; then based on that binder blending can be designed, and the soft binder grade can be determined. The second method is useful when the soft binder properties are known because they can be used to calculate the percentage of RAP that can be incorporated in the mix. Both methods require the construction of binder blending charts when RAP percentage in the mix exceeds threshold limits identified by highway agencies. To construct binder blending charts, however, there are two frequently used methods, as follows:

2.7.2.1 Superpave Method

The NCHRP report 452 (Mcdaniel and Anderson, 2001) describes two procedures to design asphalt mixes containing RAP, one for a predetermined RAP content and one for identified virgin binder properties. For the known RAP recycling level, RAP binder should be extracted and recovered, and its critical high and low temperatures should be determined. Then the Performance Grade (PG) of the virgin binder can be calculated using the following equation:

$$\mathrm{T_{virgin}} = \frac{\mathrm{T_{blend}} - (\mathrm{RAP\%} \times \mathrm{T_{RAP}})}{(1 - \mathrm{RAP\%})} \qquad \text{Eq. 2-1}$$

where $\mathrm{T_{virgin}}$ is the virgin binder critical temperature that should be calculated at high and low temperatures to determine the required virgin binder PG, RAP% is the content of RAP in the mix, and $\mathrm{T_{RAP}}$ is the corresponding critical temperature of RAP binder. For the known virgin binder properties, RAP incorporation level that satisfies PG requirements of the target binder grade can be determined as follows:

$$\mathrm{RAP\%} = \frac{\mathrm{T_{blend}} - \mathrm{T_{virgin}}}{\mathrm{T_{RAP}} - \mathrm{T_{virgin}}} \qquad \text{Eq. 2-2}$$

2.7.2.2 European Method

In Europe, if more 10% RAP is recycled in surface courses or more than 20% in binder and base courses, then the BS EN standard 13108-1:2006 states that either the penetration or the softening point of the resulted binder should satisfy the requirement of the selected paving grade bitumen, as follows:

$$log\,pen_{blend} = a * log\,pen_{RAP\ binder} + b * log\,pen_{virgin\ binder} \qquad \text{Eq. 2-3}$$

$$T_{R\&M\ mix} = a \times T_{R\&B\ RAP\ binder} + b \times T_{R\&B\ virgin\ binder} \qquad \text{Eq. 2-4}$$

where a and b are the proportions by mass of the RAP binder and the virgin binder with respect to the total binder content (a+b=1), $\mathrm{pen_X}$ is the penetration grade of designated binder, and $\mathrm{T_{R\&M\ mix}}$ is the softening point of the designated binder.

2.7.3 Degree of Blending

Interaction between RAP binder and virgin bitumen is a critical factor that has a significant impact on mix design and mechanical properties. If the two binders reach full blending during an asphalt mixing process, then the two binders perform comparably to the target binder resulting from the blending process. If no blending occurs during the mixing, however, then the resulting binder will be inhomogeneous, and mix performance will be affected by the properties of the two binders rather than one thoroughly blended binder. Theoretically, this phenomenon has been described in the literature as the Degree of Blending (DoB); it is basically a term used to describe the quality of the blending between the RAP and virgin binders. The DoB is entirely related to the way that RAP binder acts in the mix, whether it is a very aged binder, an active binder that can’t be blended, or an active binder that can be blended with the rejuvenator. In fact, the RAP incorporation limits discussed in the previous section are based on the expected DoB at different RAP percentages. The DoB is expected to be insignificant at low RAP recycling levels because there is a small amount of RAP binder to affect the properties of the virgin binder (Mcdaniel et al., 2000). Therefore, no binder change is required. However, at intermediate RAP dosages, limited DoB can be achieved during mixing, so one grade softer binder is recommended to reflect the effect of binder blending on the resulting binder properties. However, at high RAP recycling levels, the recommendation by highway agencies is to use blending charts to determine the grade of the virgin binder (Zofka et al., 2004). This suggests that the DoB is about 100% because the virgin binder fully interacts with the hard binder, and the resulting binder controls the mechanical performance of the mix.

Regardless of RAP percentage, DoB is theoretically between 0-100%, where 0% stands for the black rock case, and 100% represents the full blending scenario (Shirodkar et al., 2011). However, the black rock assumption has been disproved by several studies (Mcdaniel et al., 2000, Mcdaniel and Anderson, 2001, Al-Qadi et al., 2007, Abed et al., 2018) as these studies have clearly demonstrated that there is a blending level between the RAP binder and the virgin binder. Accordingly, researchers have pursued this critical topic and suggested different methods to quantify the DoB and its consequences on the mix design process. Bonaquist (2005) developed a method to assess DoB by dynamic modulus measurements. The method stated that the RAP binder should be extracted, recovered, and blended with the virgin binder, then the resulting binder should be characterised, and the mix dynamic modulus is estimated by using the Hirsh model based on binder characteristics and mix volumetric properties. After that, the actual dynamic modulus of the mix is determined by running dynamic modulus testing; if there is sufficient overlap between the estimated and measured dynamic modulus master curves, then the DoB is sufficient. Otherwise, there is no full blending between the two binders. This method, however, requires RAP binder extraction, recovery, characterisation, and asphalt dynamic modulus testing which is not available in all asphalt laboratories. Also, the dynamic modulus results may not reflect DoB in the mix due to the compressive forces among aggregate particles (Al-Qadi et al., 2009), which means the aggregate may affect the test results and eventually the DoB results.

Another method was developed by Shirodkar et al. (2011). They suggested an iterative procedure to calculate the DoB based on the Superpave critical high-temperature parameter $G^*/\sin\delta$ for binders recovered from predefined sizes of RAP and virgin aggregate particles. Their method was to assume an initial percentage of RAP binder that will be active in the mix, then design and manufacture samples based on this assumption, after that to sieve and isolate RAP from aggregate particles, and extract and recover binder from both portions to run DSR testing. The DoB was then determined using the following equation:

$$DoB(\%) = 100 \times \left|1 - \frac{|CHTP_{vab} - CHTP_{rab}|}{|CHTP_{vb} - CHTP_{rb}|}\right| \qquad \text{Eq. 2-5}$$

where CHTP is the critical high-temperature parameter (G*/sinδ) for the designated binder, *vab* is the binder recovered from the virgin aggregate, *rab* is the binder recovered from RAP aggregate, *vb* is the virgin binder, and *rb* is the RAP binder. If the calculated DoB is close to the assumed percentage of RAP binder participation then the correct DoB has been obtained; if not, then a new value for RAP binder participation has to be assumed, and DoB has to be calculated again until these two values converge. They reported 70% and 96% DoB for 25% and 35% RAP incorporation respectively. The limitation of this method is that at least three iterations are required before obtaining a reliable DoB result. Also, this procedure is to determine DoB at one mixing condition. Since DoB is a function of different production conditions such as mixing time, temperature, rejuvenator and aged binder properties (Zaumanis and Mallick, 2013b), then this procedure is impractical as it takes a lot of trials and material to study effects of production conditions on DoB. Coffey et al. (2013) defined the DoB as the mobilisation percentage of RAP binder in an asphalt mix. They estimated the DoB based on air voids content by assuming the initial DoB is 70% based on literature; if this assumption was true, then the air voids at 75 gyrations has to be 4%, if not then the binder content has to be adjusted based on a revised DoB percentage. They reported a DoB of 84.7 to 90% for three different RAP sources with 25% RAP incorporation. This level of RAP incorporation, however, was relatively low and reported in literature to have a minor impact on the design mix. Also, this method requires some iterations until the assumed and calculated DoB converge.

Furthermore, the DoB is not a constant property; there are different factors that can affect and change it. On the one hand, the mobilisation of RAP binder during mixing is significantly related to its viscosity, and since the viscosity depends on the binder grade and temperature then logically mixing temperature should have a significant impact on DoB. On the other hand, mixing time is another critical factor that can affect the DoB by mechanical blending during asphalt mixing. Increasing mixing time is expected to increase the blending as there is more time for the binders to contact and interact with each other while their viscosities are still relatively low.

Navaro et al. (2012) studied the effect of the mixing time and temperature on binder blending of an asphalt mixture containing 70% RAP produced at three temperatures 110, 130, and 160°C and mixed for different times between 20 seconds and 10 minutes. They studied binder blending by an image processing method and reported that these two parameters have a significant impact on the homogeneity of the mix, and demonstrated that decreasing mixing temperature from 160 to 130°C required doubling to tripling mixing time to produce a mix with the same homogeneity. Bowers et al. (2014) studied the effect of the same parameters on the blending and diffusion process between RAP and virgin binders. Predefined RAP and virgin aggregate sizes (RAP aggregate between 9.51-2.38 and virgin aggregate between 19-12.7mm) used in mixing, then binders from these aggregate particles were extracted by means of staged extraction in order to characterise properties of the binders extracted from different layers. Gel Permeation Chromatography and was used to study the properties of the recovered binders based on the molecular weight distribution of the bitumen. They concluded that mixing temperature has a critical effect on DoB, whereas increasing the mixing time up to 2.5 minutes improved the DoB, but beyond that, there was a limited impact on the DoB. Hassan et al. (2015) studied the effect of mixing time on the homogeneity and particularly the distribution of air voids in asphalt samples containing 15% RAP. X-ray Computed Tomography and optical microscopy were used in this study to quantify and analyse air voids. The researchers stated that increasing up to double the mixing time resulted in a better air voids distribution in the studied specimens.

To sum up, the DoB is a critical factor that affects volumetric properties and mechanical performance of RAP containing mixtures. There is no globally accepted technique, however, to determine this property, but various methods have been developed by different researchers. Also, the DoB can be affected by several factors, including RAP%, RAP binder properties, and production conditions of the asphalt.

2.7.4 RAP Recycling Methods

RAP recycling methods can be classified based on recycling location into in-plant and in-place recycling or based on production temperature into hot, warm, and cold recycling (Karlsson and Isacsson, 2006). The hot recycling methods generally involve indirect heating of RAP by superheating the virgin aggregate before mixing; when the aggregate contacts the RAP it transfers the heat energy by conduction which dries the RAP and softens the RAP binder. After that, the virgin binder, aggregate, and recycling agents if included are mixed, and the mixture is transported (in case of in-plant recycling), laid and compacted using traditional

HMA construction equipment. The main drawback of this method is that it is not environmentally friendly due to the high energy consumed during the production process (Recycling and Association, 2001). Because mixing and compaction temperatures of asphalt mixtures containing RAP are higher than those of the traditional HMA (Jamshidi et al., 2013) this also leads to high CO_2 emissions (Oliveira et al., 2012). This means that the environmental impact of the hot recycling process is more harmful than that of HMA without RAP. The other primary concern is excessive RAP binder ageing which causes significant technical problems with workability and compactability of the mix and limits the recycling percentage of RAP (Tao and Mallick, 2009, Tutu and Tuffour, 2016).

Cold recycling, on the other hand, involves reusing RAP at ambient temperatures. The main advantage of this method is the low mixing temperature, which significantly reduces the energy requirement during the recycling process. However, one of the primary disadvantages of this method is the inability to blend RAP binder due to the low production temperatures. Thus the RAP is treated as a black rock in the cold recycling process (Kandhal and Mallick, 1997) which reduces the economic impact of the recycling. Other drawbacks found in the literature include a long curing time to gain strength if asphalt emulsions are used, weak mechanical performance, high air voids content, or using RAP in base and sub-base layers which reduces the value of the RAP.

However, after the introduction of WMA technologies, warm recycling of RAP has become an interesting topic because it combines the described economic and environmental benefits of the recycling process and WMA technologies. On the one hand, the production temperature range of WMA is still sufficient to activate RAP binder. This means that the RAP need not be treated as black rock in warm recycling. On the other hand, WMA additives can help in reducing binder viscosity or increasing mixture workability depending on the nature of the additives; both impacts suggest the ability to incorporate more RAP in WMA mixtures. NAPA (2009) stated that incorporating 25% RAP in WMA can further decrease the environmental impact of the asphalt industry by about 15-20%. Vidal et al. (2013) pointed out that one important benefit of WMA is the possibility of incorporation of higher dosages of RAP. Giani et al. (2015) conducted a comprehensive life cycle assessment study on warm and cold recycling methods and stated that the key to achieving a sustainable asphalt pavement is by combining RAP recycling and WMA technologies. Kim et al. (2017) stated that the demand for this type of asphalt has recently critically increased due to the expected economic and environmental benefits of these technologies.

The presence of WMA additives can allow the production of asphalt mixtures containing higher percentages of RAP in two respects (D'angelo et al., 2008, Mallick et al., 2008). First, the aged RAP binder typically causes difficulties in the compaction process due to its high viscosity. Therefore, WMA additives can facilitate the compactability by reducing either bitumen viscosity or increasing workability of these mixes. Second, the reduced WMA production temperatures can help in reducing binder ageing during mixing, which compensates for the aged bitumen and encourages the use of higher dosages of RAP.

2.7.5 Warm Mix Recycled Asphalt (WMRA) Performance

The presence of aged RAP binder, virgin binder, and WMA additives in one mix can make the behaviour of that kind of asphalt quite challenging. On the one hand, the interaction between aged and virgin binders is still not well understood, and there is no standard method to evaluate or assess this phenomenon, and this problem can be complicated when RAP is recycled within the WMA production temperature window because the low production temperatures increase RAP binder viscosity which may reduce the DoB and affect mixture performance. On the other hand, WMA additives can further increase the complexity of this situation, because the effect of these additives is different from one to another and might depend on the chemistry of the bitumen. Also, it is not clear if the additives can modify RAP binder or only virgin bitumen. Accordingly, this area has been investigated by several researchers and different conclusions have been reported in the literature.

Mallick et al. (2008) studied the feasibility of recycling 75% RAP using Sasobit and a soft binder with PG 42-42 used as a rejuvenator and added at a rate of 1.5% by weight of the total mix. The authors reported that despite the high RAP content, the required density was achieved at the same compaction effort as the control mix due to the presence of the additive. They also pointed out the importance of balancing the stiffening effect of the additive with the grade of the soft binder to improve the resulting mixture performance. However, the blending between the hard and soft binder was not investigated in this study.

Tao and Mallick (2009) investigated the workability, volumetric properties, and mechanical performance of recycling 100% RAP in base courses using different dosages of wax and foaming additives. The workability was evaluated by measuring the torque required to spin a blade in the mix, whereas seismic moduli and indirect tensile strength were used to evaluate mechanical performance of the mixes. The results showed that both additives improved the workability at compaction temperatures as low as 110°C; they also demonstrated that the additives increased the moduli and the strength due to their stiffening effect. Accordingly, the authors recommended investigating the low-temperature cracking and the long-term performance of that kind of asphalt.

Oliveira et al. (2012) investigated the performance of asphalt containing 50%RAP recycled using a chemical WMA additive (surfactant). They demonstrated that this mixture exhibited comparable or slightly enhanced rutting resistance, increased stiffness modulus, and improved fatigue life. The authors also mentioned that this kind of asphalt is still in its infancy and requires more investigation and development to improve its performance. However, binder blending charts were not followed in this study. Also, it was not clear if the RAP binder was treated as black rock or active.

Vargas-Nordcbeck and Timm (2012) explored the rutting performance of WMA and WMA with 50%RAP in full-scale test sections in Alabama/ US. The main conclusion of this study was that WMA technologies tended to increase rutting potential, whereas the presence of RAP reduced rutting susceptibility due to the stiffening effect of the RAP binder. However, it was not clear if the RAP was assumed to behave as black rock or active, also it was not clear how the production temperatures of WMA and WMRA were designed in this study.

Howard et al. (2013) investigated the mechanical performance of asphalt mixtures containing 50-100% RAP recycled within the WMA production temperature window. Six mixtures were studied, two contained 50%, two 75% and two 100% RAP; one WMA technology (Sasobit) and one virgin binder grade (PG 67-22) were used in this study, and all mixtures were mixed and compacted at 116°C. Performance evaluation results showed that the rutting resistance of WMRA was satisfactory. But based on other measures such as fatigue cracking and low temperature cracking it was concluded that WMRA with 50% performed the best. However, using the same binder grade with different RAP contents may not be reasonable as the stiffening effect of the aged binder on the virgin binder can be significant. Therefore, it would not be surprising that the produced mix was susceptible to fatigue and low temperature cracking.

Guo et al. (2014) recycled 40% RAP using two WMA additives, namely Evotherm-DAT and S-IWMA and 80-100dmm pen virgin binder as a rejuvenator. Different tests were performed to evaluate the mechanical performance of the designed mix at different mix ageing levels. Based on the performance results, the researchers concluded that the control WMA outperformed WMRA in terms of low temperature cracking and moisture resistance, but WMRA performed better regarding rutting resistance. Also, RAP incorporation critically decreased fatigue resistance. However, this study did not consider the binder design in the mix design process, which would be quite useful in interpreting and analysing testing results.

Xiao et al. (2016) implemented hot and two warm recycling methods (foaming and Evotherm additives) to incorporate 20%, 30%, 40%, and 50% RAP obtained from two sources. Two virgin binders were used in this study, PG 64-22 and 58-28, in mixtures containing 40% and 50% RAP. The authors focused on the effect of RAP percentage and WMA technology on the volumetric properties of the studied mixtures and concluded that RAP incorporation did not change the optimum binder content regardless of its level. They also mentioned that the filler to binder ratio increased as the RAP percentage was increased. However, no binder blending charts were constructed in this study, and it was not clear if the RAP was assumed to be black rock or fully active. Furthermore, since most of the volumetric properties discussed in this study met the Superpave requirements at all RAP percentages, then it can be concluded that asphalt performance such as cracking or rutting can be better indicators than the volumetric properties as these measures have been used in many studies and usually can discriminate between the performance of these kinds of asphalt.

Kim et al. (2017) evaluated rutting and fatigue cracking performance of WMA mixtures with and without RAP produced in an asphalt plant and compacted in a laboratory. The results indicated that incorporating RAP between 15%-50% reduced fatigue life, and that was attributed to the increased stiffness of the mixtures when RAP was included. The presence of WMA additives which were Sasobit, Advera, and Evotherm, however, improved fatigue life but this conclusion was made based on one RAP percentage of 35%. With regard to rutting performance, the presence of RAP decreased permanent strain, indicating improved rutting performance. The presence of Evotherm and Advera actually increased permanent strain, which shows that these additives have a binder softening effect. The Sasobit however, improved rutting performance due to its binder stiffening effect. Generally, in this study, no soft binder or any other rejuvenator was used to rejuvenate the RAP. Moreover, it was not clear if the RAP was assumed to behave similar to black rock or active.

Wang et al. (2017) studied performance of binders containing 30% to 70% artificially aged binder blended with wax and surfactant WMA additives. The rotational viscosity, Superpave parameters and low temperature creep stiffness measures were used to assess performance. The results indicated that increasing the aged binder content increased viscosity, critical high temperature, fatigue and low temperature cracking susceptibility due to the increased binder stiffness. Moreover, the chemical additive improved low temperature cracking resistance, whereas the wax additive improved fatigue cracking and rutting resistance. However, only Superpave parameters were used in this study to evaluate binder performance. Other studies (Airey, 2004, Bahia et al., 2010, Subhy, 2017) have shown that modified binders exhibit complex behaviour and using Superpave parameters to characterise the behaviour of these binders may not be valid.

Kim et al. (2018) explored fatigue cracking performance of WMA mixtures containing different percentages of RAP (20%-40%) and RAS (20%) (Reclaimed Asphalt Shingles). They followed two methods in the evaluation process, four-point bending and full-scale evaluation using an accelerated loading facility. They found that incorporation of RAP and RAS decreased fatigue life but using softer binder improved fatigue life. They also mentioned that the foaming technology improved fatigue life when 20% RAP was incorporated, whereas the Evotherm technology did not. At 40% RAP, however, neither method improved fatigue life. This means that fatigue cracking distress is a critical concern of WMA mixtures containing RAP.

Omranian et al. (2018) assessed the compactability, strength, and resilient modulus of asphalt mixtures containing 30% and 50% of RAP and produced using a WMA technology called RH-WMA, which is polyethylene wax additive developed in China. They observed that incorporating RAP reduced mix workability, increased mix stiffness and strength and that was attributed to the presence of RAP aged binder. They also demonstrated that the WMA additive decreased the stiffness and strength and increased mix compactability due to the ability of the additive to reduce binder viscosity. The authors also recommended evaluating rutting, fatigue cracking, and durability to provide a better understanding of the behaviour of that kind of asphalt.

2.8 Summary and Conclusion

This literature review has shown that there is a strong tendency for highway agencies to implement WMA technologies to achieve environmental and economic benefits. The incorporation of RAP with WMA has attracted even more attention than WMA due to the expected benefits of recycling RAP in addition to the advantages of WMA technologies. There are several reasons, however, that hinder the implementation of WMA and / or WMRA mixtures that have been found in the literature. The primary performance measures, including rutting and fatigue cracking, are major concerns of these mixtures as demonstrated by several studies. Design of these mixtures is a complex process that is affected by several factors such as the type of WMA additive, design of mixing temperature, and design compaction temperatures. This situation becomes more complicated when RAP is included as the blending between the aged and soft binders can affect the outcome of the designed mix.

The conclusions drawn based on this literature review are as follows:

1. There a significant number of WMA technologies and it seems that new WMA additives and methods are being developed. Every method has unique characteristics that make its application suitable for a particular engineering problem and specific environmental conditions. Accordingly, evaluation of WMA methods is technology dependent and cannot be generalised to all methods. However, the evaluation means are the same as for traditional HMA with some modification in the mix design and material selection processes.

2. All WMA additives work by altering binder properties such as viscosity or rheological properties which makes characterising binder behaviour and predicting its performance quite challenging. This also means that using empirical bitumen testing methods such as penetration and softening point, or the Superpave grading system in characterising bitumen behaviour may be invalid, and there is a need to investigate binder behaviour beyond the limits of linear viscoelasticity using advanced bitumen testing methods. These might include multiple stress creep recovery for rutting performance, linear amplitude sweep for fatigue cracking performance, and low temperature creep stiffness for low temperature cracking evaluation, and there is very little work done on this area. Therefore, there is a need to study the impacts of WMA additives on bitumen rheology using the suggested testing methods.

3. With regard to the design of the production temperatures of WMA, this area can be divided into two sections; design of mixing temperatures and design of compaction temperatures. The latter is related to selecting optimum compaction temperatures that allow for sufficient compaction with minimum effort. In fact, there was a PhD study at the University of Nottingham (Jalali, 2016) conducted on the evaluation of compaction temperature effect of WMA performance. The main conclusion of this study is that compacting at lower temperatures does not have a significant adverse effect on asphalt performance, even when compacting at less than 100°C. Furthermore, it seems that there is general agreement that optimum WMA compaction temperatures can be determined based on the designed air voids; the temperature at which the target air voids is achieved using the

same compaction effect as reference HMA can be selected as an optimum compaction temperature. The former (design of mixing temperatures), however, can be an important factor due to two reasons. First, binder ageing is related to the mixing temperature, and it is well known that consequential pavement distress types such as premature rutting are related to bitumen shear modulus after compaction. Second, some additives dissolve in bitumen at specific temperatures. Therefore the blending between the additive and bitumen might be affected by the mixing temperature, which can be reflected in the mix performance. Accordingly, it is critical to study the effect of mixing temperatures on WMA performance, and no study so far has investigated or correlated mixing temperature of WMA with its performance. Therefore, this area should be investigated.

4. Concerning rutting performance of WMA, several studies have stated that due to the reduced binder ageing during asphalt production, WMA could be susceptible to permanent deformation, in particular, premature rutting which appears straight after opening the pavement to traffic. Accordingly, this area has to be studied to prove or disprove the effect of the reduced ageing due to WMA production on rutting performance.

5. Fatigue cracking performance of WMA could be another concern because most of the available evaluation data are based on test sections constructed in the past ten years. However, it is well known that asphalt fatigue cracking appears after the pavement has been subjected to excessive ageing. Also, most of the published studies have evaluated fatigue cracking of WMA at one temperature, and so far, there is no published WMA fatigue cracking model that links mix stiffness (i.e. testing temperature) with the applied

strain. This model is necessary to predict fatigue cracking damage in specific applications such as the Mechanistic-Empirical Pavement Design Guide (MEPDG). Accordingly, fatigue cracking prediction models of WMA have to be developed, and fatigue cracking after long-term ageing also has to be evaluated.

6. With respect to RAP recycling at WMA production temperatures, the literature shows that this area is one of the most important topics to be investigated due to the expected combined benefits of warm RAP recycling. However, most of the studies related to this area show that RAP recycling is still limited to a maximum of 30%, with 20% being a typical value. Also, some studies have used RAP in base layers, which may not be the optimum use of this material for two reasons. Firstly, base layers can be constructed using different materials such as cement-treated bases or granular bases, whereas surface layers are usually constructed of asphalt. This means that RAP is more valuable in surface layers than base layers. Secondly, it is believed that most of the highway networks in Europe and the US are already constructed, and most of the future works will be concentrated on road maintenance rather than the construction of new roads. This means that RAP will probably be limited to being recycled in surface layers of roads. Accordingly, warm RAP recycling at intermediate and high percentages should be investigated, and the performance of that kind of asphalt must be characterised.

7. With regard to the design of WMA containing RAP, several studies have incorporated RAP but unfortunately ignored three essential factors: mixing time, mixing temperature, and the degree of blending (DoB). These factors

are essential because they have significant impacts on the homogeneity, blending between aged and soft binders, and mix performance. Accordingly, this area must be investigated, and a methodology to optimise the three mentioned parameters has to be derived with the aim of improving mixture performance by optimising these parameters.

Accordingly, this PhD aims to cover the following gaps:

1. Investigate impacts of two WMA additives commonly used in Europe, namely Cecabase and Sasobit, on binder rheology and performance by implementing advanced bitumen characterisation methods rather than traditional techniques. This objective includes analysing impacts of the selected additives on the binder ageing process and the consequences of these impacts on bitumen rheology and performance.

2. Develop a methodology to design the mixing temperature of WMA by quantifying the effect of the selected additives on the aggregate coating, and by correlating mix performance with the drop in the mixing temperature.

3. Derive a methodology to quantify DoB in the case of WMRA under different production conditions.

4. Derive a methodology to quantify impacts of DoB, mixing time, and mixing temperature on mix performance.

5. Evaluate ageing effects on mixture linear viscoelastic properties and fatigue life by performing dynamic modulus and fatigue cracking tests before and after long-term ageing of asphalt.

Chapter 3: Rheological Characterisation of WMA Additive-Modified Binders

3.1 Introduction

Although the bitumen content of conventional asphalt mixtures is about 4-7%, this material controls, to a high extent, the performance and properties of asphalt. Traditionally, bitumen had been described by some empirical properties such as penetration and softening point. Based on the climatic conditions, a specific penetration grade bitumen can be selected. For example, in the UK, a 70/100 penetration grade can be used in paving wearing courses, because usually, ambient temperatures are less than 30°C in summer. However, in hot countries with ambient temperatures about 50°C in summer, a much harder binder may be used such as 25/35 or 35/50 in order to compensate the reduced binder stiffness resulting from the high temperatures and resist permanent deformation resulting from the reduced asphalt stiffness. Accordingly, binder grade should be carefully selected to withstand the local traffic loading and climatic conditions successfully.

However, with the increase in traffic volumes and axle loads, and the development of different types of asphalt additives such as polymers, rubbers, and WMA additives, it has become necessary to characterise bitumen based on its fundamental rather than its empirical properties, especially when some of these additives can alter bitumen sensitivity to loading frequency and environmental conditions. Currently, rheological properties of bitumen (Complex shear modulus and phase angle) derived from Dynamic Shear Rheometer (DSR) testing are the most accepted method to characterise bitumen.

Since WMA additives work by changing bitumen properties in order to reduce production temperatures of asphalt and due to the critical role of the bitumen in asphalt, then it is essential to study and understand the way that these additives influence the rheology and properties of the base binder. Therefore, this chapter focuses light on the effects of the WMA additive on the base bitumen properties in order to detect and understand these effects and implement them in analysing and interpreting the behaviour of WMA performance.

3.2 Materials

3.2.1 Base Binder

In this study, a 40/60 straight-run penetration grade bitumen was used as a base binder, labelled Bb. This binder was supplied by Shell Bitumen; Table 3-1 presents the empirical properties of the base binder. This type of bitumen was selected because it is commonly used in the paving of surface layers in the UK, which makes it suitable for this study.

Table 3-1. Empirical properties of the base binder

Test	BS EN	Result	Spec.
Penetration dmm	1426:2015 BSI (2015a)	49	40-60
Softening point °C	1427:2015(BSI, 2015b)	51.2	48-56

3.2.2 WMA Additives

Two representative WMA additives were used in this study. The first one was Sasobit, which is an organic additive commonly used to produce WMA in Europe, the USA and several other countries. Figure 3-1 (a) shows a sample of this additive and Table 3-2 presents some of its properties. It consists of small, white, spherical particles with a diameter between 1-3 mm and it has an approximate melting point of 100°C. The second additive was Cecabase (Cecabase RT 945) which is a chemical additive also used in Europe and various other parts of the world to produce WMA. Figure 3-1 (b) shows a sample of this additive and Table 3-3 presents some of its properties. It is liquid at ambient temperature and has a yellow to orange colour. The mechanism of both additives to produce WMA and reduce production temperatures of asphalt was presented in chapter 2.

a b

Figure 3-1. WMA additives, (a) Sasobit and (b) Cecabase

3.3 Bitumen Modification with WMA Additives

To prepare homogeneous modified bitumen samples for binder testing or even for asphalt production, a modification process was designed and followed every time a binder modification took place. The procedure followed was heating the base binder to 150°C in an oven, preparing metal tins with the required dosage of the additives, then adding the required weight of the bitumen to the tins and blending immediately. A shear mixer (Stuart 5510) as shown in Figure 3-2 was used to blend the binder with the additives; a low shear rate of 500 rpm was applied for 5 minutes as recommended by previous studies (Hurley and Prowell, 2005, Xiao et al., 2012). The temperature was controlled through the modification process by using a hot plate with temperature control. At the end of the modification process, the modified binders were stored in small vials for further binder testing.

Table 3-2. Properties of Sasobit (Jamshidi et al., 2013, Sasol, 2016a)

Properties		Units	Specification
Quantitive	Congealing temperature	°C	Min 100
	Penetration at 25°C	dmm	Max 1
	Penetration at 65°C	dmm	Max 13
	Flash point	°C	290
	PH		Neutral
	Polydispersity index		1.33
	Density (prills)	kg/m^3	590
	Brookfield viscosity at 135°C	cP	10-14
Qualitive	Odor	Odorless	
	Visual color	White to yellow	
	Physical state	Solid pills	
	Recommended dosage, % of bitumen weight		1.5-2.25

Table 3-3. Properties of Cecabase RT 945 (Arkema Group, 2016)

Viscosity at 25°C cP	Density kg/m^3	Typical dosage, % of bitumen weight	State
600	995	0.3-0.5	Liquid

Regarding the dosage of the additives. The main dosages used in this study were 2% Sasobit (denoted by 2%SMB) and 0.4% Cecabase (denoted by 0.4%CMB) of the bitumen weight as recommended by the manufacturers of these additives. However, at bitumen testing level, an additional two dosages of each additive were included to understand how increasing or decreasing additive quantity will affect bitumen characteristics and rheology. The additional dosages were 0.5% and 3.5% Sasobit (denoted by 0.5%SMA and 3.5%SMB), and 0.2% and 0.6% Cecabase (denoted by 0.2%CMB and 0.6%CMB).

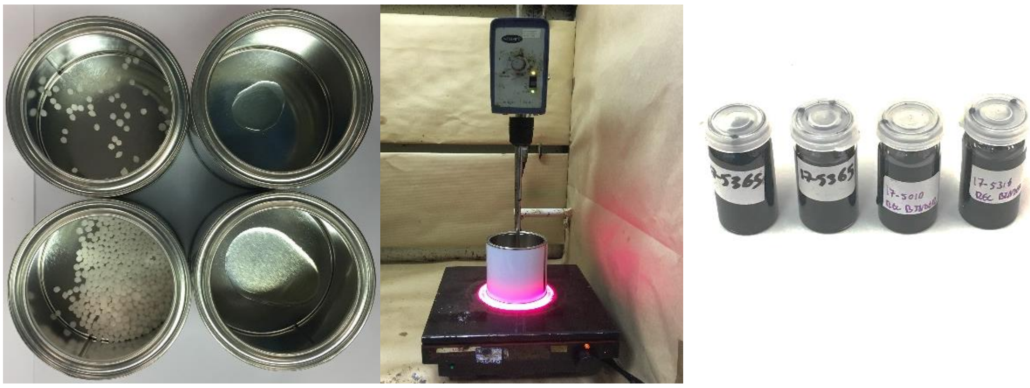


Figure 3-2. Binder modification process

3.4 Binder Testing

In order to fully characterise the effects of the studied additives, a comprehensive experimental plan, as presented in Figure 3-3 was followed. The plan includes empirical and advanced bitumen tests as follows: viscosity test to validate the equi-viscous approach to determine mixing temperatures of WMA, frequency sweep tests to detect the additive effect on the viscoelastic properties of the base binder, Multiple Stress Creep Recovery (MSCR) to study the permanent deformation resistance, Linear Amplitude Sweep (LAS) to analyse fatigue cracking performance, and low temperature creep stiffness to study the low temperature cracking performance of the studied binders.

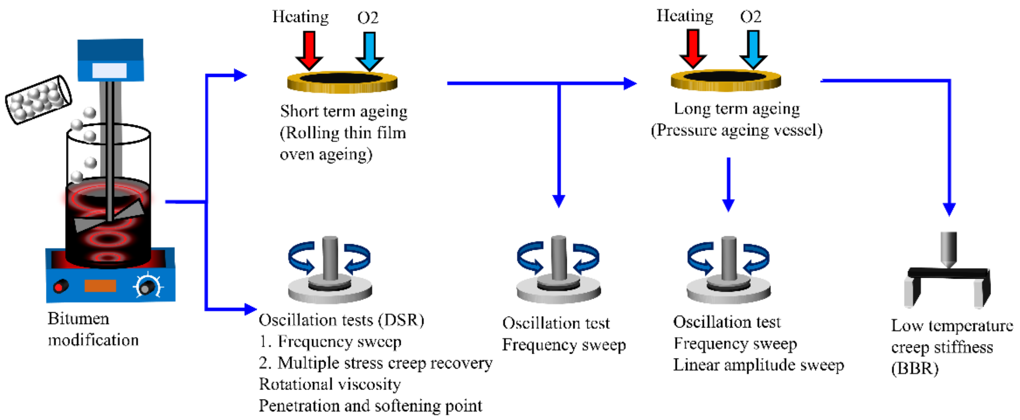


Figure 3-3. Experimental plan of bitumen characterisation

3.4.1 Penetration Test

This is an empirical test that has been widely used mainly to grade and assess the consistency of bitumen. This test involves measuring the vertical travel distance of a standard needle penetrating a bitumen sample for five seconds under a standard load and a known temperature (Hunter and Read, 2015). In this study, this test was performed according to British Standard BS EN 1426:2015 (BSI, 2015a). Figure 3-4 presents the results of this test. On the one hand, the Sasobit has a clear bitumen stiffening effect as the penetration reduced for all binders containing Sasobit. Also, this effect is inversely related to the dosage of Sasobit, the higher the dose, the lower the penetration and vice versa. On the other hand, there is no apparent effect of Cecabase on the penetration. That could be because of the very low dosages of this additive or because of the incapability of this test to detect the effect of this additive.

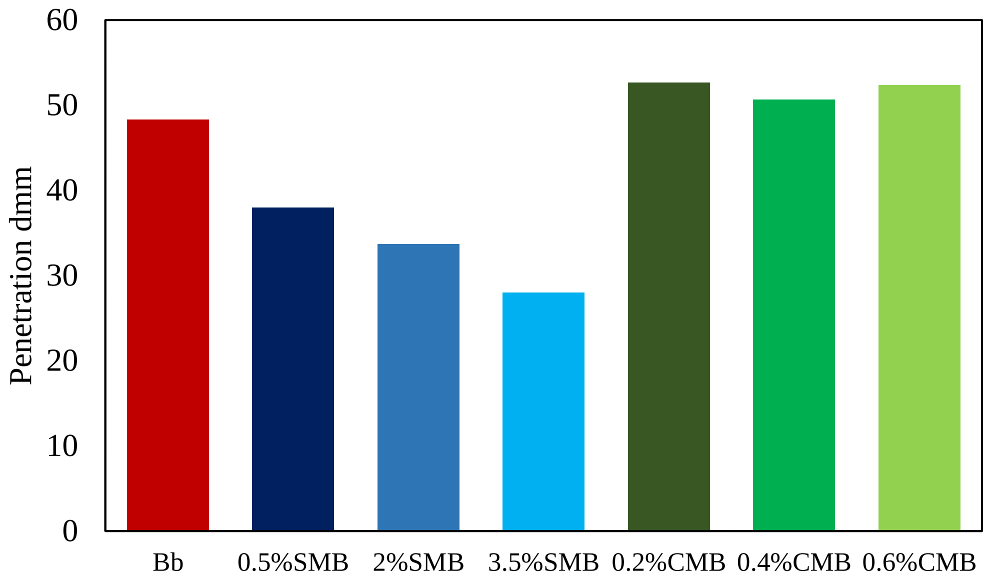


Figure 3-4. Penetration test results

3.4.2 Softening point test

This test is also an empirical test that has been used to assess the consistency of bitumen, and it determines the temperature at which the bitumen state changes from solid into a liquid. This test involves heating a disk-shaped bitumen sample placed in a brass ring under the load of a steel ball; the temperature at which the steel ball touches the base of the testing frame is considered the softening point temperature (Hunter and Read, 2015). In this study, this test was performed according to British Standard BS EN 1427:2015 (BSI, 2015b). Figure 3-5 shows the results of this test; these results correspond very well with the penetration results. There is a clear bitumen stiffening effect of Sasobit, and this effect is directly related to the dosage of the additive, whereas Cecabase did not affect this property which suggests that this test may not be useful in detecting the effect of this additive.

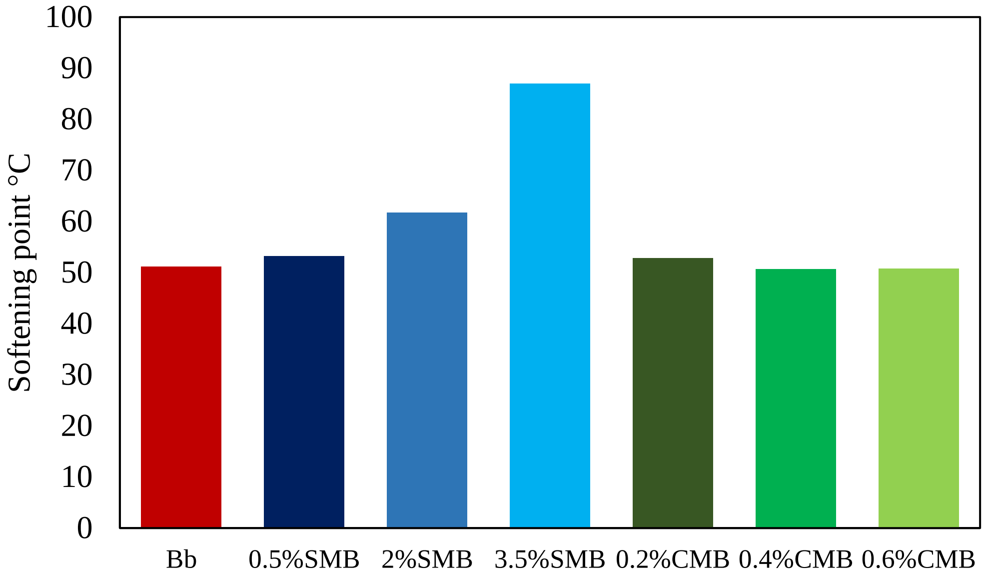


Figure 3-5. Softening point test results

3.4.3 Viscosity

Viscosity can be defined as the resistance of a liquid to flow; it can be measured as the ratio of applied shear stress to the resulting shear strain (Airey, 1997). Bitumen viscosity has many important applications including determining mixing and compaction temperatures of asphalt. This can be done by first establishing a viscosity-temperature profile, then selecting the temperatures corresponding to viscosities of 0.17 and 0.28 Pa.s as mixing and compaction temperatures (Yildirim et al., 2000). Accordingly, this approach was implemented in this study to determine the production temperatures of the used additives. Binder viscosity was measured by determining the rotational viscosity using a Brookfield viscometer according to British Standard BS EN 13302:2010 (BSI, 2010).

Figure 3-6 presents the viscosity results of SMBs in comparison with Bb. These results show that Sasobit can insignificantly reduce bitumen viscosity, even when a high dose of 3.5% is used. The results also show that Sasobit has a melting point of 100°C; lower than this temperature, this additive can increase bitumen viscosity.

Furthermore, applying the equi-viscous approach presented in section 2.6.1 reveals that Sasobit can reduce mixing and compaction temperatures by about 4°C, which disagrees with most of the cited literature. This means that the equi-viscous concept cannot be used to determine production temperatures of WMA as the reduction is quite insignificant. On the other hand, Cecabase effect on viscosity is presented in Figure 3-7. This figure indicates that Cecabase does not change bitumen viscosity, even when 0.6% of this additive is used. However, this additive is not a viscosity reducer, but it works by reducing the friction between bitumen and aggregate. This means that this result can be considered normal.

The overall conclusion of the viscosity test is the invalidity of using this test in determining rational production temperatures of WMA, and this conclusion agrees with previous studies (Wasiuddin et al., 2007, Buss, 2010, Silva et al., 2010a, Silva et al., 2010b). This means another method to determine WMA production temperatures is required.

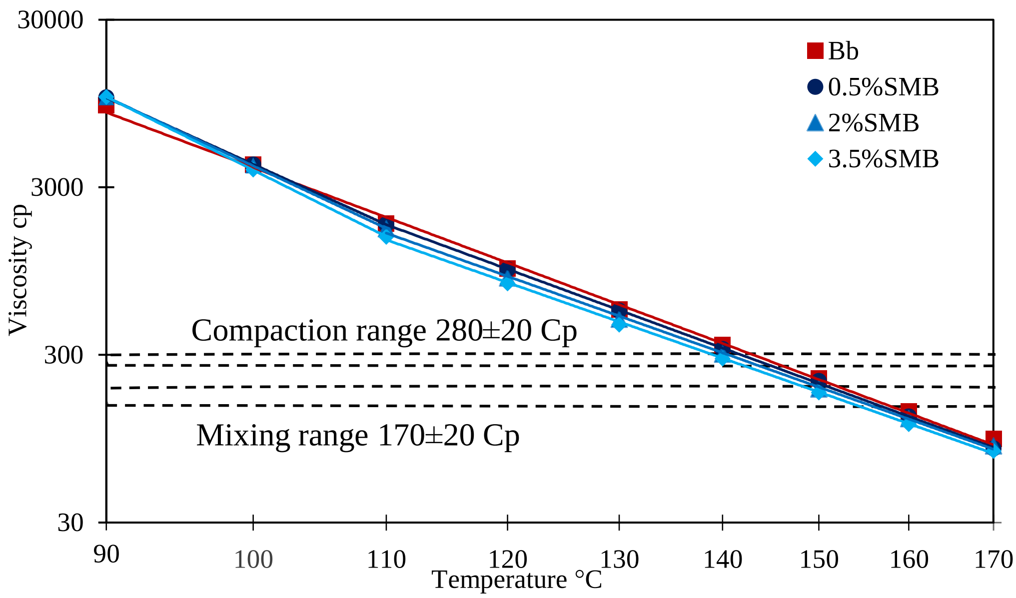


Figure 3-6. Viscosity results of SMBs

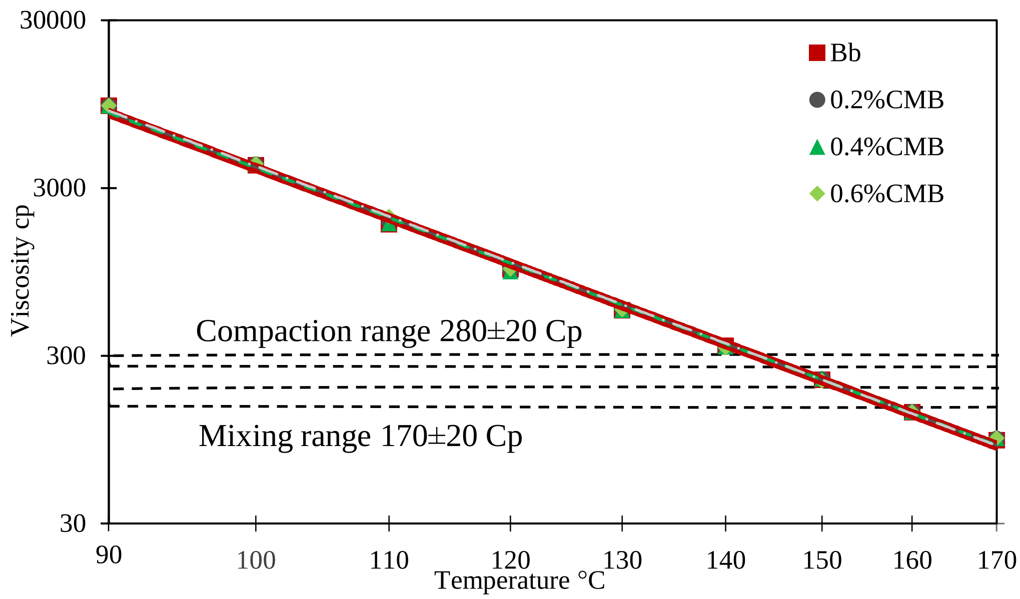


Figure 3-7. Viscosity results of CMBs

3.4.4 Rheological Characterisation of Bitumen

Bitumen is a thermoplastic, viscoelastic material that behaves like an elastic solid at low temperatures and like viscous fluid at high temperatures and exhibits time and temperature-dependent flow properties (Airey, 1997). To analyse this complex behaviour, it is necessary to study bitumen response under different conditions of loading and environmental conditions. In other words, this means analysing the rheology of bitumen, which is the study of stress-strain relationships under different temperatures and loading frequencies (Airey, 2011). The most applied concepts in studying bitumen rheological properties are the application of Dynamic Mechanical Analysis (DMA) associated with the Time-Temperature Superposition Principle.

DMA can be considered as a robust method of analysing the rheological properties of bituminous binders. Application of sinusoidal loading under controlled stress or controlled strain conditions over a range of frequencies and temperatures gives valuable information about the rheology and behaviour of the binders. Currently, the most widely accepted approach to DMA is to use a Dynamic Shear Rheometer

(DSR) to apply sinusoidal loading within the linear viscoelastic (LVE) zone (Airey et al., 2016) at different temperatures and frequencies. The DSR works by applying sinusoidal oscillatory stresses and strains to a disc-shaped bitumen sample sandwiched between two plates (Airey and Rahimzadeh, 2004). The loading can be conducted either under stress-controlled or strain-controlled conditions; the strain-controlled condition is shown in Figure 3-8. The strain at any time and resultant stress can be calculated as follows (Airey, 1997):

$$\gamma^{*} = \gamma_{o} \sin \omega t \qquad \text{Eq. 3-1}$$

$$\sigma^{*} = \sigma_{o} \sin (\omega t - \delta) \qquad \text{Eq. 3-2}$$

where γ^{*} is the strain at time t, γ_o is the peak strain, ω is the angular frequency (which equals $2\pi f$ where f is the frequency), σ^{*} is the stress at time t, σ_o is the peak stress, and δ is the phase lag between the stress and the strain which equals zero for purely elastic materials and ninety degrees for purely viscous materials. Viscoelastic materials such as the asphalt binders exhibit phase angles between those extremes (SHRP, 1994). As a result of the lag between stresses and strains, two components define the absolute value of the complex shear modulus, the storage modulus (G') which is the in-phase component and the loss modulus (G'') which is the out-of-phase component, as follows:

$$G'(\omega) = \frac{\sigma_o}{\gamma_o} \cos \delta \qquad \text{Eq. 3-3}$$

$$G''(\omega) = \frac{\sigma_o}{\gamma_o} \sin \delta \qquad \text{Eq. 3-4}$$

Figure 3-9 shows the relationship between the storage modulus, the loss modulus, the complex modulus and the phase angle. Therefore, the norm of the complex modulus can be calculated in an absolute form using the following equations:

$$G^*(\omega) = \sqrt{(G')^2 + (G'')^2} \quad \text{Eq. 3-5}$$

$$G^*(\omega) = \sqrt{({}^{\sigma_o}/_{\gamma_o}\ cos\,\delta)^2 + ({}^{\sigma_o}/_{\gamma_o}\ sin\,\delta)^2} \quad \text{Eq. 3-6}$$

$$G^*(\omega) = \frac{\sigma_o}{\gamma_o} \quad \text{Eq. 3-7}$$

In this study, a Gemini TM DSR was used to measure the rheological properties of the studied binders. These properties were measured at temperatures 0 to 80°C using a frequency range of 0.1 to 10 Hz under a controlled strain of 1%. This strain level was selected based on a previous study (Airey and Rahimzadeh, 2004) to assure that all measurements are conducted within the linear viscoelastic (LVE) zone where the shear modulus is independent of the stress magnitude which means the measurements are only affected by the temperature and loading frequency.

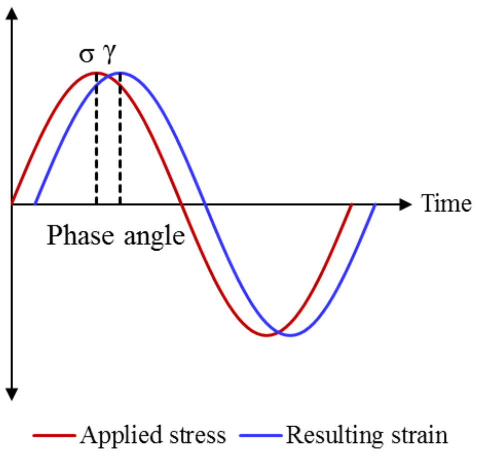


Figure 3-8. Sinusoidal stress-strain relationship

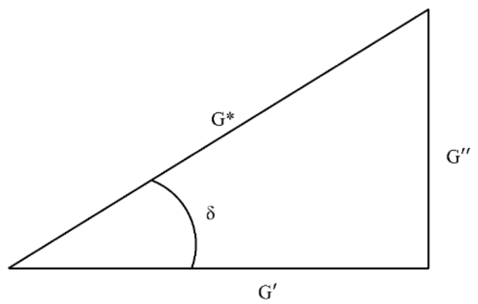


Figure 3-9. Phase angle, storage modulus, loss modulus, and shear modulus relationship

Figure 3-10 and Figure 3-11 present shear modulus and phase angle results of the base binder, respectively. It can be seen that the shear modulus has a direct relationship with the loading frequency and an inverse relationship with temperature, whereas the phase angle has an inverse relationship with the loading frequency and a direct relationship with temperature, which agrees with the typical behaviour of bituminous binders.

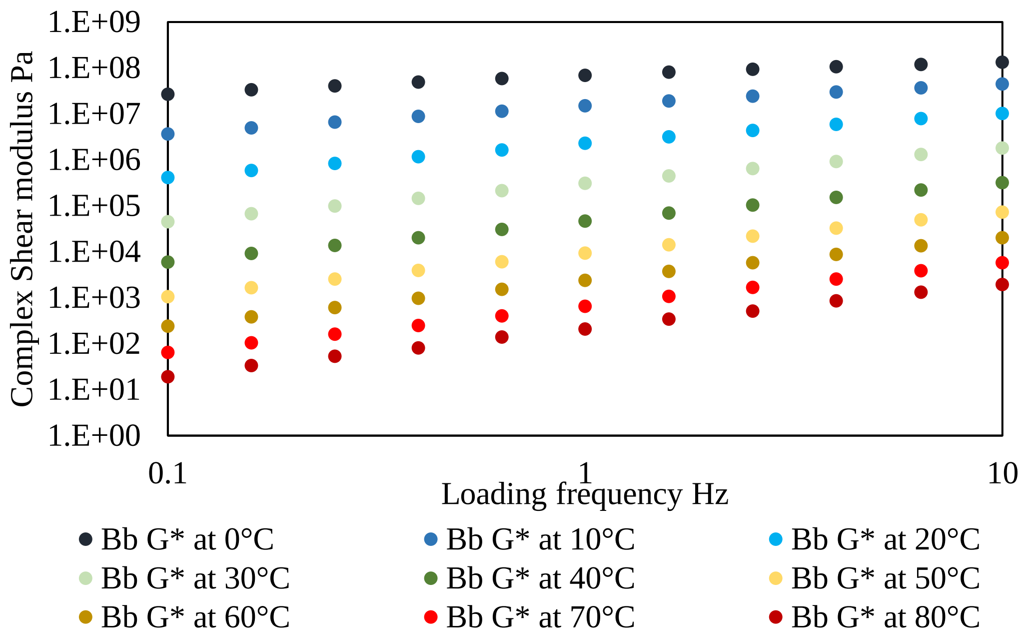


Figure 3-10. Shear modulus results of Bb

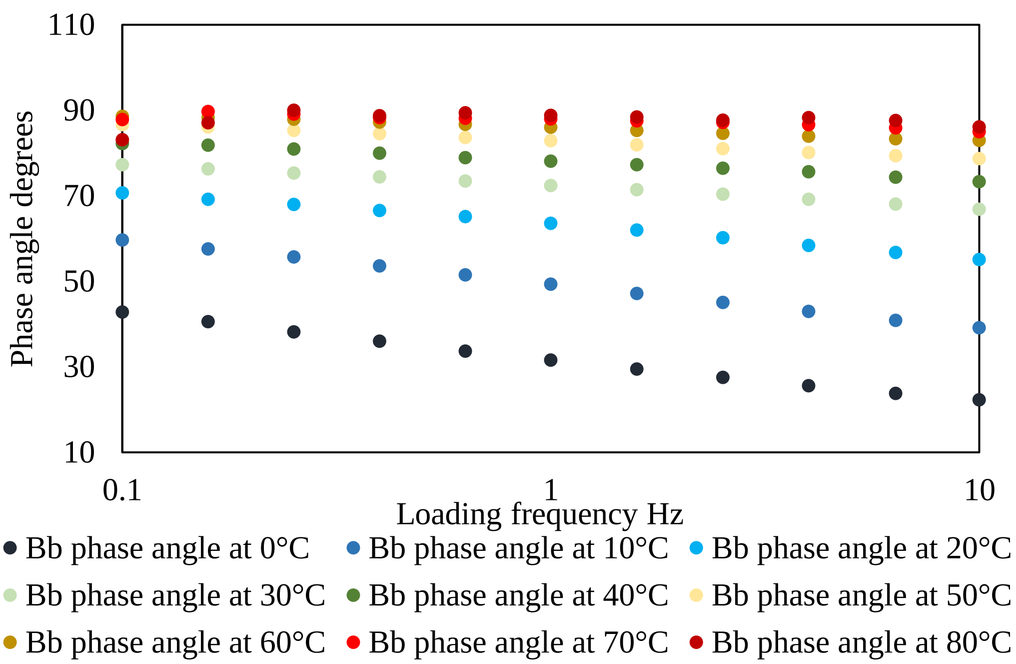


Figure 3-11. Phase angle results of Bb

3.4.5 Master Curves

Although determining the complex modulus and the phase angle across several temperatures and frequencies using the DSR is quite helpful, this data has a limited application without establishing master curves. These curves can be constructed using the Time-Temperature Superposition Principle (TTSP) to establish a continuous shear modulus curve at a reduced time or frequency (Airey, 1997). Based on this principle, the complex modulus can be represented as a function of reduced time or frequency. By this concept, bitumen behaviour can be represented over a wide range of frequencies at a reference temperature by shifting the complex modulus results of different temperatures with respect to frequency or time to a reference temperature, thus constructing one master curve to represent the complex modulus as a function of the frequency for a given temperature.

Master curves of all studied binders were constructed by utilising the TTSP. Figures 3-12 and 3-13 present the shear modulus and phase angle master curves of SMBs. These figures indicate that this additive increased the shear modulus, and the increase is directly related to the additive dosage; the higher the dosage, the higher the increase in the shear modulus. The figures demonstrate that Sasobit decreased phase angle (towards more elastic behaviour) and the decrease is inversely related to the additive dosage; the higher the dosage, the lower the phase angle. This effect of Sasobit on the thermo-rheological properties of the base binder can be explained by the way that this material behaves below its melting point. Figure 3-6 shows that this material has a melting point of 100°C; since the DSR testing temperature is between 0-80°C, then the state of Sasobit should be solid during DSR testing. Also, some researchers reported that Sasobit forms "microscopic stick-shaped particles" evenly scattered in bitumen (D'angelo et al., 2008). Other researchers ((Abraham et al., 2002, Butz et al., 2001) cited in (Hurley and Prowell, 2005, Hurley, 2006)) mentioned that below the melting point, Sasobit forms a crystalline structure that reduces motion of bitumen molecules and eventually increases bitumen viscosity. Both opinions confirm the conclusion that this material stiffens bitumen. On the other hand, the master curves of CMBs are presented in Figures 3-14 and 3-15. These figures demonstrate that Cecabase, within the limits of the added dosages, does not have any effect on the rheological properties of the base bitumen. However, this conclusion is valid only when bitumen is unaged.

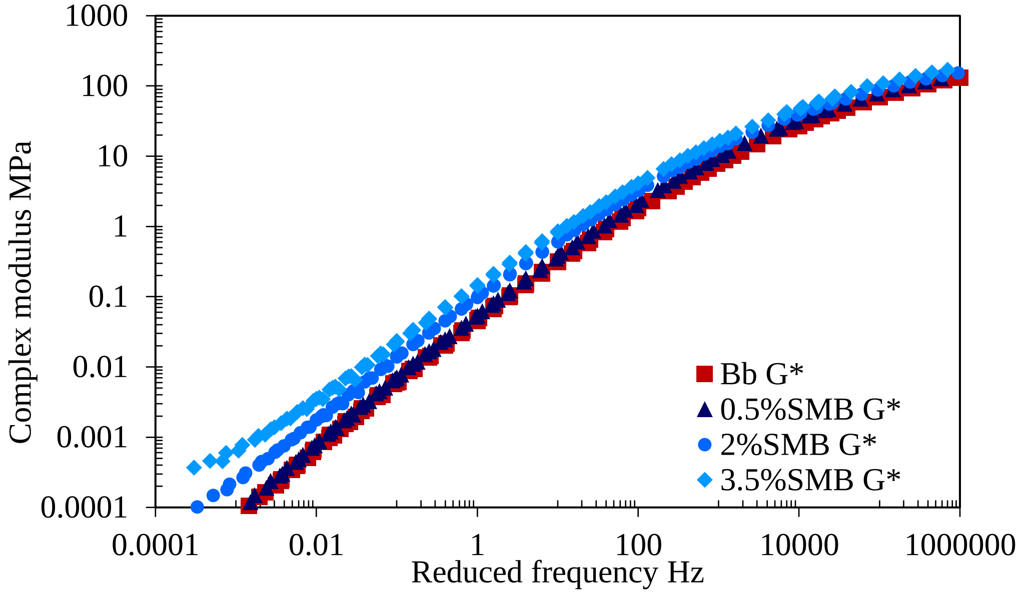


Figure 3-12. Complex modulus master curve for Bb and SMBs

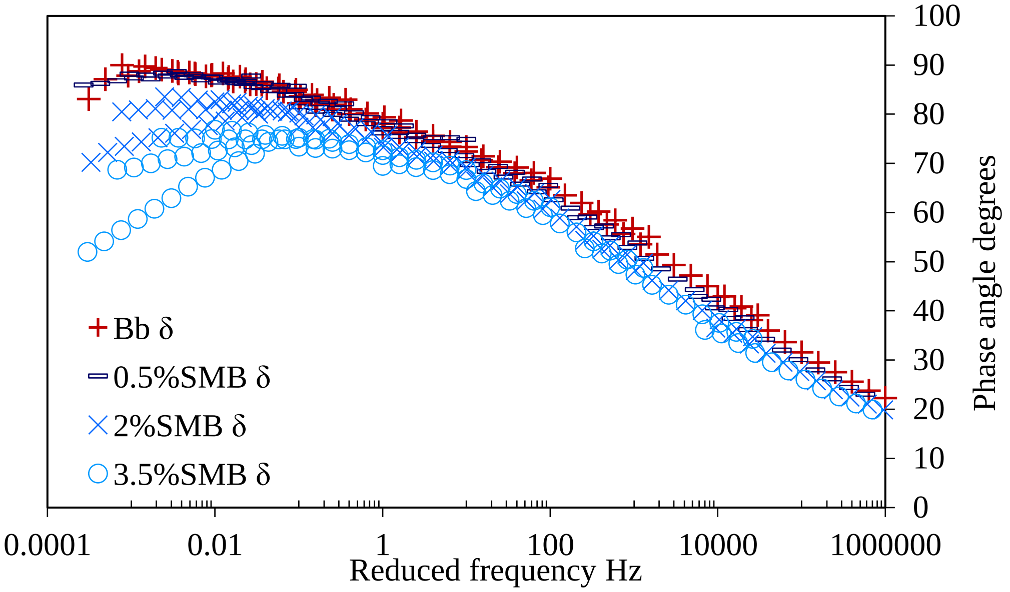


Figure 3-13. Phase angle master curve for Bb and SMBs

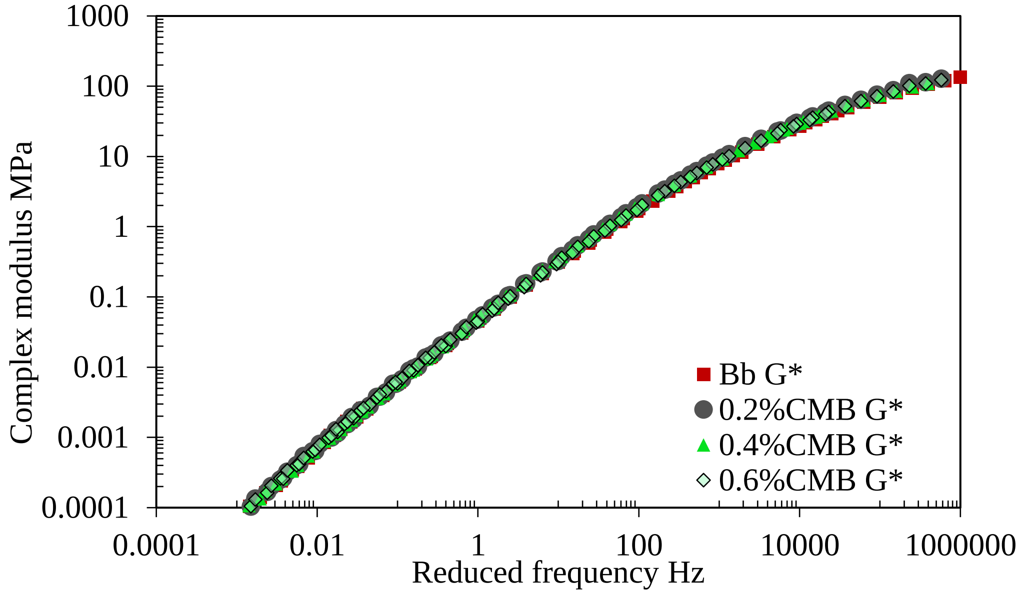


Figure 3-14. Complex modulus master curve for Bb and CMBs

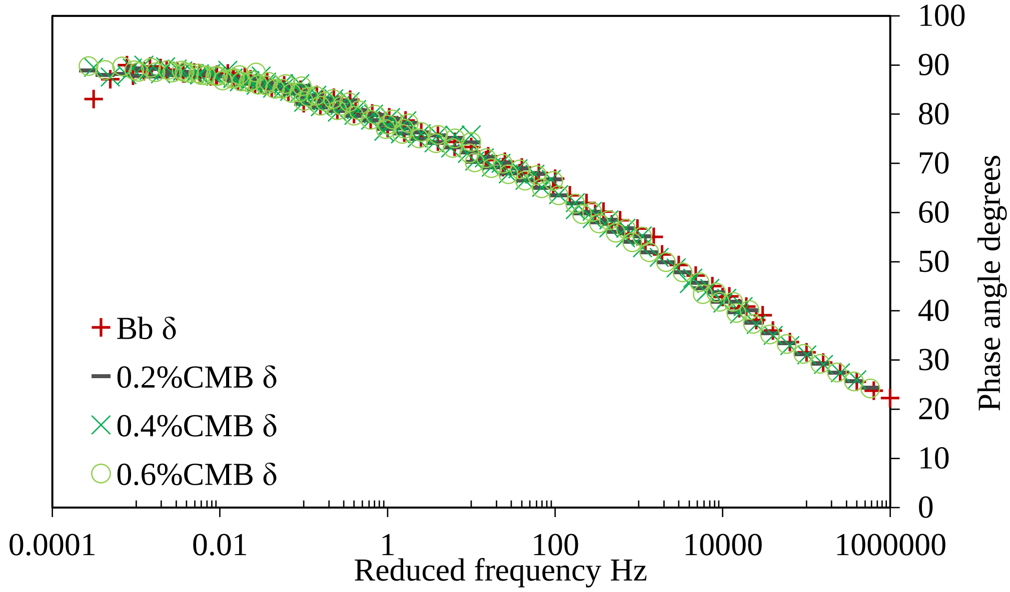


Figure 3-15. Phase angle master curve for Bb and CMBs

3.4.6 Black Diagrams

Another way to analyse the rheological properties of bitumen is by black diagrams. The black diagram is a chart of the shear modulus versus the phase angle, and it can be constructed without any data manipulation or shifting; it is quite informative in giving an understanding of the rheology of the binder and the equivalency of the TTSP principle (Airey, 2011). In other words, this diagram helps in understanding the changes in the shear modulus and phase angle over a range of temperature and frequencies. Accordingly, any effect of the used additives on the thermo-rheological properties, whether on the shear modulus, the phase angle, or both, can be detected by this chart.

Figures 3-16 and 3-17 present black diagrams for SMBs and CMBs, respectively. The first figure indicates that the Sasobit effect is pronounced at high temperatures and low frequencies, as this additive increased shear modulus and reduced phase angle indicating an improved elastic response. This means that this additive can improve permanent deformation resistance at high temperatures since these are the usual conditions where asphalt pavements become susceptible to rutting distress. On the other hand, Figure 3-17 shows that Cecabase changed neither the complex modulus nor the phase angle, which is in agreement with the viscosity and empirical test results.

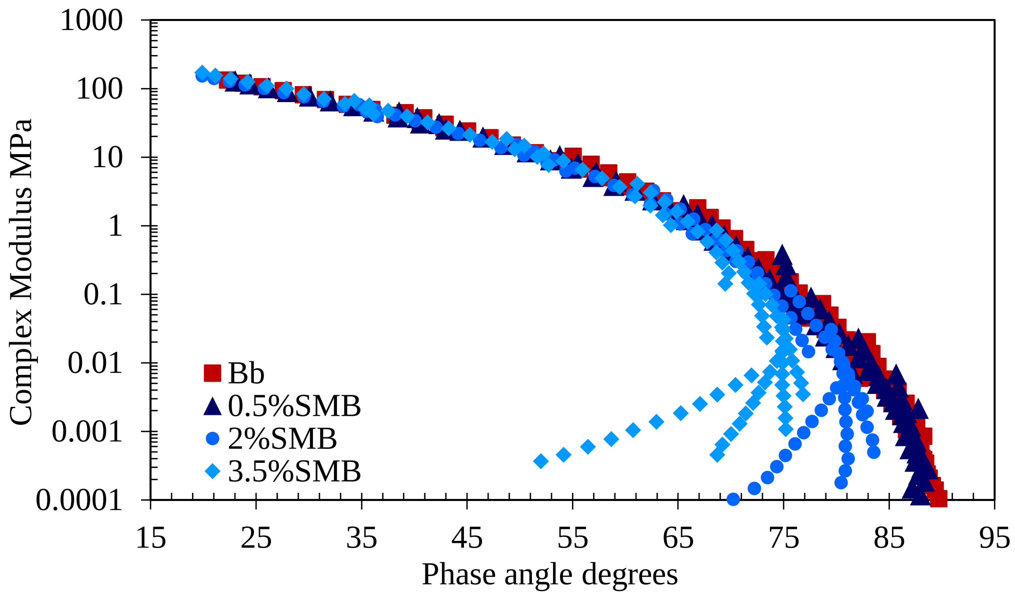


Figure 3-16. Black diagram of Bb and SMBs

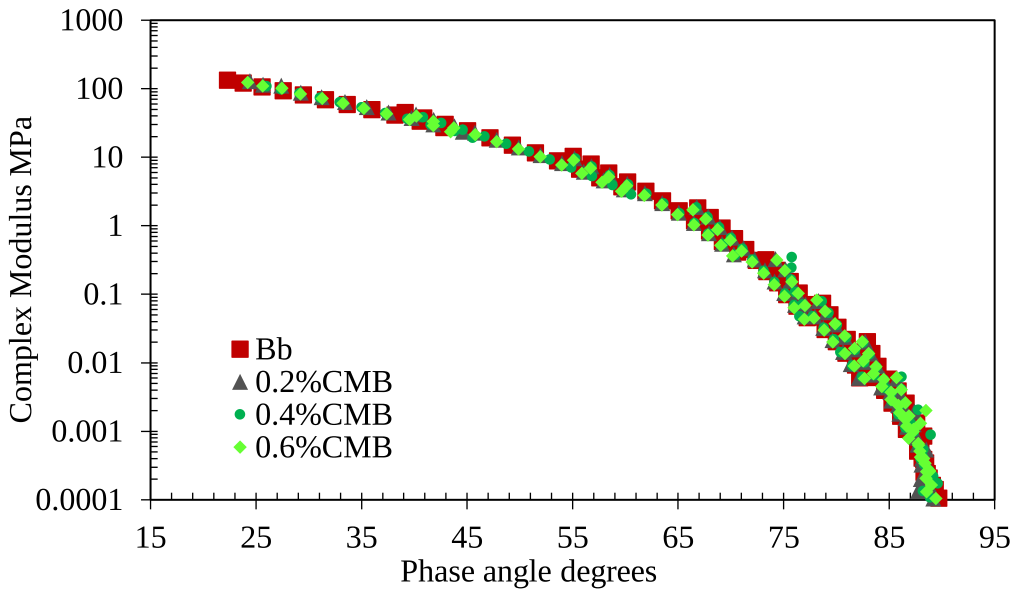


Figure 3-17. Black diagram of Bb and CMBs

3.5 Additive Impact on Bitumen Ageing

Ageing of bitumen is a complex physiochemical process that leads to increasing its stiffness and reducing its recovery properties over time, mainly due to evaporation of volatile materials and oxidation (Mastrofini and Scarsella, 1999). This process is considered as one of the main factors that contribute to the degradation of the durability of asphaltic mixtures (Airey, 2007). The durability is affected by the appearance of certain distress types such as fatigue and low-temperature cracking which appear due to the increasing bitumen stiffness and loss of its recovery properties (Barco Carrión et al., 2015, Baghaee Moghaddam and Baaj, 2016). Accordingly, studying the effect of the additives on bitumen ageing is a critical point that can help in analysing and understanding the behaviour of WMA.

3.5.1 Artificial Ageing of Bitumen

Ageing of bitumen takes place during two stages. The first stage is short-term ageing that occurs during the mixing, laying, and compaction of asphalt. The primary cause of ageing during this stage is the evaporation of volatile materials from the bitumen (Airey, 1997). The second stage is long-term ageing that takes place due to oxidation and steric hardening of the bitumen phase within asphalt over the service life of a pavement (Masson et al., 2005). There is a significant number of testing methods that simulate these two stages. Among them is the Rolling Thin Film Oven Test (RTFOT) to simulate the short-term ageing, and the Pressure Ageing Vessel (PAV) to simulate the long-term ageing (Airey, 2007). These two methods have been frequently used to study the bitumen ageing process. Accordingly, they were adopted in this study. The RTFOT test was conducted according to British Standard BS EN 12607-1:2007 (BSI, 2007), whereas the PAV test was performed following BS EN 14769:2012 (BSI, 2012b). Furthermore, the ageing effect was studied by monitoring and evaluation of thermo-rheological properties of the studied binders before and after the two ageing stages. Figure 3-18 presents G* and δ master curves of Bb after RTFOT ageing and combined RTFOT+PAV ageing. The figure indicates that G* increases and δ decreases over ageing. These results can be explained by the volatilisation and oxidation of volatile materials of the bitumen during the ageing process which leads to increasing bitumen stiffness and reducing phase angle.

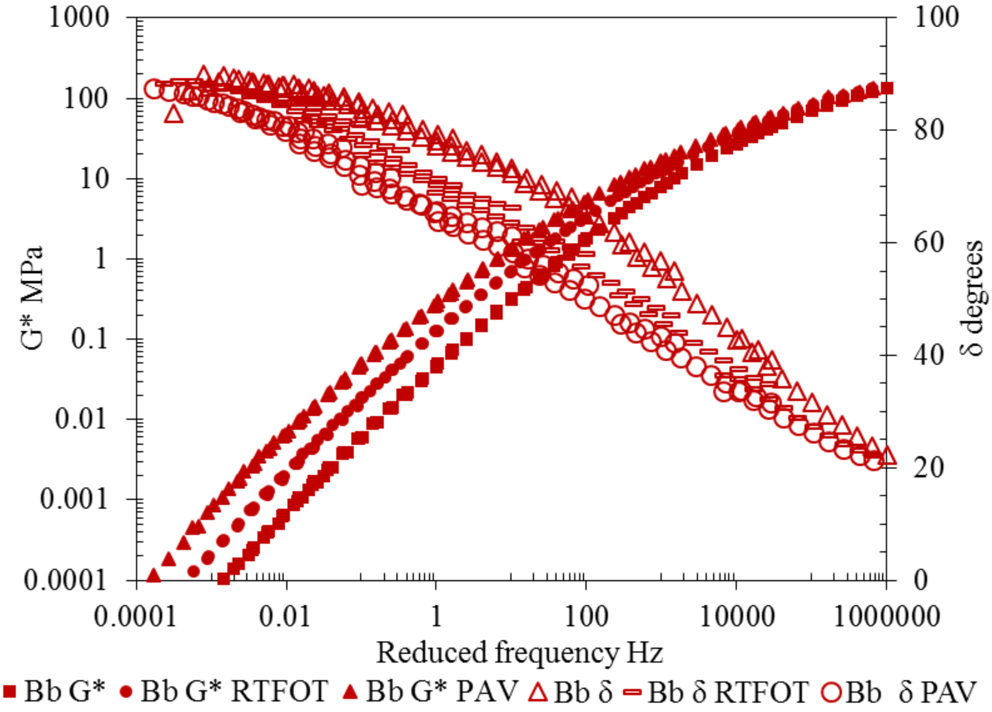


Figure 3-18. Ageing effect on Bb rheology

3.5.2 WMA Additive Impact on Bitumen Ageing

Bitumen ageing can be recognised by the evolution of G* and δ over ageing. Accordingly, if WMA additives have an impact on this property, then that impact can be captured by monitoring and evaluating the changes in the evolution of the rheological properties over ageing. This technique has been used in several studies to study the bitumen ageing process (Mastrofini and Scarsella, 1999, Zhang et al., 2010). Therefore, it was implemented in this study.

Figure 3-19 and Figure 3-20 present the G* and δ results of SMBs in comparison with Bb. Interestingly, despite the stiffening effect of Sasobit, which appeared as an increase in G* and a decrease in δ, after the combined ageing processes, SMBs have almost similar rheological properties to the Bb. This result is demonstrated in Figure 3-20, as all binders have comparable G* and δ results. This means that Sasobit stiffens bitumen immediately after modification, but it retards the evolution of bitumen properties over time. Therefore, it can be concluded that this additive has the potential to improve rutting resistance due to the increase in the shear modulus after no or short-term ageing. It also has the potential to improve cracking resistance due to the slower increase in G* after long-term ageing. On the other hand, CMB results are presented in Figure 3-21 and Figure 3-22. These figures indicate that the CMBs have a slightly lower G* and higher δ than Bb; this result can be spotted clearly at low frequencies (high temperatures) in Figure 3-22. This means that this additive has the ability to retard bitumen ageing by retarding the evolution of G* over ageing. However, it can make bitumen more viscous as δ is increasing over ageing. This can be interpreted as an improvement in cracking resistance after long-term ageing (Walubita et al., 2006), and an increased risk of exhibiting rutting due to the retardation in the shear modulus after ageing.

Furthermore, since the effect of ageing is pronounced on fatigue resistance, the critical intermediate temperature derived from the Superpave fatigue cracking parameter ($G^*\times\sin\delta\leq5000$ kPa (AASHTO, 2015)) was used to analyse the ageing effect of the additives used. The results are presented in Figure 3-23. This figure indicates that the critical intermediate temperature is generally increasing over ageing, which means that the susceptibility of bitumen to exhibit fatigue cracking is also increasing over ageing. The base bitumen critical temperatures are 14.5, 16.8, and 26.6°C after no ageing, short-term ageing, and combined short and long-term ageing, respectively. However, the presence of additives changed the rate of increase of this temperature over the ageing stages. SMBs exhibited slightly higher critical temperatures than that of the base binder, whereas the CMBs showed slightly lower critical temperatures. However, after the combined ageing stages, all modified binders exhibited critical temperatures between 18-21°C, significantly lower than that of the base binder. This means that the additives retarded bitumen ageing based on the Superpave fatigue cracking parameter, which suggests that these additives are likely to improve or prolong asphalt fatigue cracking life by reducing its susceptibility to that kind of failure after asphalt ageing takes place.

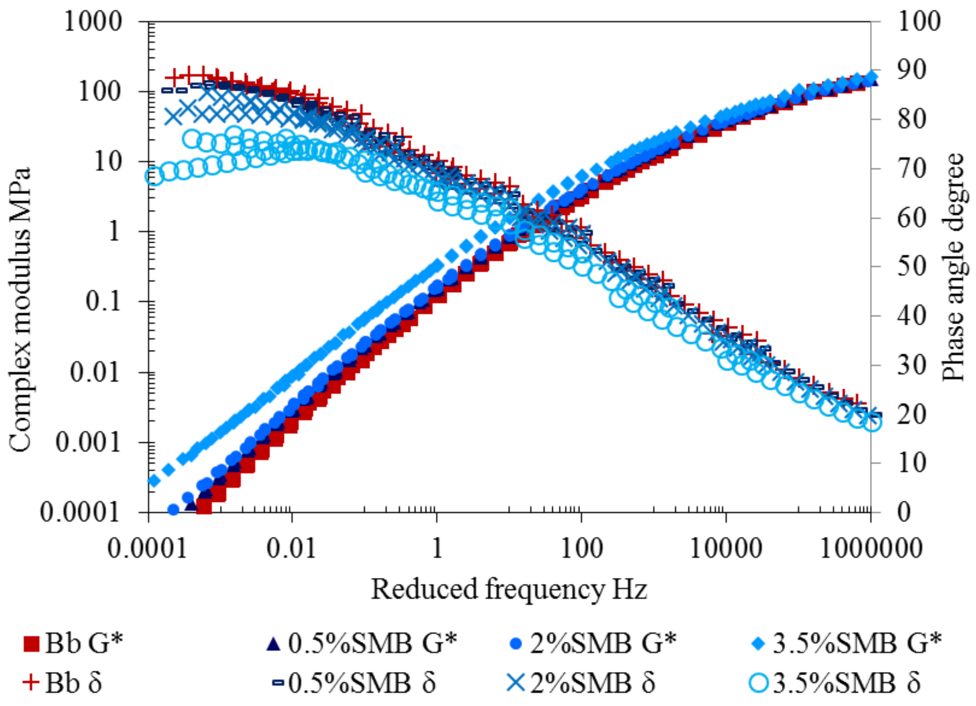


Figure 3-19. G* and δ results of SMBs after RTFOT ageing

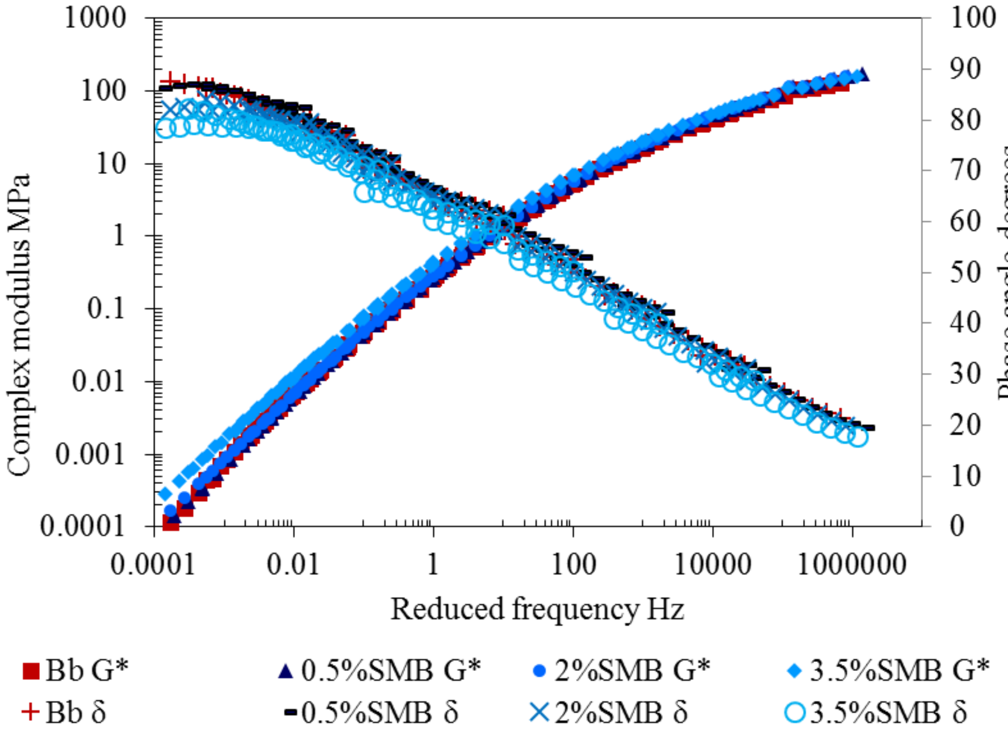


Figure 3-20. G* and δ results of SMBs after RTFOT+PAV ageing

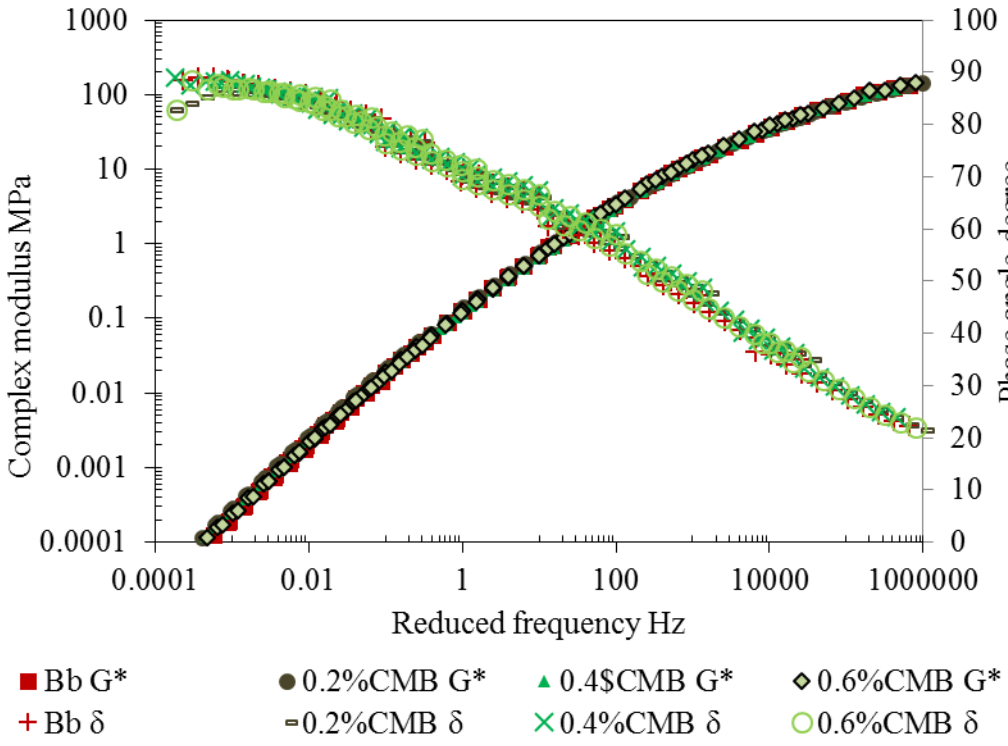


Figure 3-21. G* and δ results of CMBs after RTFOT ageing

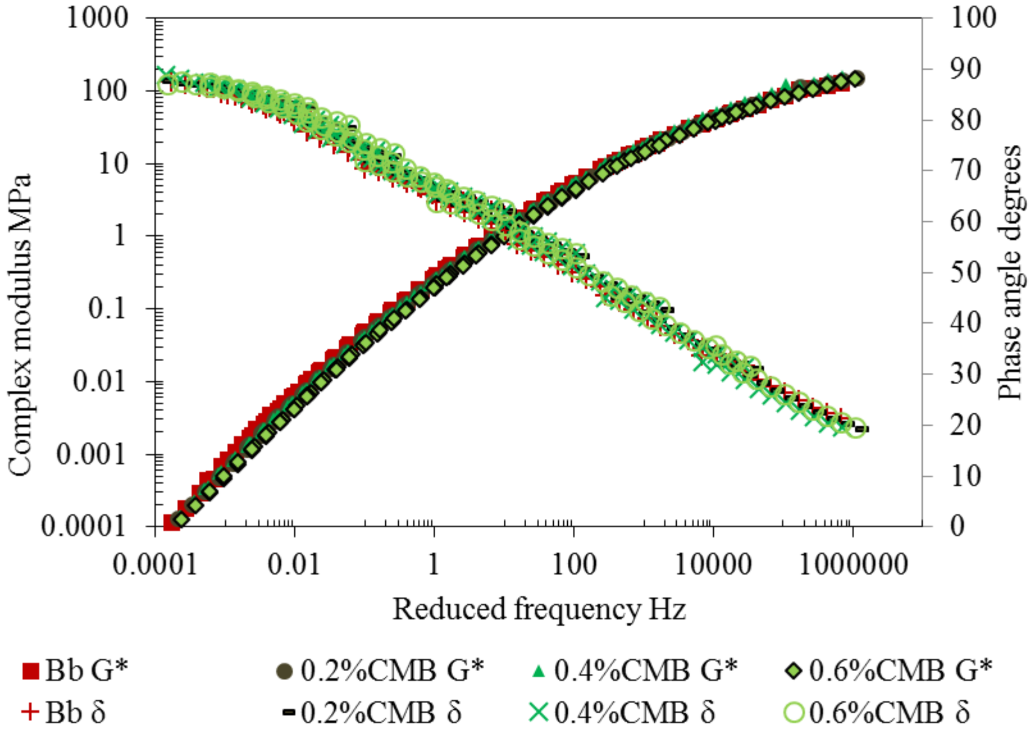


Figure 3-22. G* and δ results of CMBs after RTFOT+PAV ageing

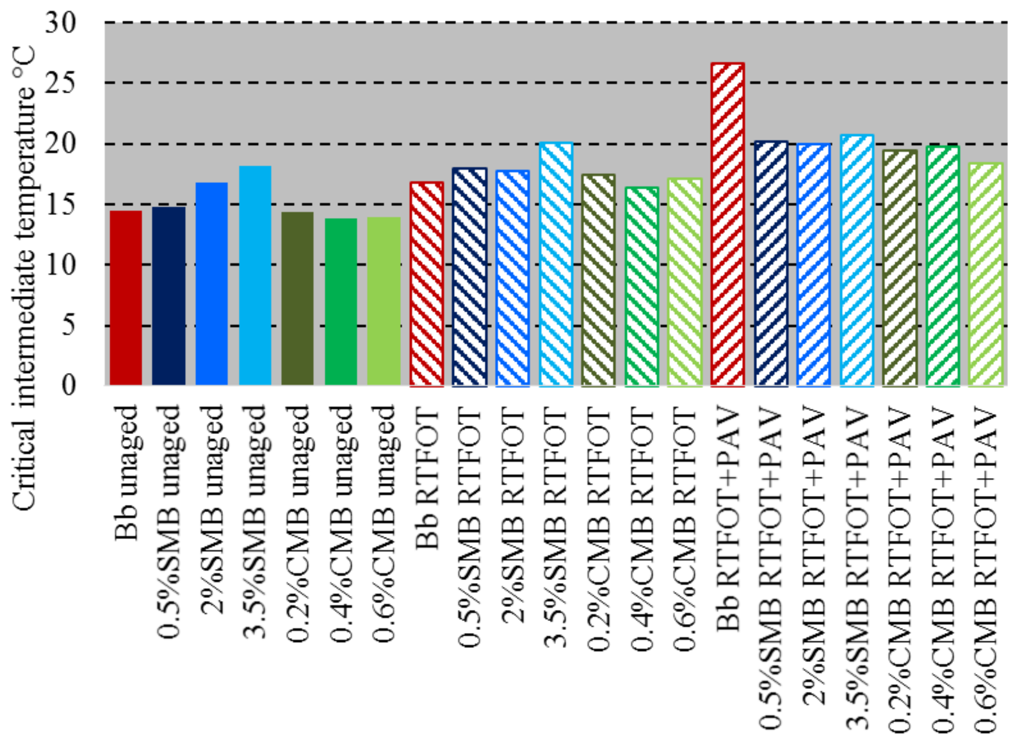


Figure 3-23. Critical intermediate temperatures of the analysed binders

3.6 Performance of WMA Modified Bitumen

The previous sections were focused on bitumen characterisation in the linear viscoelastic zone, where the bitumen rheological properties are independent of stress magnitude (Airey, 1997). This approach may be valid for unmodified binders since their behaviour can be described as thermorheologically simple. However, due to the modification process with WMA additives, modified binder behaviour becomes more complicated and more challenging to characterise, and traditional empirical methods such as penetration or viscosity, or the Superpave grading system, may not be valid in characterising the behaviour of the modified binders (Bahia et al., 2001a, Bahia et al., 2010, Subhy, 2017). Bahia et al. (2001b) reported that the Superpave rutting ($G^*/\sin\delta$) and fatigue cracking parameters had a weak correlation with the performance of asphalt mixtures manufactured with modified binders. Airey (2004) found that there was no clear relationship between Superpave parameters of modified binders and the actual asphalt performance. This suggests that these parameters may not be good indicators of the performance of modified bitumen. Furthermore, some studies (Bahia et al., 2010, Marasteanu, 2010) have demonstrated that characterising and predicting bitumen performance based on the linear viscoelastic properties may not be valid. These studies indicated that a better prediction of modified bitumen performance could be achieved when bitumen is characterised beyond the limits of linear viscoelasticity. Additionally, Sadeq et al. (2016) reported that strain concentration in the bitumen phase of asphalt could exceed linear viscoelastic limits, which causes nonlinear viscoelastic or viscoplastic deformations. This emphasises the need to characterise modified bitumen beyond the limits of linear viscoelasticity to provide better understanding of the behaviour and properties of WMA additive modified binders.

3.6.1 Permanent Deformation

At the binder level, permanent deformation is assessed nowadays by means of the Multiple Stress Creep Recovery (MSCR) test. This test can be considered as a replacement for the original SHRP parameter G*/Sinδ in the case of characterisation of modified binders beyond the limits of linear viscoelasticity to better analyse rutting performance of modified binders. The test was developed based on creep studies during the NCHRP 9-10 projects (Bahia et al., 2001a). The test can be conducted using a DSR for unaged or short-term aged binders at the high-performance grade temperature (Soenen et al., 2013). It can also be conducted to evaluate the elastic response of asphalt binders under shear creep and recovery; the elastic response can be assessed by the percentage of recovered shear strain and the non-recovered creep compliance (British Standards Institute, 2015). The test is typically conducted at two stress levels, 0.1 and 3.2 kPa; at every stress level, ten cycles of stress are applied for one second, followed by recovery for nine seconds. The percentage of binder recovery at the two stress levels can be calculated as follows:

$$\%R^N_{0.1kPa} = 100 \times (\varepsilon^N_1 - \varepsilon^N_{10})/\varepsilon^N_1 \qquad \text{Eq. 3-8}$$

$$\%R^N_{3.2kPa} = 100 \times (\varepsilon^N_1 - \varepsilon^N_{10})/\varepsilon^N_1 \qquad \text{Eq. 3-9}$$

where $\%R^N_{0.1KPa}$, $\%R^N_{3.2KPa}$ are the percentage recoveries at 0.1kPa and 3.2kPa respectively, N is the cycle number, ε^N_1 is the shear strain at the end of the creep stage and ε^N_{10} is the shear strain at the end of the recovery stage. The average percentage recovery for both stress levels can be calculated by averaging the percentage recovery over the ten cycles of the test. The recovery is an important indicator of the binder response to applied stress and can be used to analyse the effect of modification on bitumen performance (Hill, 2014). On the other hand, the non-recoverable creep compliance J_{nr} has been well correlated with the rut depth (Anderson, 2014) and 4.5, 2.0, 1.0 and 0.5 kPa^{-1} J_{nr} values have been determined as requirements for standard, heavy, very heavy and extreme heavy traffic volumes (AASHTO, 2013a). The non-recoverable creep compliance can be determined by calculating the non-recovered strain at the end of the relaxation stage divided by the applied stress as follows:

$$J_{nr}^{N} = \varepsilon_{10}^{N}/\sigma \qquad \text{Eq. 3-10}$$

where J_{nr}^{N} is the non-recoverable creep compliance and σ is the applied stress. The average J_{nr} can be calculated by averaging the J_{nr} over the ten cycles and for both stress levels.

In this study, this test was conducted using a Gemini™ DSR at temperatures of 30, 40, 50, and 60°C using a 25 mm spindle and 1 mm gap to detect the permanent deformation performance of the binders and to analyse the sensitivity of the rutting resistance to temperature change. It was performed without any binder ageing to simulate the worst condition scenario, also to reflect the ageing retardation effect of the studied additives. Figure 3-24 presents creep results at 60°C, which demonstrate that the wax additive reduced shear strain, whereas the chemical additive slightly increased it. This figure shows that there are two different mechanisms through which the additives can alter bitumen response to the applied stress: firstly, by changing bitumen stiffness which in turns leads to decreasing or increasing the resultant strain; secondly, by impacting bitumen recovery properties which means changing the ability of the bitumen to recover its initial shape after the load is removed. It can be seen these are different mechanisms, so it was decided to uncouple these effects to better understand the effect of the additives.

The creep element was studied by isolating creep results at the end of the loading period (1 sec.), then normalising the results for the modified binders at each temperature against the base binder creep results, as shown in Figure 3-25. This figure clearly demonstrates that the wax additive significantly reduced the creep, and this reduction is directly related to the additive dosage: the higher the additive concentration, the greater the reduction. In contrast, the chemical additive slightly increased the creep, by approximately 2-10% at all testing temperatures, and it seems that the increase is directly related to the additive concentration. The recovery element was analysed similarly by calculating bitumen recovery, then normalising the results against the base binder results at each testing temperature, as shown in Figure 3-26. This figure indicates that the organic additive significantly improved creep recovery properties at intermediate and high dosages. This improvement is also related to the additive concentration, the higher the dosage, the greater the improvement. Meanwhile, the chemical additive very slightly reduced creep recovery.

Furthermore, the non-recovered creep compliance, which is the outcome of the combined effect of the two mechanisms (bitumen stiffness and creep recovery), was calculated and the results are presented in Figure 3-27, again normalised against the base binder. This figure confirms the conclusion that the organic additive is likely to significantly improve permanent deformation resistance, whereas the chemical additive slightly reduces this property. However, by uncoupling the creep stage from the recovery stage, it becomes clear how these additives alter bitumen rheological properties. The organic additive significantly increases bitumen stiffness and increases its creep recovery properties, which leads to a better rutting resistance. On the other hand, the chemical additive increases bitumen susceptibility to permanent deformation, mainly due to reducing its stiffness, which results in an increased Jnr magnitude.

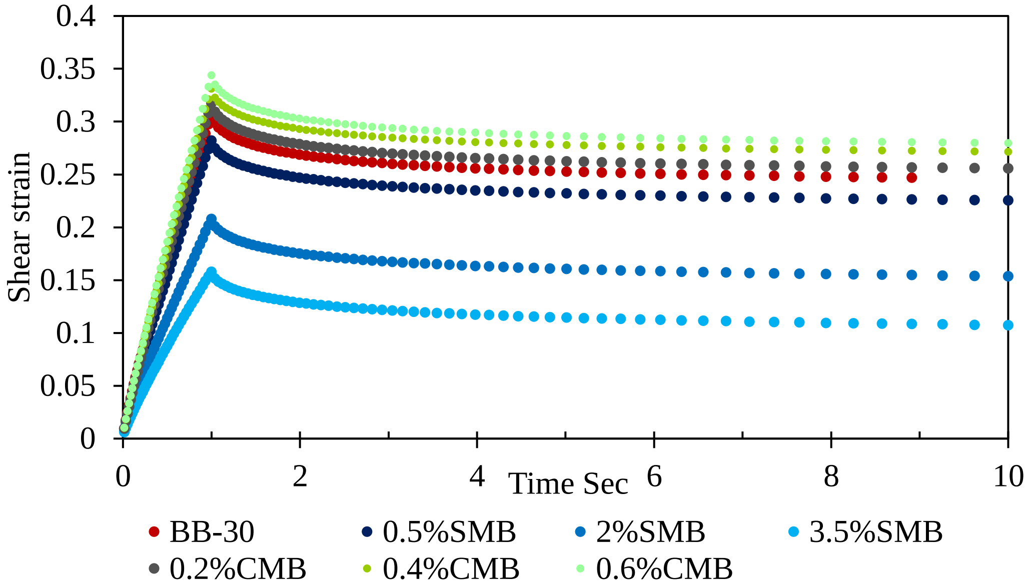


Figure 3-24. Creep and recovery of the studied binders at 60°C and 3.2 kPa loading stress

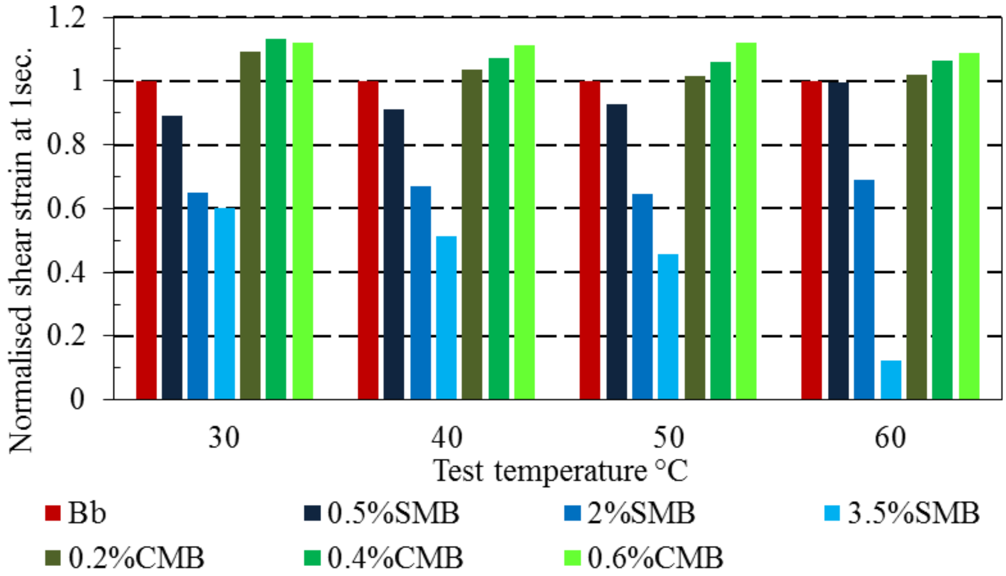


Figure 3-25. Normalised creep results at 1 sec. of loading

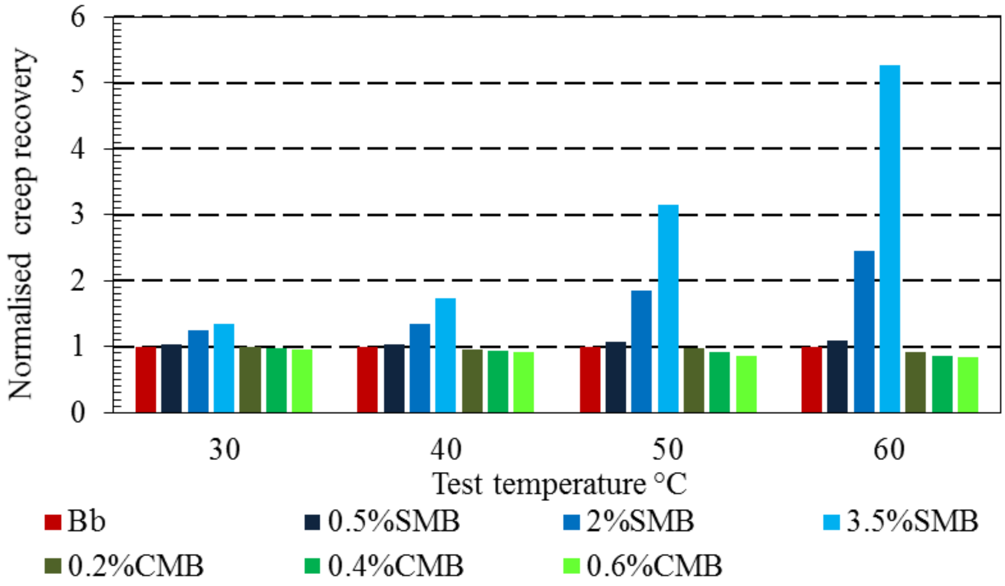


Figure 3-26. Normalised creep recovery results

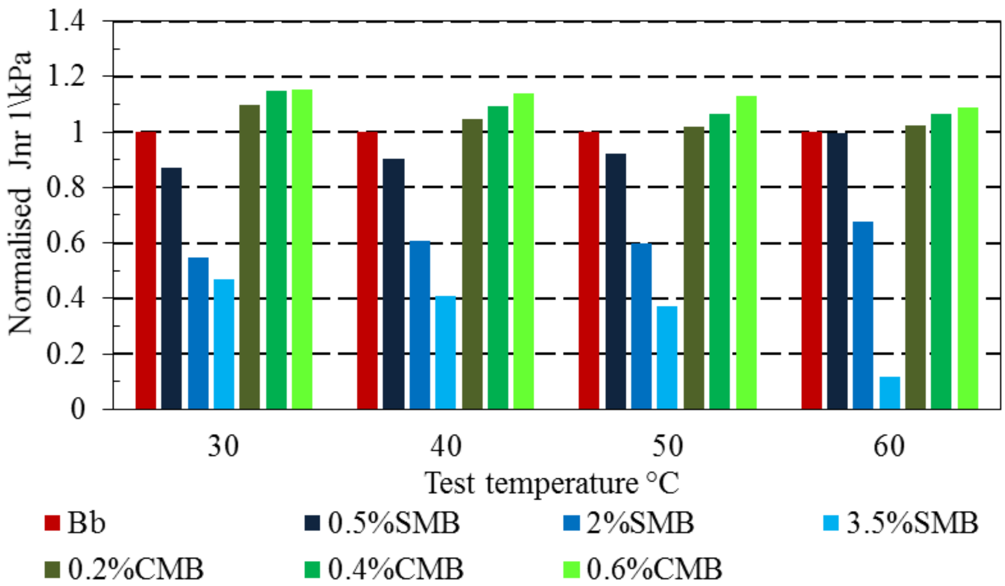


Figure 3-27. Jnr normalised results

3.6.2 Fatigue Cracking of Bituminous binders

Fatigue cracking is a major failure mode of flexible pavements. Historically, fatigue cracking has been studied at the asphalt level. However, it is known that fatigue damage initiates as micro-cracks within bitumen or mastic components of asphalt (Airey et al., 2004). Accordingly, researchers have developed different methods to characterise the fatigue performance of bitumen; one of them is the Linear Amplitude Sweep (LAS) test. This test was developed based on Viscoelastic Continuum Damage (VECD) principles by Kim et al. (2006). Later it was adopted as a standard AASHTO test to evaluate and predict fatigue life due to the repeatability of testing results, simplicity and speed of the test. This test is performed in two stages using a DSR and an 8 mm spindle. The first stage is to determine the viscoelastic properties (G* and δ) of the intact material in the LVE zone; this can be done by performing a frequency sweep test at the test temperature. The second stage is to subject the same sample to a linear strain sweep of increasing amplitude from 0.1% to 30% (Hintz et al., 2011). The fatigue life is predicted in the LAS test as follows (AASHTO Tp 101, 2014): from the first stage of the test, the material constant (α) which determines the energy release rate of the material, is calculated using the following equations:

$$log\, G'(\omega) = m \times log(\omega) + b \qquad \text{Eq. 3-11}$$

$$\alpha = {}^{1}/_{m} \qquad \text{Eq. 3-12}$$

where G' is storage modulus as a function of frequency (ω), m and b are fitting parameters. From the second stage, the fatigue damage is calculated as a function of time as follows:

$$D(t) = \sum_{i=1} [\pi \gamma_i^2 (C_{i-1} - C_i)]^{\frac{\alpha}{1+\alpha}} \times (t_i - t_{i-1})^{\frac{1}{1+\alpha}} \qquad \text{Eq. 3-13}$$

where D(t) is the fatigue damage as a function of time, γ_0 applied strain for a given data point, and C is the material integrity parameter which can be determined as follows:

$$C(t) = {}^{|G^*|(t)}/_{|G^*|_{initial}} \qquad \text{Eq. 3-14}$$

Equation 3-14 states that the integrity parameter is the ratio of the shear modulus at any time to the undamaged shear modulus, so the initial value of this parameter equals 1. However, as the material starts to damage during the test, the value of this parameter starts dropping, and it reaches 0 for a completely damaged material. To describe the relationship between D and C, the following power model has been used (Hintz et al., 2011):

$$C_{(t)} = C_o - C_1 \times (D)^{C2} \quad \text{Eq. 3-15}$$

where C_1 and C_2 are the model fitting parameters. To detect the failure zone, researchers have proposed different methods. Johnson (2010) observed that the damage growth correlated well with the reduction of the parameter $|G^*|\sin\delta$. Therefore, he suggested a 35% reduction in this parameter to identify the failure zone. However, Bahia et al. (2013) reported that the peak shear stress better captures the failure zone. Micaelo et al. (2015) mentioned that the peak shear stress criterion resulted in a fatigue life that was comparable with other testing methods. Accordingly, the damage at the failure zone can be determined using the peak shear stress as follows:

$$D_f = \left(\frac{C_o - C_{at\ peak\ stress}}{C_1}\right)^{1/C_2} \quad \text{Eq. 3-16}$$

where D_f is the damage at failure calculated based on the value of the integrity parameter at the peak stress. Finally, fatigue life can be calculated as a function of the maximum expected strain as follows:

$$N_f = A \times (\gamma_{max})^B \quad \text{Eq. 3-17}$$

$$A = \frac{f \times (D_f)^{\kappa}}{k \times (\pi \times C_1 \times C_2)^{\alpha}} \quad \text{Eq. 3-18}$$

$$k = 1 + (1 - C_2) \times \alpha \quad \text{Eq. 3-19}$$

$$B = 2 \times \alpha \quad \text{Eq. 3-20}$$

where N_f is the predicted number of load applications until failure, γ_{max} is the maximum expected strain, and A, k, and B are model parameters.

Based on the prescribed approach, the LAS test was conducted in this study using a Gemini^{TM} DSR at an intermediate temperature of 19°C using an 8mm spindle; this temperature was selected because all modified binders exhibited an approximate critical intermediate temperature of 19°C (AASHTO Tp 101, 2014). Bitumen testing specimens were prepared by heating the bitumen to 160°C; then a small quantity was poured into an 8mm silicone mould and left for about 15 minutes to cool down. The DSR was zeroed, and testing spindles were conditioned at 63°C before attaching the specimen to provide sufficient adhesion during testing. The gap was then set to 2.050 mm; then, the extra bitumen was trimmed. Finally, the gap was set to 2.000mm, and the specimen was conditioned at 19°C for 15 minutes before testing. It is critical to heat testing spindles to high temperatures in order to assure a proper adhesion between testing specimen and spindles.

LAS results can be presented in different ways, including the damage characteristic curve; this curve defines the relationship between material integrity and damage growth in the material (Kutay and Lanotte, 2017). Material integrity represents the ratio of the normalised modulus of the material at time t to the undamaged modulus. Figure 3-28 presents the characteristic curves based on the VECD analysis derived previously. At the same material integrity level, the figure indicates that the Sasobit improved fatigue resistance as it reduced damage intensity. On the other hand, the results indicate that adding Cecabase reduced fatigue resistance as the damage intensity was increased. One possible reason for these results could be the failure mechanism of the LAS test. The fatigue failure zone is detected by the peak shear stress; as shown in Figure 3-29. The figure indicates that the shear stress is increasing to achieve the required strain until the sample fails, and the failure zone can be determined by the sharp drop in the shear stress. In other words, stiffer materials can perform better because they can sustain higher stresses before failure.

To add meaning to these observations, the number of load applications to failure at different strain levels was predicted using Eq. 3-17, as shown in Figure 3-30. Generally, the results confirmed the previous conclusion that Sasobit improved fatigue life, whereas Cecabase shortened it. The figure shows that SMBs are more strain-sensitive than other binders. This indicates that at low strain levels, SMBs can perform better than CMBs. Conversely, at high strain levels, CMBs can perform better due to the general lower stiffness of these binders, which makes them less strain sensitive. Moreover, by comparing the results in Figure 3-23 and Figure 3-30, it can be concluded that the Superpave fatigue cracking parameter showed that all modified binders are likely to improve fatigue performance after ageing, whereas LAS results indicated that Sasobit improved fatigue life and Cecabase reduced it. This result can probably be explained by the effect of the additive on the rheological properties of the base binder. The improved elastic and creep recovery properties of SMBs increased the fatigue life of these binders, whereas the poorer properties of CMBs decreased fatigue life. This result also demonstrates the importance of characterising material behaviour beyond the limits of viscoelasticity.

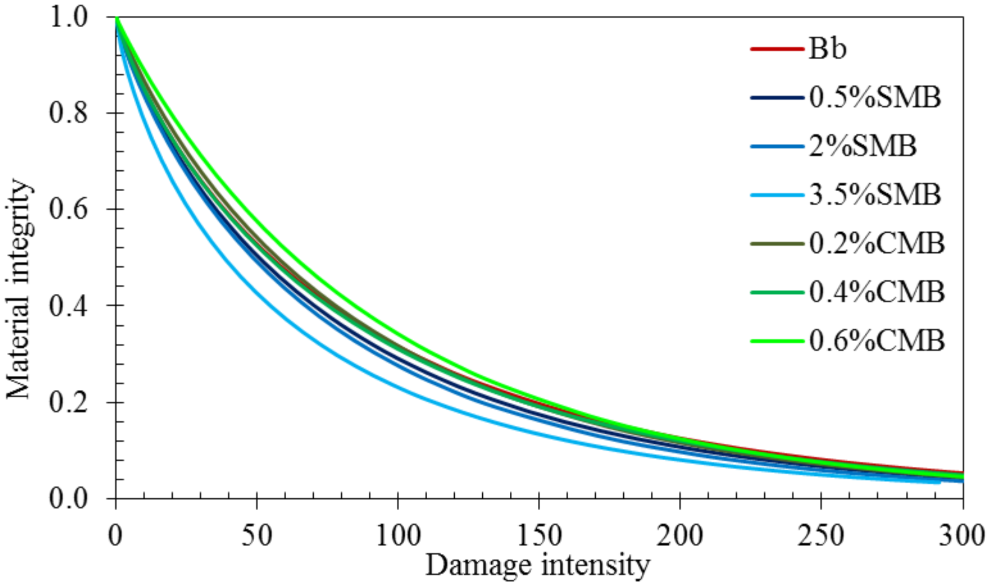


Figure 3-28. Damage characteristic curves

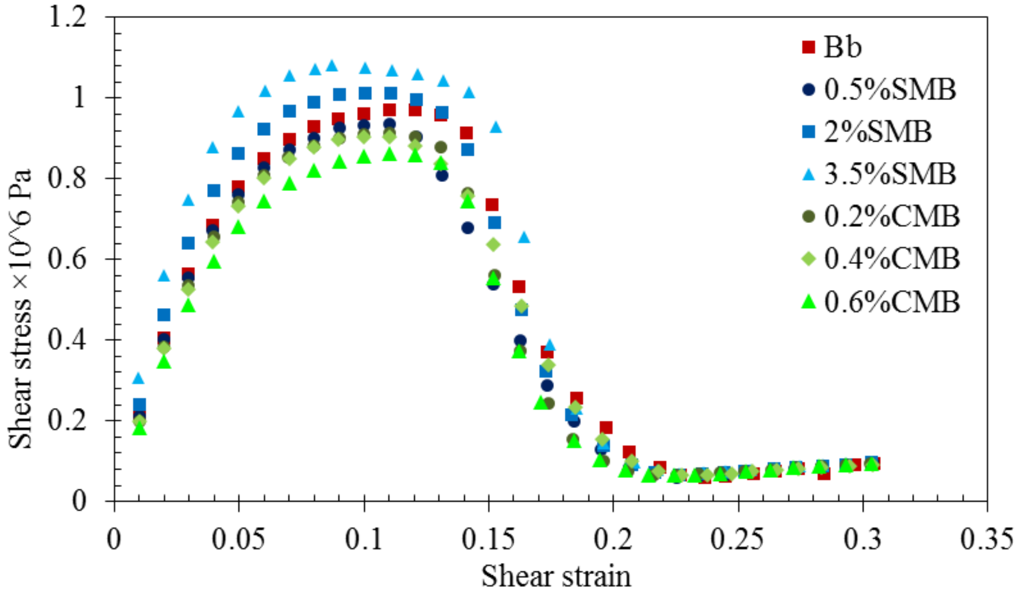


Figure 3-29. Identification of failure zone by the peak shear stress method

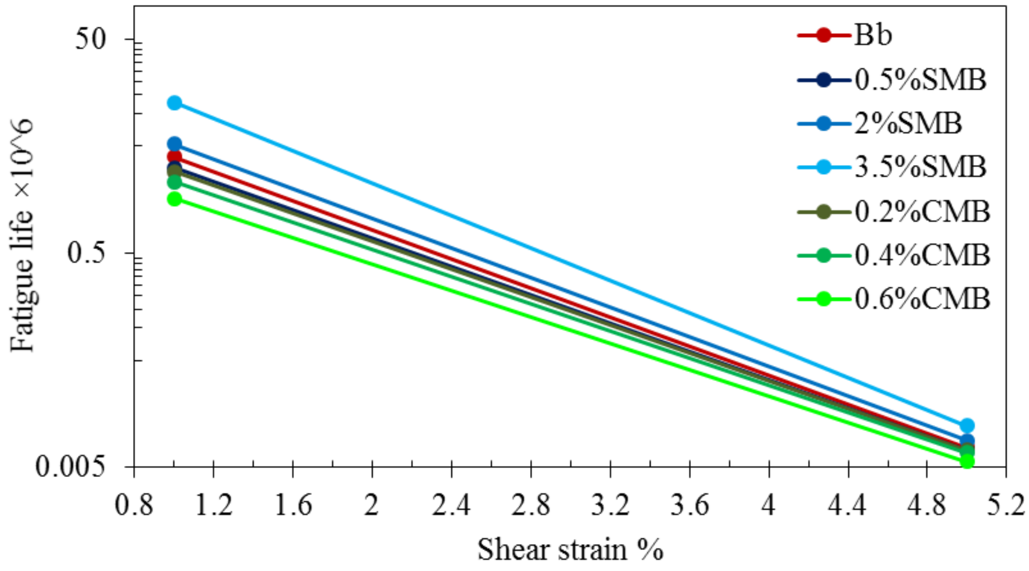


Figure 3-30. Fatigue lines of the studied binders

3.7 Summary and Conclusions

In this chapter, the effects of WMA additives on bitumen empirical and thermorheological properties were investigated. Based on the results of this chapter, the following conclusions can be drawn:

1. Penetration and softening point tests showed that Sasobit stiffens bitumen, whereas Cecabase does not have a clear impact on these measures.

2. Rotational viscosity tests indicated that Sasobit very marginally reduced binder viscosity. This reduction is equivalent to about 4°C reduction in the mixing temperature of asphalt if the equi-viscous approach is applied. This indicates that this approach is not a suitable method to determine mixing temperatures of WMA, and this conclusion coincides with previous research results. On the other hand, Cecabase did not reduce bitumen viscosity. Hence, it is not a viscosity reducer additive.

3. Frequency sweep test results for the unaged binder condition showed that Sasobit reduces the phase angle and increases shear modulus, and this trend is directly related to the additive concentration. The results also indicated that Cecabase did not alter the rheological properties of the base binder.

4. With respect to bitumen ageing, both of the additives retarded the bitumen ageing process as indicated by the critical intermediate temperature results which showed that SMBs and CMBs are susceptible to cracking up to 21°C after the combined RTFOT and PAV ageing, whereas Bb was susceptible to cracking up to 27°C.

5. MSCR results showed that Sasobit improves rutting resistance, whereas Cecabase may increase bitumen susceptibility to that distress type. By uncoupling the creep element from the recovery element, it was possible to understand the way WMA additives altered Bb properties and ultimately its response to stress. Sasobit improved elastic and creep recovery properties whereas Cecabase degraded bitumen stiffness noticeably and its creep recovery property slightly.

6. LAS results demonstrated that Sasobit can enhance fatigue life, and this result can be explained by the improved elastic and creep recovery properties, whereas Cecabase reduced fatigue life. This result contradicted the Superpave fatigue cracking parameter results, which indicated both additives can improve fatigue resistance. This is probably due to the poorer rheological properties of CMBs, which could not be captured by the Superpave parameter.

7. The main conclusion of this chapter is the unsuitability of the conventional and Superpave parameters to characterise WMA additive modified binders, and the importance to analyse the behaviour of these binders beyond the limits of linear viscoelasticity using advanced bitumen characterisation methods rather than the traditional methods.

Chapter 4: Design and Analysis of WMA Production Temperatures

4.1 Introduction

WMA was principally developed to decrease negative environmental impacts of asphalt by reducing production temperatures of this material. Most WMA-related studies mentioned that the available WMA techniques can offer about 10-40°C reduction in mixing temperature, this range often being determined based on either an equi-viscous approach or on the additive manufacturer's directions. However, both of these methods cannot explain or detect the effect of WMA additives on the production process of asphalt. Thus, the mechanism of the additives to reduce production temperatures of asphalt cannot be studied using these ways. Furthermore, the equi-viscous approach was approved to be ineffective in determining mixing temperatures of WMA, since it resulted in an unreasonable reduction (4°C) as concluded in chapter three. Moreover, are there any effects of the mixing temperatures on WMA performance? Because WMA performance should be at least comparable or even better than traditional HMA in order to be accepted as a satisfactory alternative (Capitão et al., 2012, Jamshidi et al., 2013).

This is a critical question because if there is a relationship between the performance and the production temperatures of WMA, then the production temperatures must be designed not only based on the reduction that an additive can offer but also based on the minimum temperature that allows for a comparable performance of WMA to a reference HMA. This means that an integrated design method of WMA that links the production temperatures of this mix with its performance is required. This kind of analysis should assure maximised reductions of WMA production temperatures without compromising its performance. Accordingly, this chapter introduces a novel methodology developed in this study to design WMA production temperatures based on the prescribed understanding. It also presents the results of applying the developed methodology to design the production temperatures of the selected WMA additives in this study.

4.2 Materials and Mix Design.

4.2.1 Binders and Aggregate

In this study, a base binder of grade 40/60 of a type frequently used in road surface layer construction has been used for the reference HMA. For WMA, two binders were used of the same grade of the control mix were used, one modified with 2% Sasobit and one with 0.4% Cecabase as recommended by the manufacturer of these additives. The empirical and rheological properties of these binders (Bb, 2%SMB, and 0.4%CMB) are presented and discussed in chapter three.

On the other hand, the aggregate used in the production of all studied mixtures was Granite aggregate obtained from Bardon Hill quarry, Leicestershire, UK. It was supplied as four fractions with different nominal sizes; 14, 10, 6.3 mm and dust. Table 4-1 presents some specifications of this aggregate. Accordingly, by mixing specific percentage of each size, the final aggregate batch met the grading requirements for 0-14 mm asphalt concrete surface layer according to British standard BS 4987-1:2005 (BSI, 2005), as shown in Figure 4-1. This aggregate gradation was used thoroughly in all mixes that did not contain RAP.

4.2.2 Mix Design

The asphalt concrete AC 0/14 surface course was designed in accordance with BS 4987-1:2005 (BSI, 2005). An optimum binder content of 5.1% was selected from the same standard, and the target air voids was 5%. The maximum theoretical specific gravity (G_{mm}) was determined according to procedure C of BS EN 12697-5:2009 (BSI, 2009) by using the densities of the constituent materials of asphalt, as follows:

$$G_{mm} = \frac{100}{\frac{P_{14}}{\rho_{14}}+\frac{P_{10}}{\rho_{10}}+\frac{P_{6.3}}{\rho_{6.3}}+\frac{P_{dust}}{\rho_{dust}}+\frac{P_{binder}}{\rho_{binder}}} \qquad \text{Eq. 4-1}$$

where P_x is the percentage of the designated material in the mix, and ρ_x is the density of the designated material in the mix. Accordingly, the density of the designed mixes can be calculated as follows:

$$\rho_{bulk} = G_{mm} \times (1 - V_v) \qquad \text{Eq. 4-2}$$

where ρ_{bulk} is bulk specific gravity of the mix, V_v is the air voids content in the mix. Following this method, a reference HMA was designed. Similarly, two WMA mixes were designed, one using Sasobit denoted by SWMA, the other one using Cecabase denoted by CWMA. The only difference between HMA and WMAs was the mixing and compaction temperatures of these mixes. In fact, NCHRP report 691 clearly states that WMA does not need a particular design method, but can be designed as a special case of HMA (Bonaquist, 2011a). Technically speaking, the main difference between HMA and WMA design is in determining mixing and compaction temperatures, which depends to a high extent on the selected WMA technology.

However, determining WMA production temperatures is still controversial because there is no accepted global framework for this. As indicated in chapter two, different methods were suggested in the literature to select or determine production temperatures of WMA. However, these methods could be valid only for limited applications or may result in unreasonable production temperatures. That could be the reason behind the conclusion of Bonaquist (2011a) who suggested to measure aggregate coating and mix workability to determine mixing temperatures of WMA and to determine compaction temperatures based on mixture compactability.

However, the specified coating measurement methods such as ASTM D2489 or AASHTO T-195 are for examining and assessing coating at asphalt plants. Moreover, in these methods, about 200-500 aggregate particles of a size larger than 9.5 mm are sieved and visually inspected, and the coating is calculated as the percentage of coated particles. It can be realised that this is a time-consuming process, and visual inspection precision can be questionable. Also, particles smaller than 9.5 mm are not considered in the coating evaluation. Accordingly, it was suggested in this research to quantify aggregate coating by application of image processing techniques in order to determine coating-based optimum mixing temperatures of WMA.

Table 4-1. Aggregate specifications

Aggregate type	Size mm	Specific gravity t/m^3
Granite	0-14	2.81
Granite	0-10	2.83
Granite	0-6.3	2.84
Granite	0-4	2.82

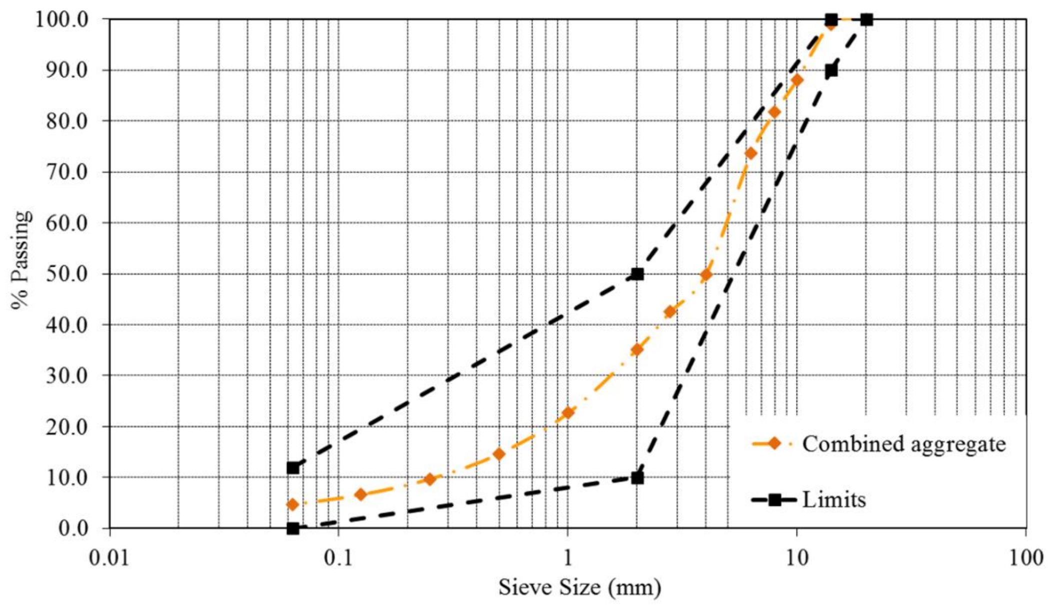


Figure 4-1. AC 0/14mm aggregate gradation

4.3 Aggregate Coating Quantification

4.3.1 Methodology

One of the main aims of this chapter was to quantify aggregate coating against mixing time to investigate the additive influence on coating, and to develop a method to determine mixing temperatures of WMA based on this fundamental property. However, as mentioned earlier, the available methods to examine coating are for mixing in asphalt plants and are based on visual inspection, the usual procedure being to determine the required time to achieve 95% of the coating. Accordingly, it was suggested in this study to directly determine the degree of aggregate coating during the mixing process by the application of image processing techniques. The methodology followed to achieve that aim was as follows:

1. Mix HMA and WMA at different reduced mixing temperatures. The mixing temperature of HMA was found to be 155°C based on the viscosity of the base binder. Accordingly, WMA and non-standard HMA (asphalt mixed at

reduced temperatures without WMA additives, coded by NSHMA) were mixed at 145, 130, and 115°C allowing for 10, 25, and 40°C reduction the mixing temperatures.

2. In order to collect the required data for aggregate coating quantification, the mixer was stopped during the mixing process every 10 seconds during the first minute of mixing then every 20 seconds until the mixture was fully coated and an image in RGB format was taken during every stop. These times were selected based on trial mixes, which showed that the coating changes dramatically over the first half minute of mixing then it gradually retards until thoroughly coating is achieved.

3. Study and analyse the collected photos in Matlab using a code that was written for this purpose. Matlab was chosen to analyse the data because it offers powerful image processing tools, and the analysis can be conducted on the entire collected data in less than a minute, which saves a lot of time.

4.3.2 Development of Image Processing Tool

The first step in the proposed image analysis was to convert the image from RGB format to grayscale format using the following equation (Mathworks®, 2016):

$$\mathrm{I} = 0.2989 \times \mathrm{R} + 0.5870 \times \mathrm{G} + 0.11 \times \mathrm{B} \qquad \text{Eq. 4-3}$$

where *I* is image index in grayscale which has a minimum value of zero for black and a maximum value of 255 for white, *R*, *G* and *B* are the values of red, green and blue colours respectively in RGB format. The next step was to filter the data in order to reduce the noise by using an averaging filter. However, to reduce the effect of the filter on the data, it was applied to a 3-by-3 neighbourhood of pixels only.

Figure 4-2 shows an example of converting an image to grayscale; whereas Figure 4-3 shows an image histogram before and after the filtering process. It can be seen in this figure that the binder and the aggregate have a range of index values rather than a single value. Therefore, index values can be used to determine percentages of the binder (coated areas) and aggregate (uncoated areas). Accordingly, the percentage of the coated area can be calculated using the following equation:

$$\mathrm{CA\%} = \sum_{\mathrm{LLBI}}^{\mathrm{ULBI}} \mathrm{im}(i,j) / \sum_{0}^{255} \mathrm{im}(i,j) \times 100 \qquad \text{Eq. 4-4}$$

where *CA* is the percentage of coated area, *LLBI* is the lower limit of the binder intensity, *ULBI* is the upper limit of the binder intensity, *im* is the image being analysed with *i* and *j* dimensions. This equation seems to be reasonably easy to solve, but the difficulty is in determining the right threshold value that identifies the binder and aggregate accurately. The lower limit of the binder index is zero, representing pure black. However, the upper limit is the parameter that can most affect the results because if a value higher than the correct value is used then the coated area determined will be higher than its actual value and vice-versa. Figure 4-4 presents two histograms; the first is for a 100% coated asphalt mix while the second is for aggregate only before mixing. It can be seen that there is an overlap in the intensities of the binder and the aggregate and that this overlap could cause an error since, in the overlapped areas, the probabilities of a pixel belonging to either binder or aggregate are close. On the grayscale shown in Figure 4-4, the overlapped area is represented by a grey colour that has a range of intensities between 80 and 150. In this range, it is hard to judge whether the pixel belongs to the binder or the aggregate. In other words, if a pixel has an intensity of 120, for example, this pixel has almost the same probability of being binder and aggregate.

To overcome this problem, the overlapped area has been excluded from the calculations by excluding the overlapped intensities. Thus, excluding the pixels that have intensities that fall in the "grey" range. This procedure reduces the source of uncertainty in the data and improves the accuracy of the results. Mathematically:

$$\mathrm{CA} = \sum_{0}^{\mathrm{ULBI}} \mathrm{im(i,j)} / \left(\sum_{0}^{255} \mathrm{im(i,j)} - \sum_{\mathrm{LLOI}}^{\mathrm{ULOI}} \mathrm{im(i,j)} \right) \times 100 \qquad \text{Eq. 4-5}$$

where *ULOI* and *LLOI* are the upper and lower limits of the overlapped intensities, and other abbreviations are as previously defined. Furthermore, to validate the results of this image analysis method, two 'masks' were created for the binder and the aggregate. The binder mask has an intensity of 0, which appears as black colour, whilst the aggregate mask has an intensity of 255, which is white. In this way, the results can be validated by comparing the processed image before and after processing. Figure 4-5 presents three examples of images before and after analysis. It can be seen that the Matlab code captures the coated and uncoated areas accurately, as shown in the figure. This means that the developed Matlab code and the applied image analysis approach can accurately discriminate between the coated and uncoated areas. Therefore this method was successfully applied to quantify aggregate coating.

Figure 4-2. Image format conversion from RGB (left) to grayscale (right)

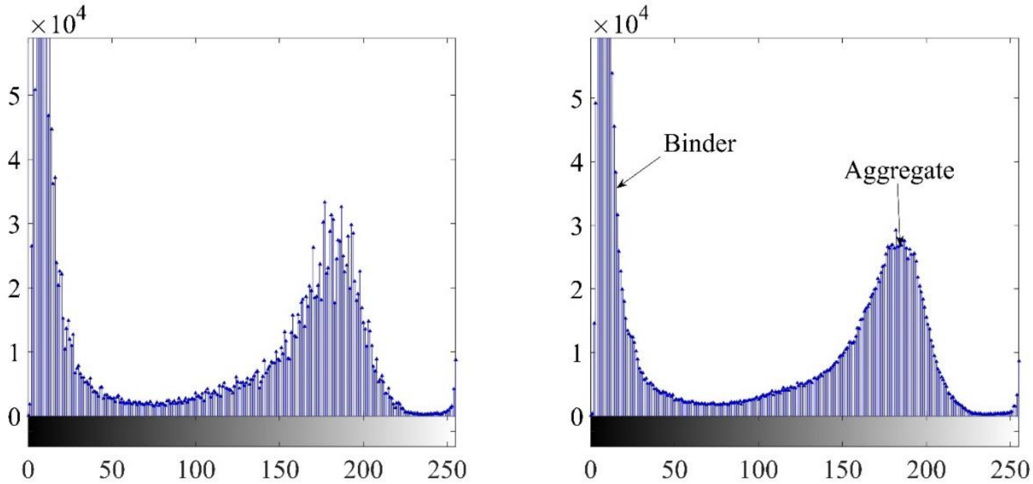


Figure 4-3. Histograms of an image before (left) and after filtering (right)

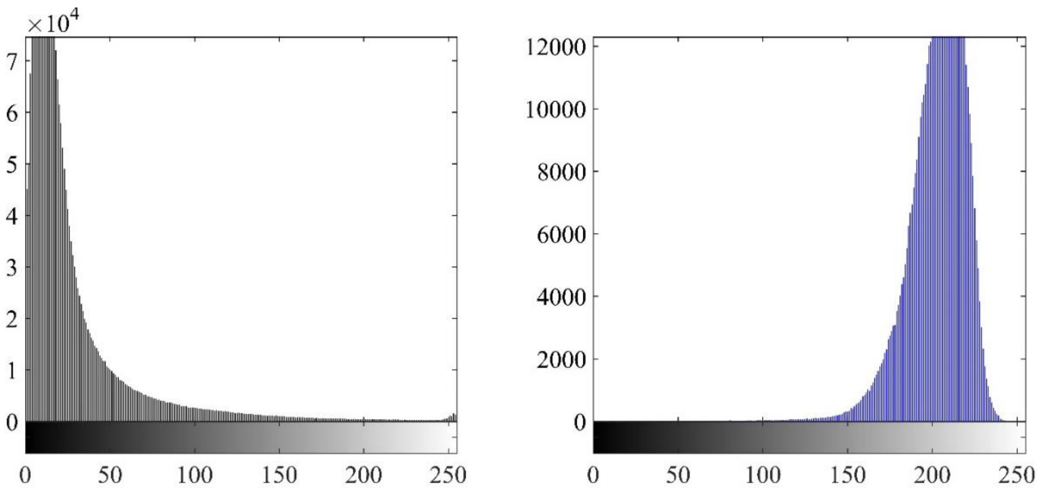


Figure 4-4. Histograms of two images: entirely coated asphalt (left), aggregate only (right)

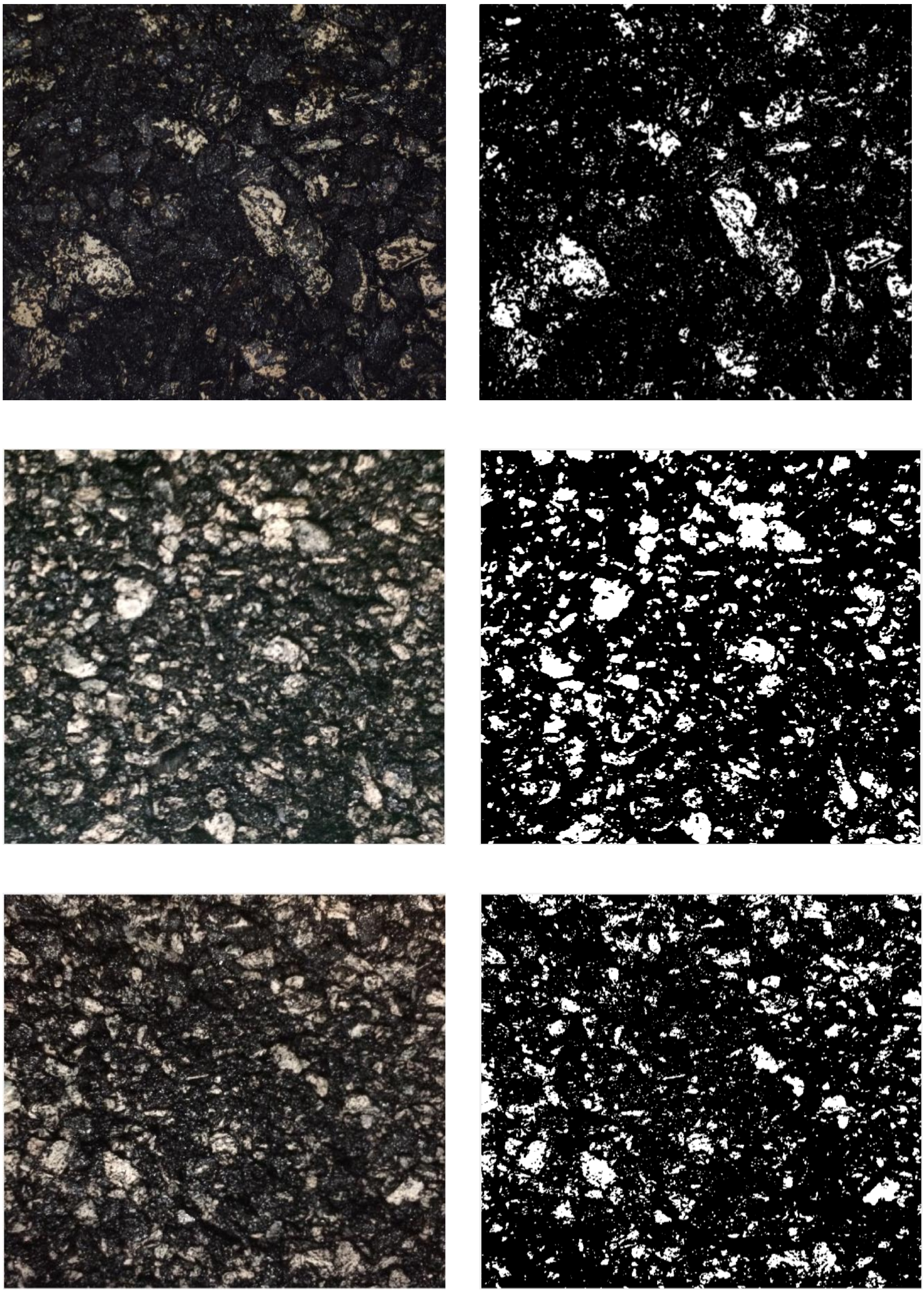


Figure 4-5. Examples of images before (left) and after processing (right)

4.3.3 Results of Aggregate Coating

Based on the suggested aggregate coating analysis approach, a control HMA, a non-standard HMA denoted by NSHMA, WMA produced using Sasobit denoted by

SWMA and WMA produced using Cecabase denoted by CWMA, were mixed at 155 (for the control HMA), 145, 130 and 115°C. To acquire the required data for coating analysis, the mixing was stopped, and an image was taken every 10 seconds for the first minute of mixing, then every 20 seconds subsequently. Every mixing experiment was repeated two times. Using the derived aggregate coating method, the collected data was analysed and Figure 4-6, Figure 4-7, and Figure 4-8 present the results of the analysis of all mixtures. Generally speaking, the coating significantly increases over the first forty seconds of mixing (rapid coating stage), then it tends to gradually increase until it reaches after a specific mixing time (gradual coating stage). To model this behaviour, the following power function was implemented:

$$\text{Coating}\ \% = A \times mixing\ time^{B} + C \qquad \text{Eq. 4-6}$$

where *A*, *B* and *C* are constants. This model was found to be a best fit to the coating behaviour. The coefficient of regression (R^2) values of the fitted functions and model constants are presented in Table 4-2; it can be seen that R^2 ranging from 0.77 to 0.99, indicating high goodness of fit to the data. the fitted models are also presented in the figures of aggregate coating results.

Regarding the effect of additives on coating, the results demonstrate that both of the additives can improve mixing at temperatures as low as 115°C. However, a longer mixing time was required at this low temperature to achieve full aggregate coating. Moreover, mixing HMA down to 145°C can be done without using any additive as shown in the figures, but it required slightly more mixing time than that when mixing at 155°C. However, mixing at 130 or 115°C without additives required a longer mixing time, about 90 and 160 seconds respectively to achieve an apparently acceptable coating, which is significantly longer than the standard mixing time in asphalt plants, about 30-40 seconds (Hunter et al., 2015).

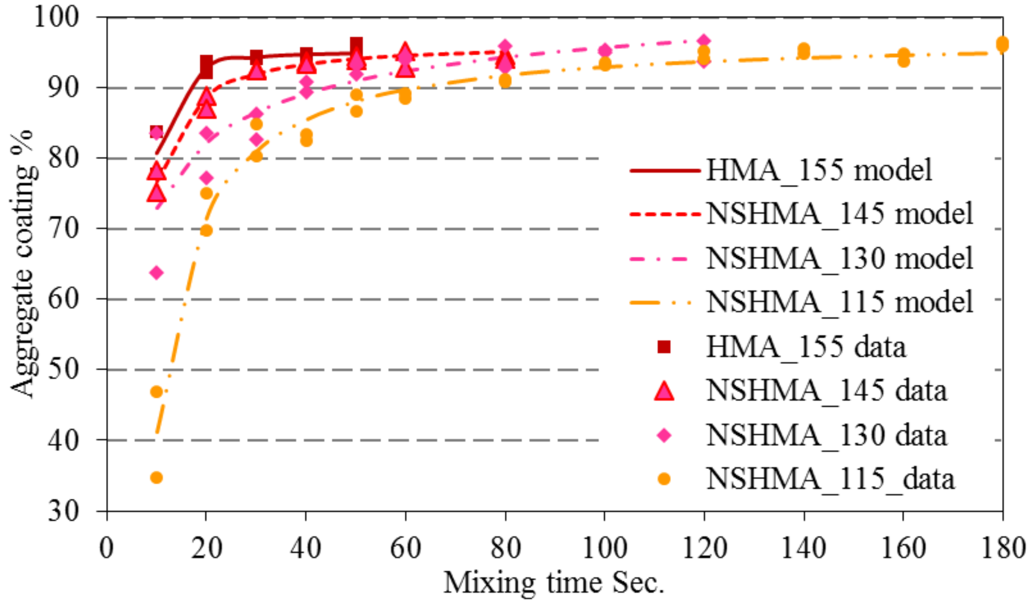


Figure 4-6. Aggregate coating results of HMA and NSHMA

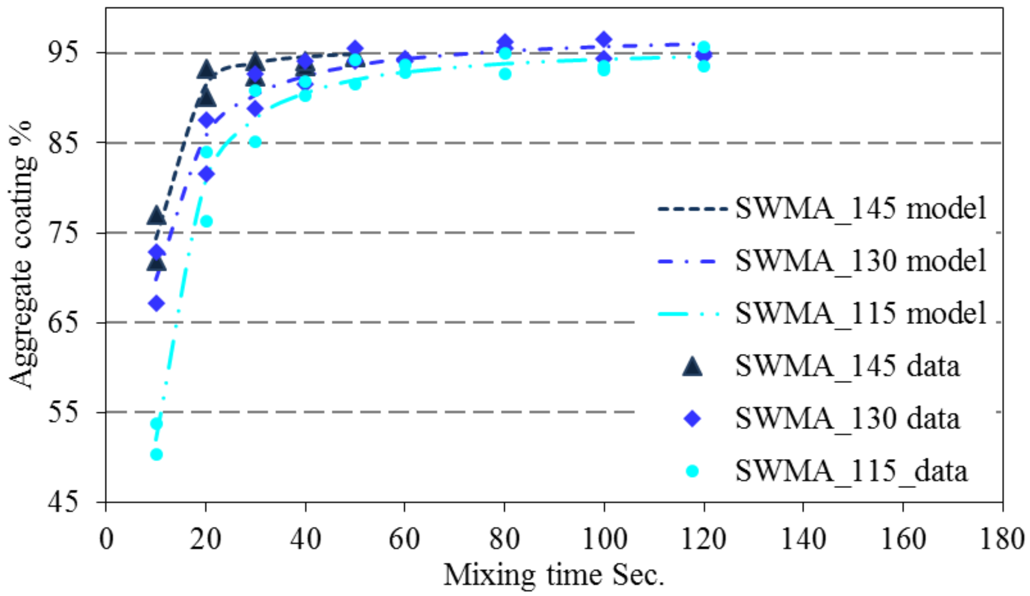


Figure 4-7. Aggregate coating results of SWMA

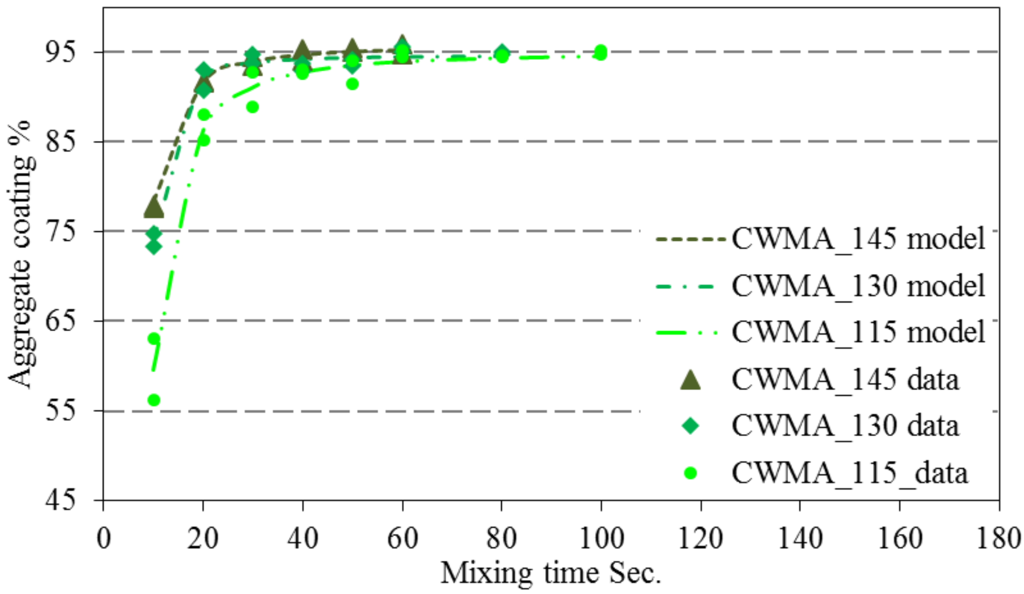


Figure 4-8. Aggregate coating results of CWMA

Table 4-2. Model parameters and R^2 of aggregate coating results

MIX	A	B	C	R^2
HMA 155	-6340	-2.645	95.1	0.93
NSHMA 145	-464	-1.38	96.14	0.97
NSHMA 130	-96.6	-0.42	109.6	0.77
NSHMA 115	-766.2	-1.137	96.99	0.98
SWMA 145	-5733	-2.438	95.34	0.96
SWMA 130	-498.8	-1.259	97.27	0.95
SWMA 115	-1632	-1.573	95.57	0.98
CWMA 145	-2213	-2.104	95.71	0.99
CWMA 130	-16160	-2.895	94.65	0.99
CWMA 115	-3800	-2.032	94.9	0.97

Furthermore, mixing asphalt without any additives at 115°C could not be adequately achieved, despite the results showing that it was coated to about 95% in 160 seconds since this was in part an apparent coating and the fine aggregate beneath the sample surface was not properly coated. Figure 4-9 shows an example of this problem. It can be seen that the fine aggregate particles or dust were uncoated. This implies that at this high binder viscosity, most of the binder coats the large aggregate particles with a thick binder film due to their low surface area, whilst the dust particles require more binder for coating due to their high surface area. Evidently, this could not be achieved at 115°C because of the agglomeration of the binder due to its high viscosity when mixing at low temperatures. It is worth mentioning that this problem was not noticed when using WMA additives, and both large and fine aggregate particles were coated entirely at the end of the mixing process. This can only be explained by the influence of the additives, which play an essential role in improving coating at mixing temperatures lower than standard ones.

Figure 4-9. Non-standard HMA mixed at 115C°; please note uncoated fines and dust particles when mixing asphalt at 115°C without WMA additives

In order to understand and establish a relationship between mixing time and temperature, 95% particle coating was selected as the required aggregate coating level, since this value is recommended by the Transportation Research Board (2012). Therefore, the time required to achieve this coating level was plotted against mixing temperature, and it was found that a power-law relationship can approximately represent the data, as shown in Figure 4-10. This chart shows the relationship between mixing temperatures and the time required to achieve sufficient aggregate coating, and it can be used as a guide to design mixing temperatures of WMA. Based on the results of this chart, it can be seen that if 40 seconds of mixing time is considered, the HMA can be mixed at 155°C, whilst both SWMA and CWMA can be mixed at 145°C, a 10°C reduction. To increase the reduction in mixing temperature, mixing time could be increased to 60 seconds, which may still be considered reasonable, in which case SWMA could be mixed at 135°C and CWMA at 130. This could mean that mixing temperature can be reduced by about 20°C for SWMA and 25°C for CWMA. Nevertheless, this also means that HMA can be mixed at approximately 145°C without using WMA additives if mixing time is increased to 60 secs, a 10°C reduction in mixing temperature. This implies that increasing mixing time can definitely help in reducing mixing temperatures, even without using WMA additives.

Moreover, it can be seen in the results that none of the samples achieved 100% coating, and this may be due to two factors: firstly, the light intensity when taking images; the mixer used in this experiment was located in a room where it was quite difficult to control this factor, and it may, therefore, have affected image index values and thereby given rise to error; secondly, the reflectivity of the light on the binder makes spots on the binder look white, and these spots were calculated as uncoated areas since they had index values more than 230. This factor also leads to error, especially when asphalt is almost fully coated because that makes the area reflecting the light larger. Figure 4-11 clearly illustrates the error in coating data due to these factors. It can be seen that after 10 seconds of mixing (A) uncoated areas are present with indexed values between 150-255. After 30 seconds of mixing (B), most of the previously uncoated areas have been coated. After 50 seconds of mixing (C), the sample should be 100% coated (because these images are related to mixing of HMA at 155°C), and this point was confirmed during the experimental work where the sample was visibly 100% coated. However, the image analysis shows that there are still some uncoated areas. This is evidence of the error due to light reflection and intensity factors. The average error, which is calculated as the difference between the coating results of images corresponding to 100% of actual coating and the results of those images calculated by image processing, was 4.9%.

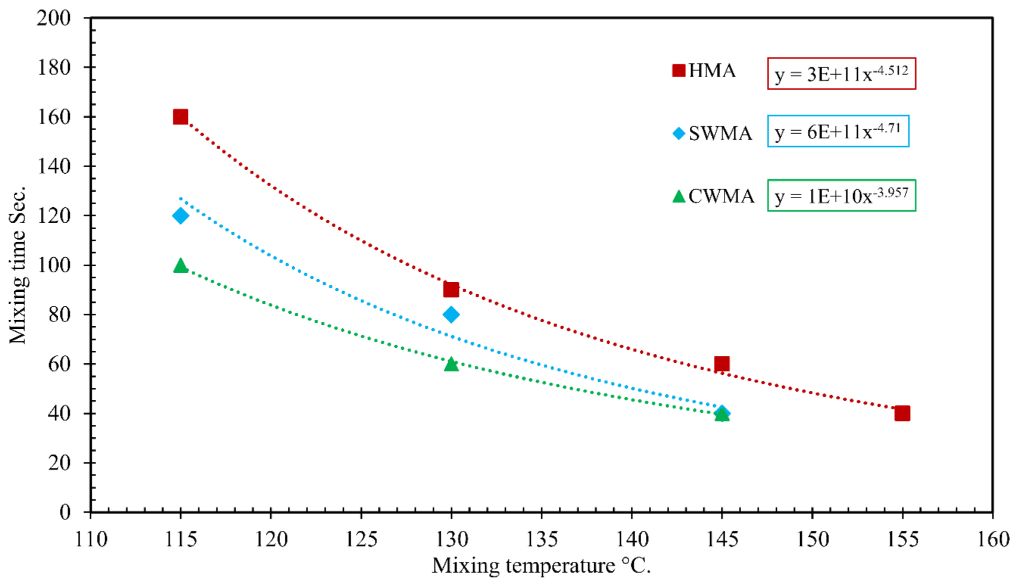
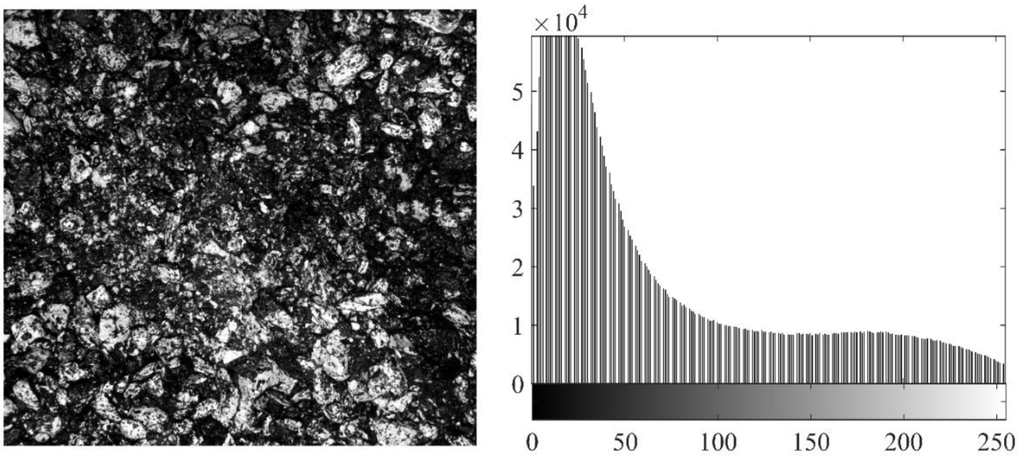
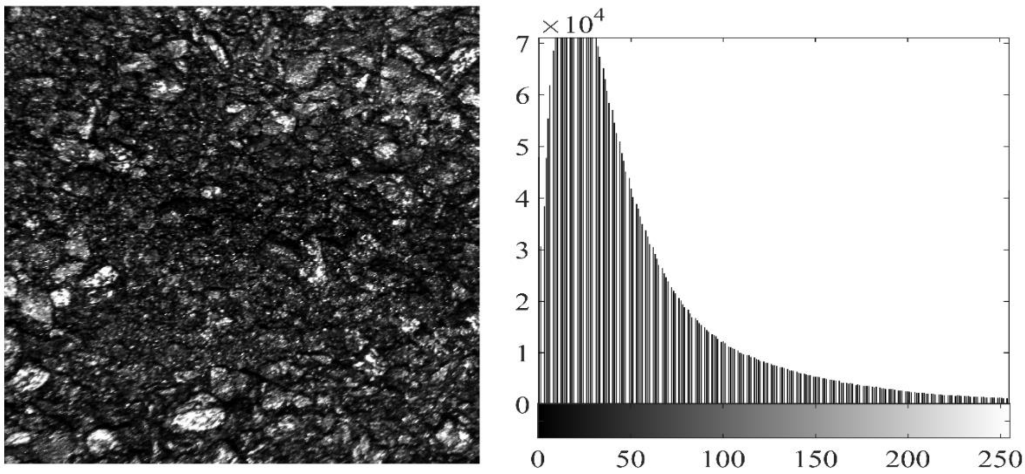


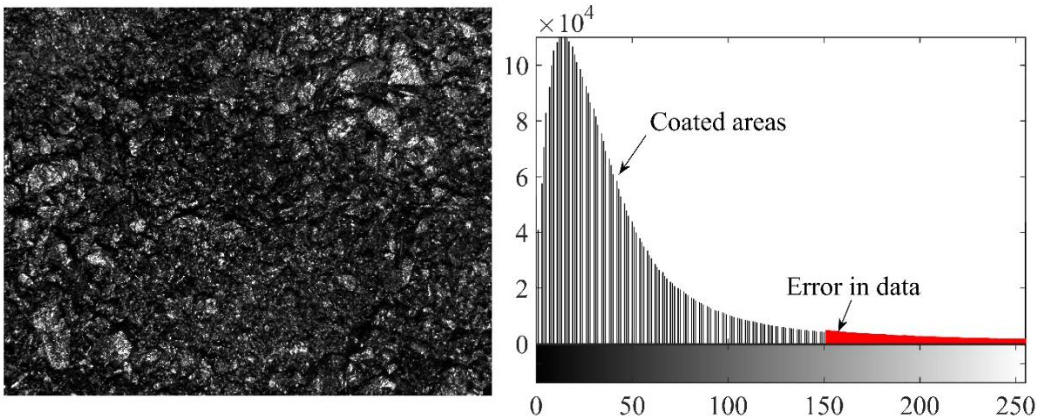
Figure 4-10. Mixing time and temperature relationship



A



B



C

Figure 4-11. Images with greyscale histograms after A 10 sec, B 30 sec and C 50 sec mixing time; HMA at 155C°

4.4 Performance of WMA

The previous section explained the relationship between mixing temperatures of WMA and aggregate coating since this is the property that WMA additives should improve during asphalt mixing when the production temperature is reduced. However, does providing sufficient coating mean equivalent performance to reference HMA? Also, is there a relationship between mixing temperatures and performance of WMA? These questions can only be answered by examining WMA performance at different reduced temperatures, then investigating if the results are affected by the mixing temperatures. To apply this concept, two performance indicators were selected in this study; indirect tensile stiffness modulus (ITSM) and indirect tensile strength (ITS). The reasons for selecting these measures were because they are fundamental properties of asphalt, and they can be measured in a reasonable time.

To prepare the required testing specimens, asphalt slabs of al studied mixtures were fabricated by heating binder and aggregate to the designated mixing temperature of each mix, then mixing and compacting materials in steel models with dimensions of 305×305×65 mm using a roller compactor. The compaction temperature of all mixtures was kept 110°C in order to isolate the effects of this factor on mix performance. Finally, cylindrical samples were cored from the slabs, and the top and bottom surfaces of the cores were cut so that the final sample dimensions were 100 mm diameter and between 40-45 mm height since these dimensions comply with the standards followed to perform ITSM and ITS tests.

4.4.1 Indirect Tensile Stiffness Modulus

Stiffness modulus is one of the most critical indicators of asphalt mechanical properties. It can describe the stress-strain relationship under different temperatures and frequencies, and it has been used in pavement analysis and design for several decades. This modulus can be determined in several ways, including bending tests on prismatic samples, direct triaxial or uniaxial loading, and indirect tensile tests (Hunter et al., 2015). The indirect tensile stiffness modulus (ITSM) testing method was applied in this study using the Nottingham Asphalt Tester (NAT) machine, which shown in Figure 4-12. The test was conducted in accordance with BS EN 12697-26:2004 (BSI, 2004b). The test temperature was 20°C, so all specimens were stored in a temperature-controlled cabinet at the test temperature for at least four hours before testing. The test was conducted in controlled strain mode within the linear viscoelastic region, a horizontal deformation of 5 microns was achieved by applying the required stress, and the rise time was set to 124 microstrains as recommended by the test standard. The stiffness modulus can be calculated as follows:

$$S_m = F * (\upsilon + 0.27)/(Z * H) \qquad \text{Eq. 4-7}$$

where S_m is the stiffness modulus in MPa, F is the peak applied force in N, υ is Poisson's ratio, Z is the horizontal deformation in mm and H is the specimen thickness in mm.

Figure 4-13 presents the test results; it can be seen that there is a relationship between mixing temperature and ITSM, but that it is different from one mix to another. The NSHMA shows a very slight dependency on mixing temperature, but the results are quite comparable to that of the control HMA. CWMA also shows only a slight dependency on the mixing temperature, although the ITSM values are almost 10% lower than for HMA. However, SWMA shows a significant dependence on mixing temperature; the higher the mixing temperature, the higher the ITSM. The slope of the fitted lines can be used as an indicator of temperature dependency. Both NSHMA and CWMA have comparable slopes (4.59 and 3.39), whereas SWMA has a significantly higher dependency, 34.46, which shows the strong relationship between production temperatures and that kind of WMA. To validate the statistical significance of these results, a T-test statistical analysis between the control HMA and the other asphalt types was performed, as shown in Table 4-3. It can be seen that all ITSM results are comparable or insignificantly different from the control HMA, except the Sasobit samples produced at 130 and 145°C which were significantly stiffer than the control HMA. This result may be explained by the blending and dissolving of Sasobit in the base bitumen. It is true that this material has a softening point of about 100°C, but it seems that the dissolving of this material is a temperature-dependent; the higher the mixing temperature, the better the blending, which in turn increases binder stiffness eventually mix stiffness. Furthermore, these results prove that there is a relationship between mixing temperatures of some types of WMA and its performance.

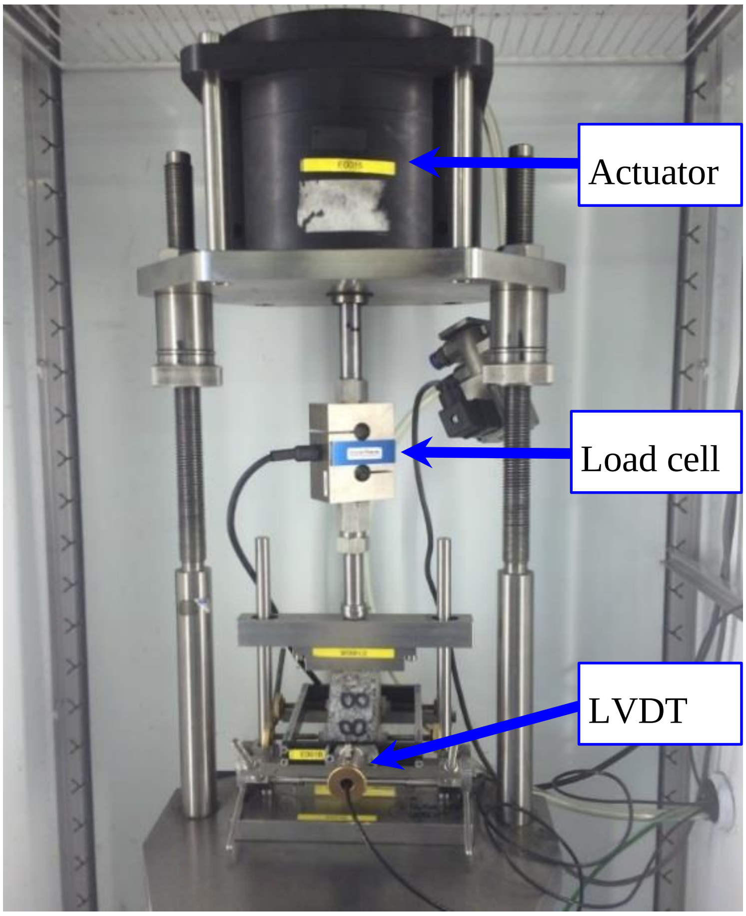


Figure 4-12. Nottingham Asphalt Tester

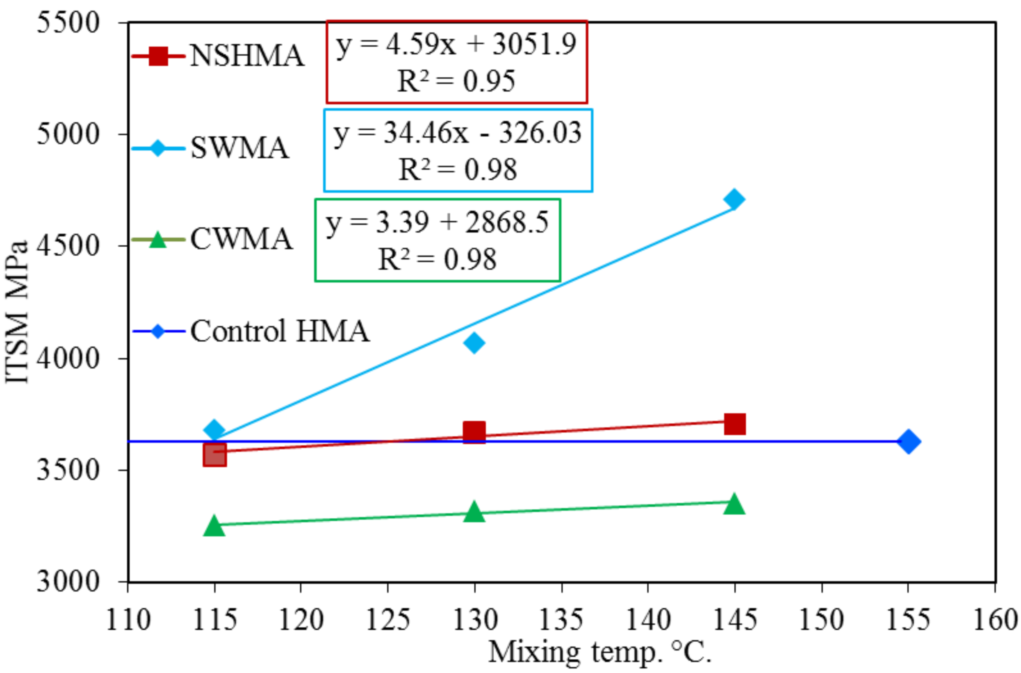


Figure 4-13. ITSM results

4.4.2 Indirect Tensile Strength

ITS is another critical parameter that can be used in assessing material quality. It has been reported that this indicator is correlated with rutting and fatigue cracking life; higher strength leading to reduced rutting, whereas fatigue life increases exponentially with an increase in the strength (Khosla and Harikrishnan, 2007). These results show the significant correlation of this factor with advanced pavement performance indicators such as rutting and fatigue cracking. Several other studies have also used this indicator to evaluate fatigue performance of asphalt mixtures based on fracture mechanics (Roque et al., 1999, Roque et al., 2015). Accordingly, this indicator was implemented in this research to evaluate HMA and WMA performance. The test was performed according to BS EN 12697-23:2003 (BSI, 2003b) at 20°C using an Instron servo-hydraulic loading frame shown in Figure 4-14. As per the test standard, a monotonic load was applied at a rate of 50 mm/min. During the test, the force was recorded every 50 N while the deformation was recorded every 50 microns, the strength is a function of the maximum load at failure as in the following equation:

$$ITS = 2 * P/\pi * D * H \qquad \text{Eq. 4-8}$$

where P is the force at failure, D is the diameter and H is the height of the specimen. Figure 4-15 presents the ITS test results. It can be seen that there is a similar influence of temperature to that for ITSM. The ITS of NSHMA has very little dependence on mixing temperature down to 130°C, with a slight drop off at 115°C, and comparable results to the control HMA. Very similar behaviour can be seen for the CWMA mix. However, the SWMA mix shows a similar relationship with mixing temperature as that for ITSM, except that while the ITSM results are all higher than the control HMA those for ITS are lower for mixes at 130°C and 115°C. To validate the statistical significance of these results, a T-test was conducted between ITS results of all mixtures and the control HMA and the results are presented in Table 4-3. The results indicate that none of the ITS measurements is significantly different from the control mix, except the SWMA mixed at 115°C. This means that mixing the SWMA lower than 130°C causes a significant reduction in the ITS in comparison with the control mix, with the same ITS as the control mix achieved at about 135°C. Accordingly, SWMA should not be mixed lower than 130°C if a comparable strength to the control HMA to be achieved.

Moreover, the ITS results also confirm that there is a strong relationship between SWMA mixing temperatures and its performance. Therefore, the mixing temperature of this mix should be determined based on two criteria; coating and strength, the stiffness was excluded since it was higher than that of HMA at all mixing temperatures. The highest of the mixing temperatures resulting from these measures should assure equivalent performance and aggregate coating. In this case, SWMA can be produced at 135°C for one minute to assure sufficient coating and comparable strength. On the other hand, CWMA performance did not show dependence on mixing temperatures; therefore, its mixing temperature can be selected based on the desired mixing time. Considering 1 minute of mixing leads to producing this mix at 130°C.

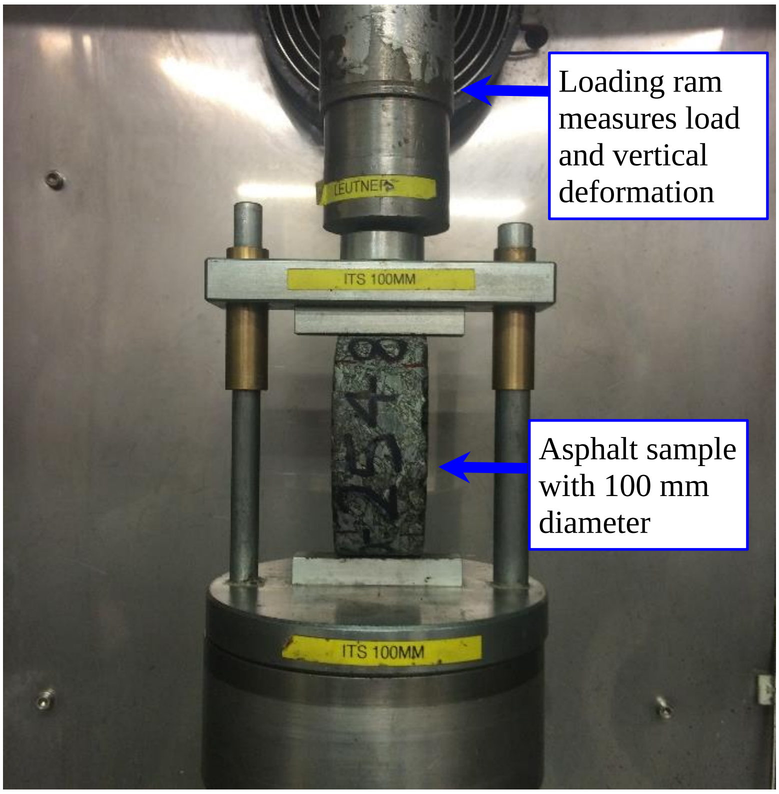


Figure 4-14. ITS test setup

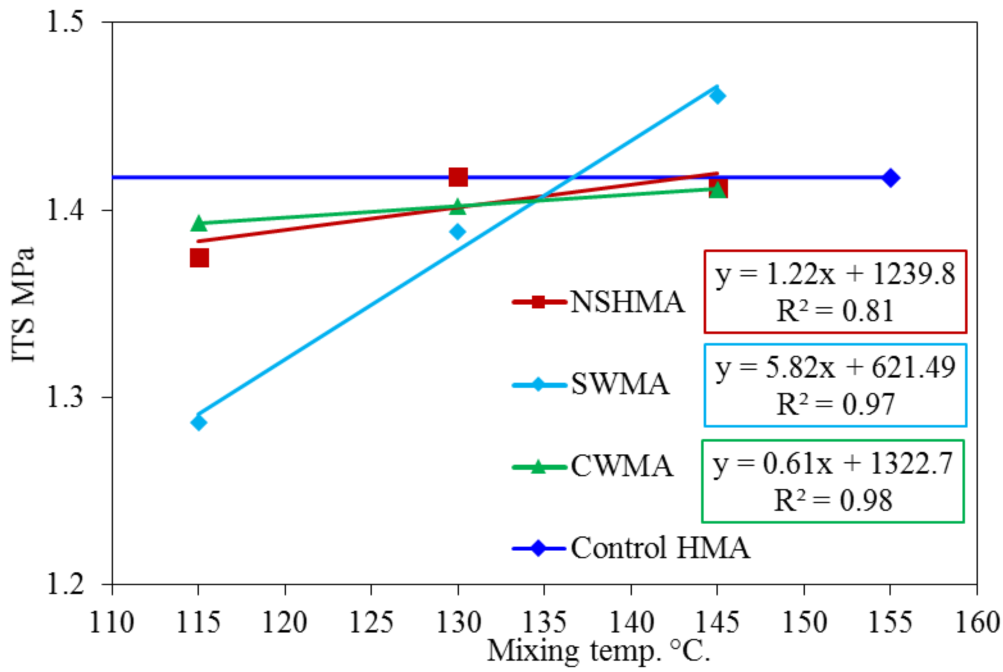


Figure 4-15. ITS Results

Table 4-3. ITSM and ITS statistical analysis results

Mix type	Mixing temp. °C	Additive	ITSM		ITS	
			P value	Significant	P value	Significant
NSHMA	145	None	0.34	No	0.46	No
NSHMA	130	None	0.39	No	0.49	No
NSHMA	115	None	0.41	No	0.26	No
CWMA	145	Cecabase	0.34	No	0.44	No
CWMA	130	Cecabase	0.09	No	0.36	No
CWMA	115	Cecabase	0.08	No	0.14	No
SWMA	145	Sasobit	0.005	Yes	0.34	No
SWMA	130	Sasobit	0.04	Yes	0.4	No
SWMA	115	Sasobit	0.42	No	0.03	Yes

4.5 Workability Analysis of WMA

In order to fully understand the effects of the studied additives, their impacts on mixture workability were investigated. Workability was evaluated by recording the effect of WMA additives on the compactability of the mixtures. Traditionally, the gyratory compaction method has been used to assess the workability of asphaltic mixtures. However, some studies have reported that this compaction method is insensitive to compaction temperature (Huner and Brown, 2001, Jalali, 2016). It was pointed out in these studies that the reason for the insensitivity of gyratory compaction to compaction temperature is basically the nature of compaction in this method, which is strain-controlled compaction. This means that regardless of the compaction temperature, the compactor applies the required compaction shear stress (not the axial stress) to compact the sample.

In the light of the findings of these studies, and as an alternative, the workability was evaluated in this study by the amount of compaction effort applied by the roller compactor to achieve the required density. The compaction effort was calculated by counting the number of passes of the roller compactor at every compaction pressure, since the machine applies four compaction pressures, 172, 276, 379, and 483 kPa. The compaction procedure was to apply a number of passes up to a maximum of 10 at the first compaction pressure; if the sample was not compacted to the required density, then the compaction pressure was increased to the second level and so on until the samples were compacted to the desired air voids. The number of passes at every level was then converted to an Equivalent Number of Passes (ENP) at 172 kPa pressure by multiplying the number of passes at every compaction level by the compaction pressure and dividing by 172 kPa. It can be seen that this method is quite material and manpower intensive. On the other hand, it could be an accurate method to evaluate asphalt compactability because the roller compaction method is the technique that is most simulative of field compaction of asphalt.

Table 4-4 and Figure 4-16 present the compactability results collected from HMA and WMA slabs produced during this study. Every number in the table is calculated from an average of 3–7 asphalt slabs compacted at the designated compaction level. These results demonstrate that the control mix required 50.8 ENP to be compacted at 150 °C, whilst it needed 70.4 ENP to be compacted at 110 °C. On the other hand, the CWMA required 39.3 and 54.2 to compact to the target density at 120 and 110 °C, respectively. It can clearly be seen that the chemical additive improved asphalt workability as the compaction effort is reduced by about 23% in comparison with the NSHMA at 110 °C compaction temperature. The SWMA also enhanced mixture workability as it improved compactability as low as 110 °C with approximately 24% reduction in the compaction effort in comparison with NSHMA. To estimate an optimum compaction temperature for WMA mixtures, a linear interpolation was applied assuming that the target compaction effort is 50.8, which was the effort required by the control mix when compacted at 150 °C. Based on this assumption, the optimum compaction temperature for CWMA is 113 °C and for SWMA is 116 °C with 32 and 29 °C reduction in the compaction temperature respectively. This conclusion implies that the studied additives are more beneficial in the compaction process than the mixing process.

Table 4-4. Compactability results

Mix type	Compac-tion tem-perature C°	Compaction pressure KPa				Equivalent number of passes at 172 KPa
		172	276	379	483	
HMA 145	145	10	10	10	0.67	50.0
NSHMA 110	110	10	10	10	8.00	70.5
CWMA 120	120	10	10	5.86	0.14	39.4
CWMA 110	110	10	10	10	2.20	54.3
SWMA 125	125	10	10	8.80	0.00	45.4
SWMA 110	110	10	10	10	2.00	53.7

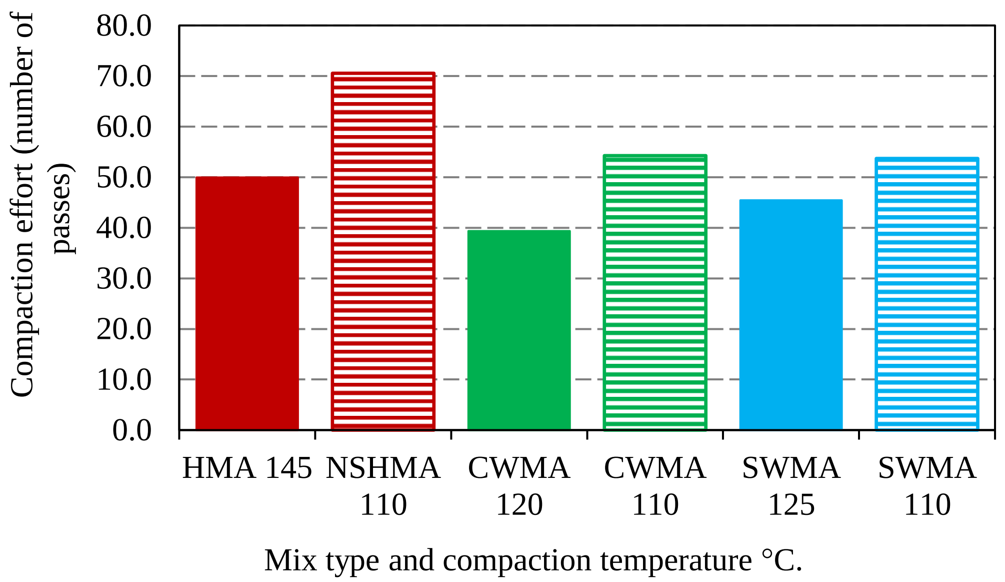


Figure 4-16. Analysis of compactability results

4.6 Summary and Conclusions

In this chapter, the effects of mixing temperatures of WMA on aggregate coating and performance were investigated. The coating was quantified as a function of mixing time and temperature by means of image processing. The performance of WMA as a function of mixing time was investigated in terms of ITSM and ITS. The coating results showed that the additives improved coating even when mixing at 115°C. However, when reducing mixing temperatures, a longer mixing time was required to achieve sufficient coating regardless of whether the mix contained additives or not. But in the presence of WMA additives, less additional mixing time was required than in the case of NSHMA.

On the other hand, correlating mixing temperatures with the performance revealed that the performance of SWMA significantly depends on the mixing temperature. This result may be explained by the blending between the base binder and the Sasobit. Increased mixing temperature leads to higher dissolving and better blending of Sasobit, which in turn stiffens bitumen and increases mix stiffness. Meanwhile, mixing temperature did not exhibit any effect on CWMA performance at all mixing temperatures. However, it marginally reduced ITSM by about 10% and ITS by less than 5% in comparison with the control HMA. These results suggest that assuring sufficient coating does not necessarily assure acceptable WMA performance, especially when organic or wax additives are incorporated because the performance of this kind of WMA showed a strong correlation with the mixing temperatures of asphalt.

Accordingly, an optimum mixing temperature can be designed by integrating aggregate coating results with the performance results. Applying this concept to the studied additives suggest that SWMA can be mixed even at 115°C. However, by considering ITS results in the analysis, SWMA should be mixed at 135°C to assure comparable strength; accordingly, this temperature was considered as optimum mixing temperature for that mix. On the other hand, CWMA performance did not depend on mixing temperatures; accordingly, it can be designed based on a coating criterion, and it is considered that 1 minute of mixing results in an optimum mixing temperature of 130°C for CWMA which was followed in this study. Moreover, workability evaluation of asphalt by the roller compactor showed that the used WMA additives were more effective in enhancing asphalt compactability than improving aggregate coating at reduced temperatures. This was evident in the compaction results which showed that CWMA can be compacted at 113°C and SWMA at 116°C when applying the same compaction effort used to compact the control mix at its standard compaction temperature.

Chapter 5: Design and Analysis of WMA Containing RAP

5.1 Introduction

Warm recycling of asphalt is one of the critical contemporary topics of asphalt engineering. This mixture has received a great deal of attention from asphalt organisations because of its expected economic and environmental advantages represented by the combined benefits of WMA technologies with RAP. In fact, some researchers clearly stated that combining asphalt recycling with WMA technology is the best approach to achieving a sustainable and environmentally friendly asphalt industry (Giani et al., 2015).

Incorporation of RAP in asphalt mix design is a complicated dilemma. The interaction between RAP binder and aggregate with the virgin materials depends on several factors such as RAP quantity, properties, and asphalt production conditions. Accordingly, the designer must make some assumption in the design process such as the selection of the virgin binder grade, the amount of useful RAP binder in the mix, or the degree of blending between RAP and virgin binders. These assumptions should be accurate because they can significantly impact the mechanical performance of the designed mix.

In contrast, the degree of blending (DoB) between RAP and virgin materials has been reported as one of the critical factors that affects performance of mixtures containing RAP. DoB controls the quality of the produced asphalt in terms of different factors such as mix homogeneity and mechanical performance. If the DoB is insufficient, then the properties of the mix are variable from one location to another. Because in this case RAP aggregate particles are coated by binder with different properties than the virgin binder, i.e. poor binder blending. This means that the asphalt produced will be inhomogeneous as its characteristics vary within the mix. On the other hand, mechanical performance is significantly affected by DoB because it depends on the properties of the mix. If the properties are variable due to an insufficient DoB, then mix response to the applied stress will be variable as well. Some parts of the mix may exhibit severe cracking while other parts may exhibit rutting due to the variation of asphalt properties over the mix.

Accordingly, this chapter investigates the design and analysis of WMA containing a high percentage of RAP. Particular attention was paid to study and quantify DoB, and to analyse its impact on the resulting mix properties. Several methods to quantify DoB found in the literature, as reported in chapter two. These methods, however, mostly depend on separating RAP particles from the virgin aggregate particles to study binder properties extracted and recovered from each part, which means they depend on binder properties in the quantification process.

In this research, a new simple but effective method to quantify DoB and to analyse its impact on the volumetric and mechanical response of the mix is introduced. Critical mix design factors such as mixing temperatures and mixing time are investigated since these factors can significantly alter DoB. Furthermore, the impact of DoB on mix volumetric and mechanical properties are studied because this area is still unclear and requires more investigation.

5.2 Mix Design of WMRA

Conventional HMA design is a straightforward process. The bitumen grade, aggregate gradation, and production conditions of asphalt can be selected based on local standards and material properties. When RAP is incorporated in asphalt

(AC_RAP), however, mix design becomes a complicated process due to several factors, including RAP percentage, RAP binder properties, and DoB. RAP incorporation level controls the amount of RAP binder introduced into the mix; increasing RAP quantity leads to increasing the amount of the aged binder in the mix. Consequently, reducing mix workability and compactability. In this regard, several studies demonstrated the usefulness of using WMA additives to aid workability and compactability of these mixtures (Kristjánsdóttir et al., 2007, Tao and Mallick, 2009, Dinis-Almeida et al., 2016). These studies demonstrated that the primary role of WMA additives is to enhance the poor workability of asphalt mixing containing RAP. This also means more RAP can be incorporated into the mix without compromising mix compactability by using WMA additive as compaction aiders.

In this study, a detailed mix design method was developed to incorporate high percentages of RAP up to 50% to be produced within the WMA temperature production range. A novel analysis method was then followed to analyse resulting mix performance and to optimise production conditions in order to produce WMRA with similar characteristics to the control HMA. The following sections explain the details of the developed mix design and analysis methods.

5.2.1 RAP Incorporation level

In many countries, RAP incorporation level is still limited to a maximum of 30% due to several technical reasons such as the capacity of common asphalt plant to accommodate more RAP, standard specifications which still restrictive when it comes to increasing RAP content, or even the mistrust which is mainly related to the uncertainty in the performance of this mixture. However, several studies, as mentioned in the literature review, showed the tendency and the applicability to increasing RAP level in order to achieve more economical and environmental benefits. This has been accomplished by using WMA additives, which alleviated the technical problems related to mix workability and compactability. These studies also showed the potential to incorporate high percentages of RAP without compromising mix performance. Accordingly, in this study, a RAP percentage of 50% (RAP aggregate as a percentage of the total aggregate skeleton) was incorporated since several studies have investigated lower dosages of RAP.

5.2.2 RAP Compositional Analysis

The RAP used in this study was supplied by FM Conway, London, UK, obtained by milling of road surfaces. The RAP was characterised by conducting a compositional analysis to obtain RAP binder content and RAP aggregate gradation. The analysis showed that RAP binder content was 5.7 with penetration grade of 17 dmm, and RAP aggregate consisted of fractions as shown in Figure 5-1. It can be seen that the RAP has high quantity of fine particles, this could be attributed to the crushing and milling of aggregate particles during the reclamation process of RAP. Nevertheless, to design aggregate gradation of WMRA, specific proportions of virgin aggregate was mixed with the RAP aggregate to match the gradation of the control mix, AC 14mm. The final grading is shown in Figure 5-1. It can be seen that there is a very good matching between the control mix grading and the WMRA mix. This step is critical to eliminate or minimise any effects of differences in aggregate gradation on the performance of the studied mixtures.

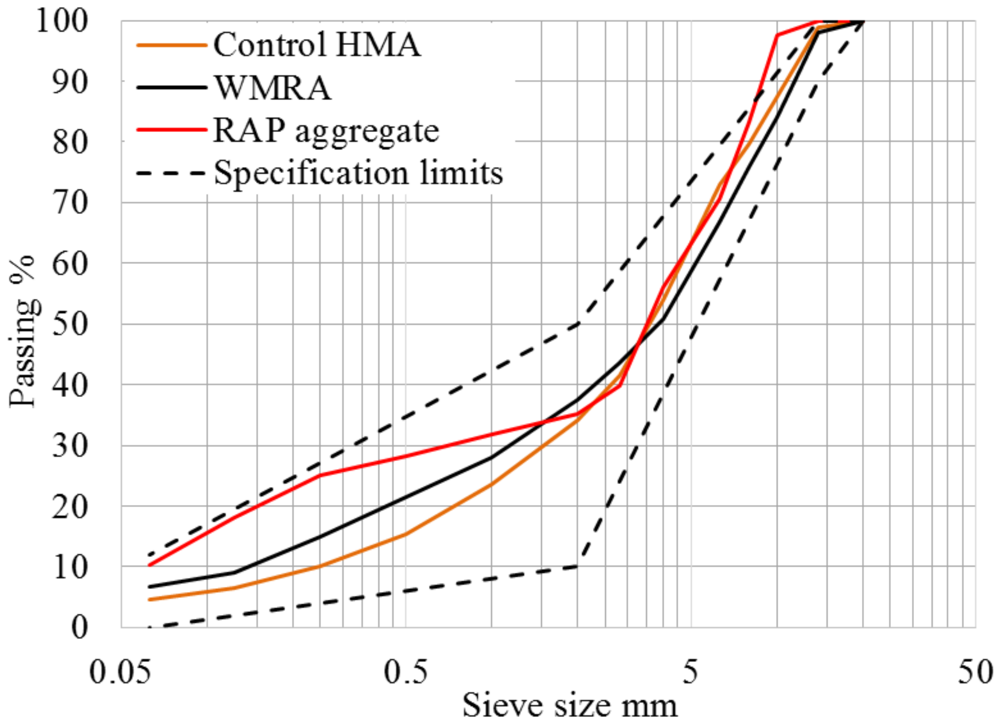


Figure 5-1. Aggregate gradation of the control HMA aggregate, RAP aggregate, and AC mix containing 50% RAP

5.2.3 WMRA Mix Design

Having determined RAP content, it is possible to design the mix and calculate the required virgin binder grade and content by following the design according to a known RAP percentage method reported in chapter two. The detailed design procedure is as follows:

1. Based on the nominal aggregate size of the control mix, the RAP was sieved on sieve size 16 mm and all particles larger than this size were removed.
2. Rap binder extraction and recovery were carried out according to BS EN 12697-3:2013 (BSI, 2013).
3. Determinate RAP binder content according to BS EN 12697-1:2012 (British Standards Institute, 2012).

4. Measure the particle size distribution of RAP aggregate after extracting the binder according to BS EN 12697-2:2015 (BSI, 2015c).

5. Determinate RAP maximum theoretical specific gravity according to British Standard BS EN 12697-5 following the volumetric procedure.

6. Determinate the RAP aggregate effective specific gravity using the following equation (Hossain et al., 2010):

$$Gse_{RAP\ aggregate} = (100 - RAP_{bc})/(100/Gmm_{RAP} - RAP_{bc}/Gb_{RAP}) \quad \text{Eq. 5-1}$$

where $Gse_{RAP\ aggregate}$ is the effective specific gravity of the RAP aggregate, Gmm_{RAP} is the maximum theoretical specific gravity of the RAP, RAP_{bc} and Gb_{RAP} are RAP binder content and the specific gravity of the RAP binder. *Gse* of the RAP aggregate was determined to calculate the maximum theoretical specific gravity of the WMRA mix using Equation Eq. 4-1 so that it can be used to calculate air voids content in the samples.

7. Calculate the required mass of RAP aggregate based on designed RAP% as follows:

$$RAP_{agg} = RAP\% * total\ aggregate\ mass \quad \text{Eq. 5-2}$$

where RAP_{agg} is the mass of RAP aggregate, $RAP\%$ is the designed percentage of the RAP in the mix.

8. Calculate the mass of RAP binder that will be introduced into the mix, as follows:

$$RAP_b = RAP_{agg} \times (\frac{1}{1-RAP_{bc}} - 1) \quad \text{Eq. 5-3}$$

where RAP_b is the mass of RAP binder. Accordingly, the RAP mass that has to be introduced into the mix is basically the sum of RAP aggregate weight calculated using Equation Eq. 5-2 and RAP binder weight using Equation Eq. 5-3.

9. Calculate the mass of the rejuvenator binder and the percentage of each binder with respect to the total binder weight in the mix, as follows:

$$Rej_b = Total\ binder\ mass - RAP_b \qquad \text{Eq. 5-4}$$

$$Rej_{b\%} = Rej_b \ / \ Total\ binder\ mass \qquad \text{Eq. 5-5}$$

$$RAP_{b\%} = RAP_b \ / \ Total\ binder\ mass \qquad \text{Eq. 5-6}$$

where Rej_b is the mass of the required soft binder in the mix, $Rej_{b\%}$ is the percentage of the soft binder and $RAP_{b\%}$ is the percentage of the RAP binder with respect to the total binder weight in the mix.

10. After determining the percentages of each of the binders, the required grade of the rejuvenator can be calculated using the following logarithmic equation (BSI, 2006):

$$log\ pen_{blend} = a * log\ pen_{RAP\ binder} + b * log\ pen_{rejuvenator} \qquad \text{Eq. 5-7}$$

11. Doing the required mathematical manipulation, the grade of the rejuvenator binder that brings the RAP binder into a specific binder grade can then be calculated as follows:

$$pen_{rejuvenator} = 10^{((log\ pen_{blend} - a * log\ pen_{RAP\ binder})/b)} \qquad \text{Eq. 5-8}$$

where $pen_{rejuvenator}$ is the required binder grade, pen_{blend} is the target grade which is the grade of the control mix binder, pen_{RAP} is the grade of the RAP binder, a and b are the proportions of RAP binder and rejuvenator respectively.

Following this procedure, a mix design Excel sheet, as shown in Table 5-1 was designed. The sheet calculates the grade and weight of the required virgin binder based on the grade and quantity or RAP in addition to the mix standard volumetric properties.

5.2.4 Blending Charts and Blended Binder Properties

Based on the suggested mix design procedure, the proportions of the rejuvenator and RAP binder were determined, while the rejuvenator grade was calculated based on Equation Eq. 5-8. For 50% RAP aggregate incorporation, RAP binder percentage was 56% of the total binder. With these conditions and using a RAP binder of penetration 17.3 dmm and setting the target binder to 49.0 dmm, the soft binder grade required was 188 dmm. Figure 5-2 shows the proposed blending chart for the blended binders, and Table 5-2 shows penetration test results of the blended binders based on the calculated proportions. It can be seen that the rejuvenated binder penetration (47.3 dmm) is quite close to the target binder penetration (49.0 dmm) which implies successful RAP binder rejuvenation and the use of the correct binder proportions.

Table 5-1. Mix design sheet for asphalt mixtures containing RAP

Mix design sheet for WMRA mixtures				
Target air voids %				5
Maximum density kg/m^3				2.559
Target density kg/m^3				2.431
Binder content %				5.1
Volume of mould m^3				0.00608634
No. of samples				1
Scale-up factor				1.005
RAP binder %				5.7
RAP aggregate %				94.3
RAP binder grade (pen dmm)				17
Target binder grade (pen dmm)				49
Volumetric properties	Mass %	Mass (g)	Scaled up mass (g)	Mass for all samples (kg)
RAP weight 16-5029		7,445	7,482	7.482
RAP aggregate (for design)	50.0	7,021	7,056	
Aggregate size 14 mm	10.0	1,404	1,411	1.411
Aggregate size 10 mm	0.0	0.0	0.0	0
Aggregate size 6.3 mm	15.0	2,106	2,117	2.117
Dust	25.0	3,510	3,528	3.528
Sum	100.0	14,042	14,538	14.538
Total binder (kg)		0.7546	0.7584	0.758
Binder from RAP(g)		426		
Required virgin binder weight (g)		332	334	0.334
RAP binder %				0.56
Base binder %				0.44
Required virgin binder grade (pen)				188

Moreover, Figure 5-3 presents the results of rotational viscosity tests of the RAP binder and the soft binder in comparison with the control binder, conducted using a Brookfield viscometer. The results indicate that blending of the RAP binder with the soft binder based on the suggested proportions results in a rejuvenated binder with quite comparable viscosity to the target binder, which confirms the previous conclusion. Furthermore, to further validate the results of the suggested blending proportions, rheological properties of all binders were tested using a DSR by running frequency sweep tests and constructing complex modulus master curves. Figure 5-4 shows the constructed shear modulus master curves. The effect of the soft binder on the rheological properties of the hard binder can clearly be seen, also the match between the rejuvenated binder and the target binder properties, which suggests the recovery of the RAP binder's rheological properties. Moreover, Figure 5-5 is an isochronal plot of the parameter G*/sinδ at 10 rad/sec frequency. The rejuvenation process using the soft binder shifted the G*/sinδ results to values close to those of the target binder, which means that the rejuvenated binder has similar rheological properties to the target binder. Also, the critical high temperature which is the temperature at which the parameter G*/sinδ equals 1.0 kPa (AASHTO, 2013b) was determined as shown in Table 5-2. The results show that the critical high temperatures for the target binder and the rejuvenated binder are quite similar, with a difference of 1.4°C which means that both binders have a similar critical high temperature.

These results suggest that Equation Eq. 5-8 has accurately determined the grade of the required soft binder, and binder blending based on the penetration criterion can satisfy the Superpave performance grade requirements since both binders have a similar critical high temperature after the rejuvenation process. The results also suggest that both binders will perform similarly if the blending between the soft binder and RAP binder is 100%. This point is quite crucial because estimating RAP binder participation and the degree of blending depends on this assumption; if the two binders are 100% blended in the mix, then a similar performance to that of the control HMA should be obtained. If the blending is less than 100%, a difference in the performance between WMRA and HMA will be observed, which also means that the production conditions have to be adjusted until similar performance to the control HMA is obtained.

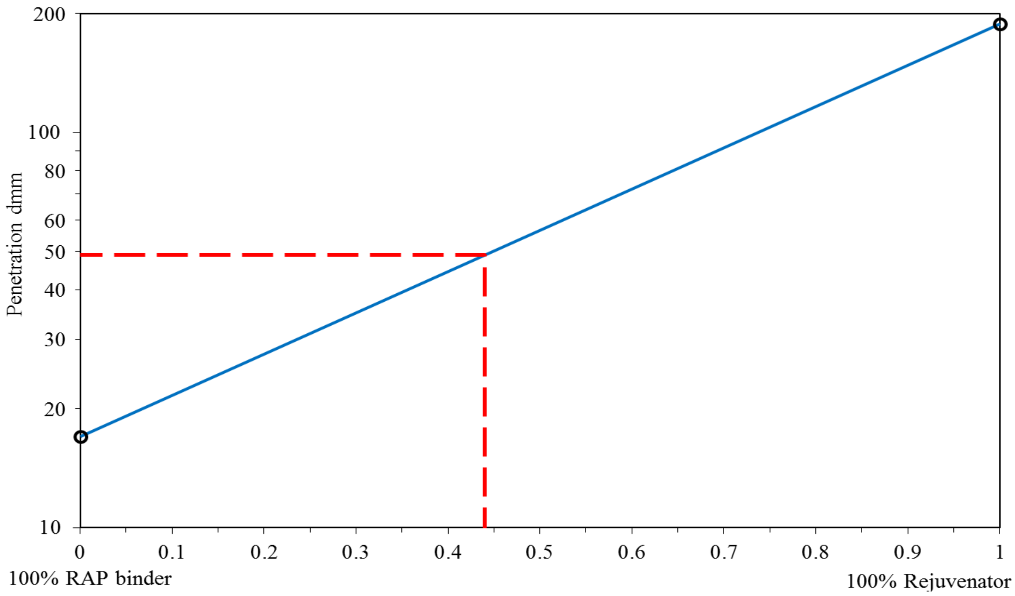


Figure 5-2. RAP and soft binder blending chart

Table 5-2. Properties of the RAP binder, rejuvenator, and the base binder

Indicator \ Binder type	RAP binder	Rejuvenator	Base binder	Rejuvenator binder
Penetration dmm	17.3	188.3	49.0	47.7
Critical high temperature °C	81.6	57.8	69.8	71.2

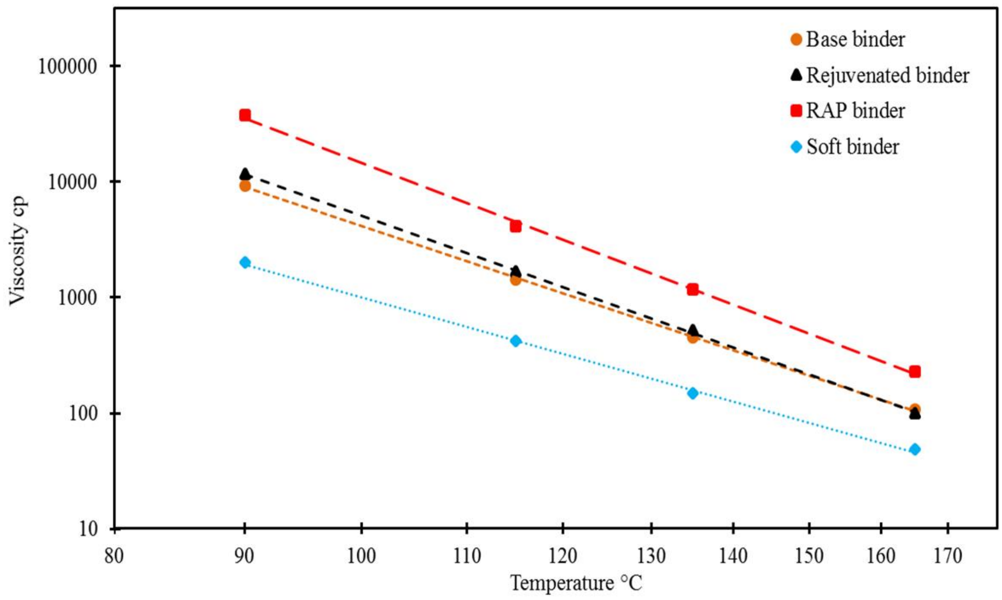


Figure 5-3. Rotational viscosity results

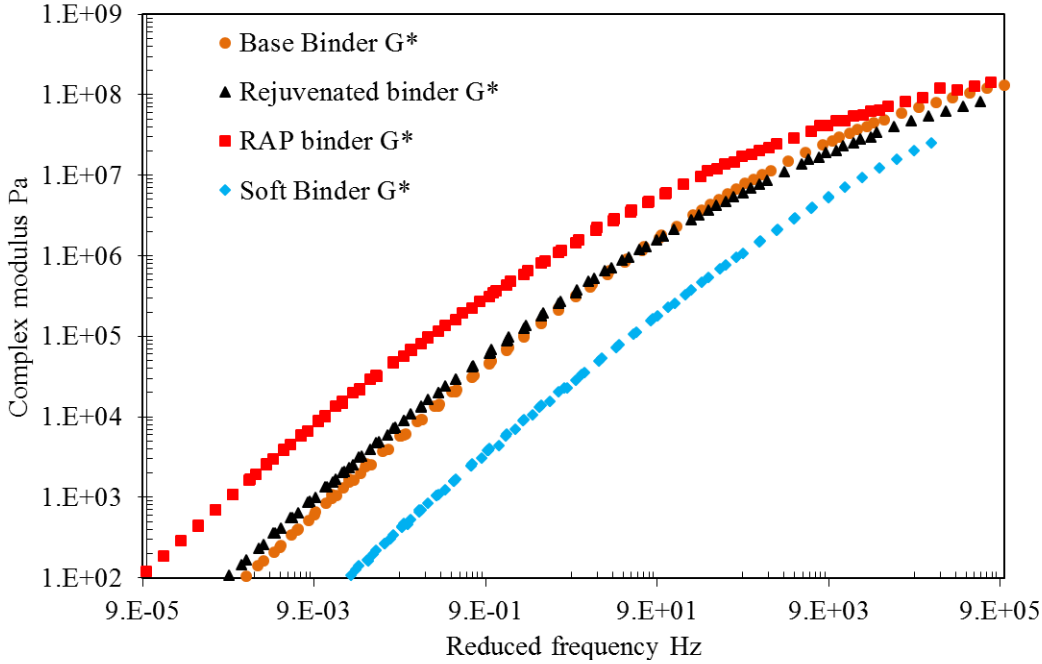


Figure 5-4. Complex modulus master curves

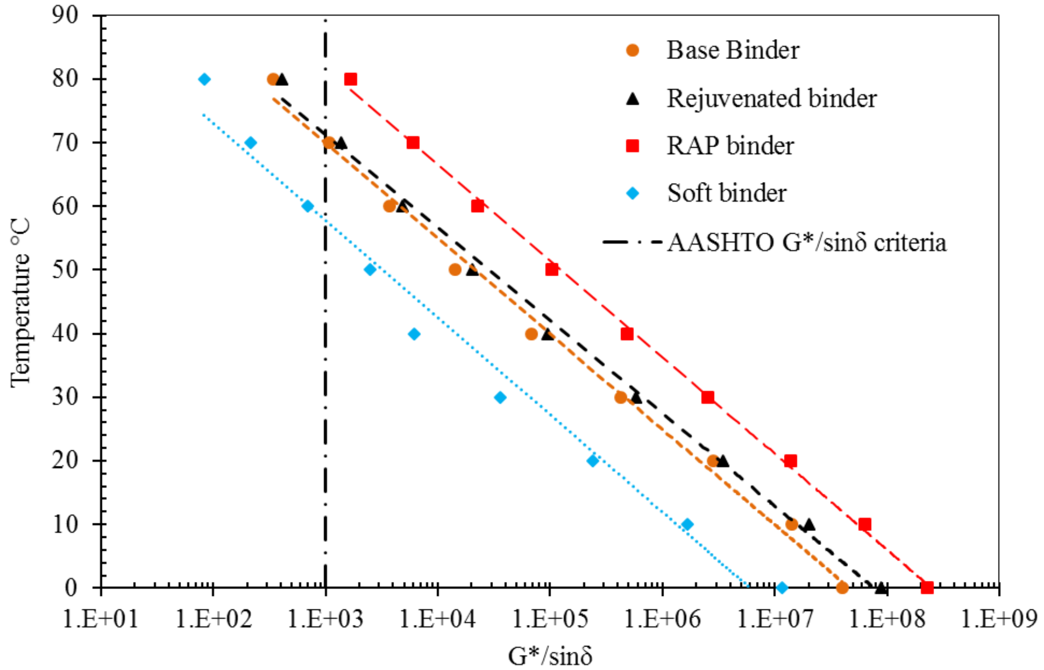


Figure 5-5. Critical high-temperature parameters

5.2.5 Asphalt Mixing and Compaction

After determining the mix design proportions, the control HMA was mixed and compacted according to the standard production temperatures of 40/60 binder grade. The aggregate was heated overnight, and the binder was heated for 3 hours; then the ingredients were mixed in a bucket mixer and compacted at the designated compaction temperature in square moulds to prepare slabs with dimensions 306 x 306 mm. After that, cylindrical specimens with 100 mm were cored, and the top and bottom were trimmed in order to obtain homogeneous samples for testing. The choice of sample diameter was due to compliance with the testing standards used in this study to evaluate the mechanical properties of the studied mixtures. In the case of fabricating RAP samples, the following procedure was followed:

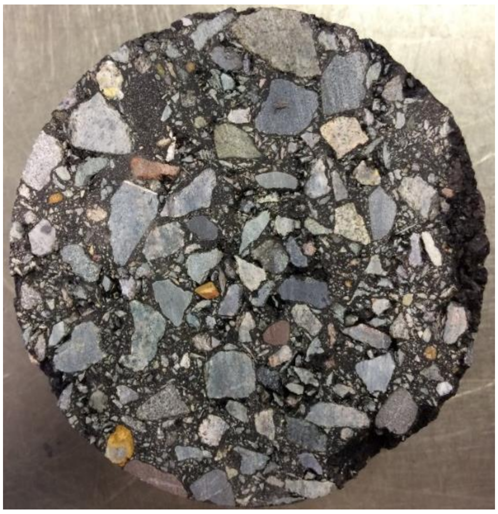
1. Weigh and batch virgin aggregate portions as determined by the mix design.
2. Use a riffle box to prepare a homogeneous RAP sample with the required weight.

3. Heat the virgin aggregate overnight at the required mixing temperature.
4. Heat the RAP and soft binder for 2-3 hours at the production temperature.
5. Mix the RAP and virgin aggregate only for 30 seconds. This step is essential to reduce the RAP binder film and to help in preventing RAP agglomeration.
6. Add the soft binder and mix for the specified mixing time.
7. Compact the asphalt at the required compaction temperature, then core and trim normally.

Following this procedure, HMA control specimens were mixed at 155°C and compacted at 145°C; whereas WMRA samples were mixed at three different temperatures, 135, 115, and 95°C, and compacted at 10°C less than the mixing temperature. At every mixing temperature, 1, 3, and 5 minutes of mixing time were used. This choice of mixing times and temperatures was made in order to understand the effect of these factors on the blending process, giving information that can be utilised in optimizing the production conditions of WMRA mix. Figure 5-6 (a, b, c and d) shows four WMRA samples fabricated in this way but under different production conditions.

It must be mentioned that no WMA additive was used in the manufacturing of WMRA samples. This is because of the grade of the virgin binder, which can be mixed at rather lower temperatures than the control mix without using and WMA additives. Furthermore, as concluded in chapter four, increasing mixing time improves aggregate coating when the mixing temperature is reduced. Also, some researchers demonstrated that the blending process between the soft and the aged binder is a function of mixing time, temperature, rejuvenator and aged binder properties (Zaumanis and Mallick, 2013a). Accordingly, to isolate the effects of the virgin binder and the production conditions on DoB from the effect of WMA additives, it was decided not to incorporate any additives at this stage of the study.

a



b

c

d

Figure 5-6. Images a and b are WMRA samples containing 50% RAP mixed at 95°C for 1 minute, images b and c are also WMRA samples containing 50% RAP but mixed at 135°C for 3 minutes

5.3 Volumetric Properties and Performance Measures

5.3.1 Air Voids

Figure 5-6 (a, b, c and d), which presents four samples mixed at different temperatures for different times, demonstrates visually that the homogeneity of the samples is significantly affected by the production conditions. Figure 5-6 (a and b) clearly show the agglomeration of RAP particles due to the lower production temperature and mixing time, whereas Figure 5-6 (c and d) show a well-mixed sample. Moreover, the air voids content and distribution are significantly affected by these factors. Thus, the air voids content was used to assess the volumetric properties. It was calculated by measuring the bulk density following British Standard BS EN 12697 (BSI, 2012a) and determined as the difference between the maximum density and the bulk density divided by the maximum density.

Figure 5-7 presents the results of the air voids content. The results show a clear relationship between air voids content and mixing time and temperature parameters. The higher the mixing temperature, the lower the air voids until it reaches the target value of 5%. The trend is complicated by mixing time. For example, mixing at 135°C for 3 minutes was sufficient to achieve the target air voids, whereas at 115°C, a longer mixing time was required, and the target air voids were barely obtained after 5 minutes of mixing. At 95°C, the target air voids were never achieved, even after 5 minutes of mixing. The different air voids under different production conditions may be explained by two factors.

The first is the phenomenon of particle clustering. Clustering can be defined as the agglomeration of RAP particles during mixing, which leads to changing the effective aggregate gradation and a decrease in the volume of free fine particles in the mix, thus affecting air voids content (Bressi et al., 2015). Increasing mixing temperature reduces RAP particle clustering by lowering RAP binder viscosity during mixing. The second factor is the actual size range of RAP aggregate particles and the amount of activated binder in the mix. At low production temperatures, less RAP binder is activated, and the remainder of the RAP binder is inactive, increasing the effective sizes of RAP particles. This changes the effective particle gradation from the design gradation and leads to a dry mix since the active binder content is less than the design binder content. As the production temperature increases, the viscosity of RAP binder decreases, and more active binder is introduced into the mix, filling the air voids, and reducing the effective sizes of RAP aggregate particles toward the design gradation. This leads to lower air voids in the mix and improved volumetric properties. These explanations are consistent with the air voids results in Figure 5-7.

To validate the statistical significance of these results, a T-test was performed, and a total number of 45 specimens (5 samples for every condition) were tested. Air void results for every parameter were verified against the air void results for the samples produced at 135°C and five minutes of mixing since target air voids were achieved under these conditions. Table 5-3 presents the results of the analysis. It can be seen that most of the results are statistically significantly different, apart from the samples produced at 3 minutes, mixing at 135°C or 5 minutes mixing at 115°C. This means that according to the air voids criteria, AC containing 50% RAP can be produced as low as 115°C if it is mixed for 5 minutes without compromising the air voids, whereas other production conditions may significantly affect the air voids content.

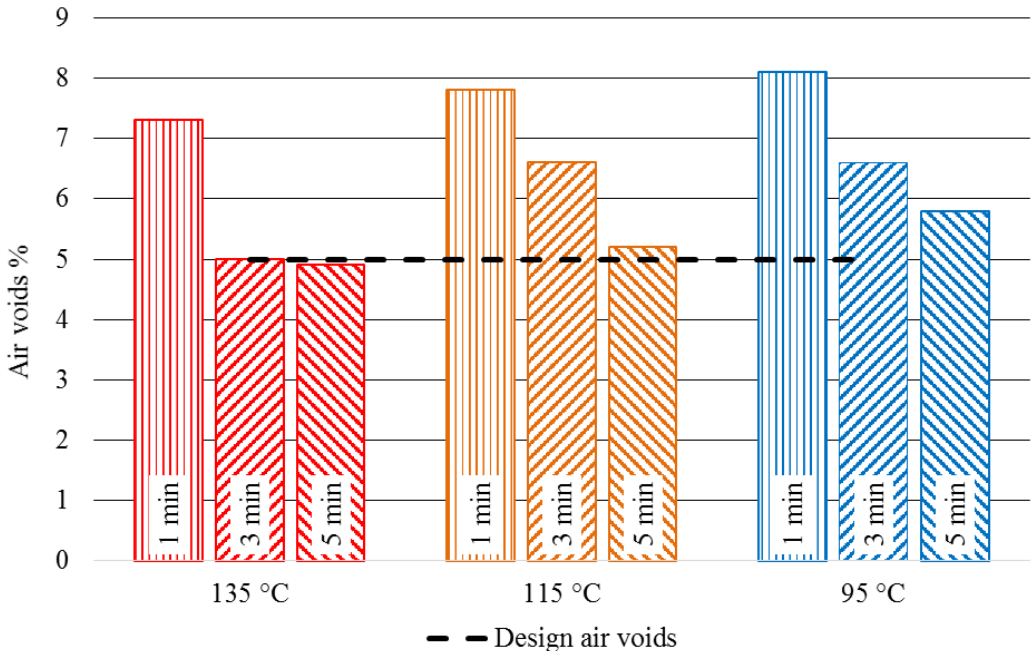


Figure 5-7. Air voids results

Table 5-3. Air voids Statistical analysis

Time min.	Mean AV%	Standard deviation	P value	Significant?
135°C				
1	7.3	0.6	0.0002	Yes
3	5.0	0.7	0.41	No
5	4.9	0.4	NA*	No
115°C				
1	7.8	0.8	0.0001	Yes
3	6.6	0.3	0.0001	Yes
5	5.2	0.4	0.207	No
95°C				
1	8.1	0.8	0.0001	Yes
3	6.6	0.7	0.001	Yes
5	5.8	0.5	0.01	Yes

* Not applicable

5.3.2 Indirect Tensile Stiffness Modulus (ITSM)

This measure was selected to evaluate specifically the performance of the asphalt samples containing RAP. In this test, 5 microns horizontal deformation is applied by exerting the required stress, which equates to 50 microstrains for 100 mm diameter samples. Since this is a small strain, it is mainly absorbed and resisted by the binder in the mix, which plays a significant role in preventing the movement of aggregate particles (Thom, 2014). In other words, this test is sensitive to binder stiffness because it is the parameter that resists the applied deformation during testing, and this point was confirmed in chapter two. Accordingly, it was implemented to study the effects of production conditions on the homogeneity and performance of WMRA mix.

The stiffness test results are presented in Figure 5-8. A similar but inverse trend to the air voids can be seen, with an improvement in the ITSM as the temperature or mixing time increases. Furthermore, mixing at 135°C for 5 minutes shows comparable performance to the control HMA. These results give information on the blending process between the soft binder and the RAP binder. In the case of poor binder blending, the rejuvenator binder dominates the sample response since it represents the binder surrounding particle contacts, whereas much of the RAP binder is still unblended. However, as the mixing temperature, mixing time or both increases, the ITSM values increase, which means the two binders are blending and the hard binder is stiffening the soft binder during the mixing process. The binder stiffening process can be explained by the forced diffusion of the soft binder into the aged binder. The diffusion process is affected by several critical factors, including mixing time, temperature, soft and aged binder properties (Zaumanis and Mallick, 2013a). Accordingly, the greater the mixing time and temperature, the higher the diffusion rate, which ultimately leads to stiffening of the soft binder and increased ITSM. Moreover, it can be concluded from the results that when mixing at 95°C, poor blending is achieved and the RAP binder is acting almost like black rock, even after 5 minutes of mixing. Mixing at 115°C shows partial participation of the RAP binder and the blending improves with mixing time. However, mixing at 135°C for three or five minutes clearly indicates proper blending between the two binders, which is reflected in the ITSM results.

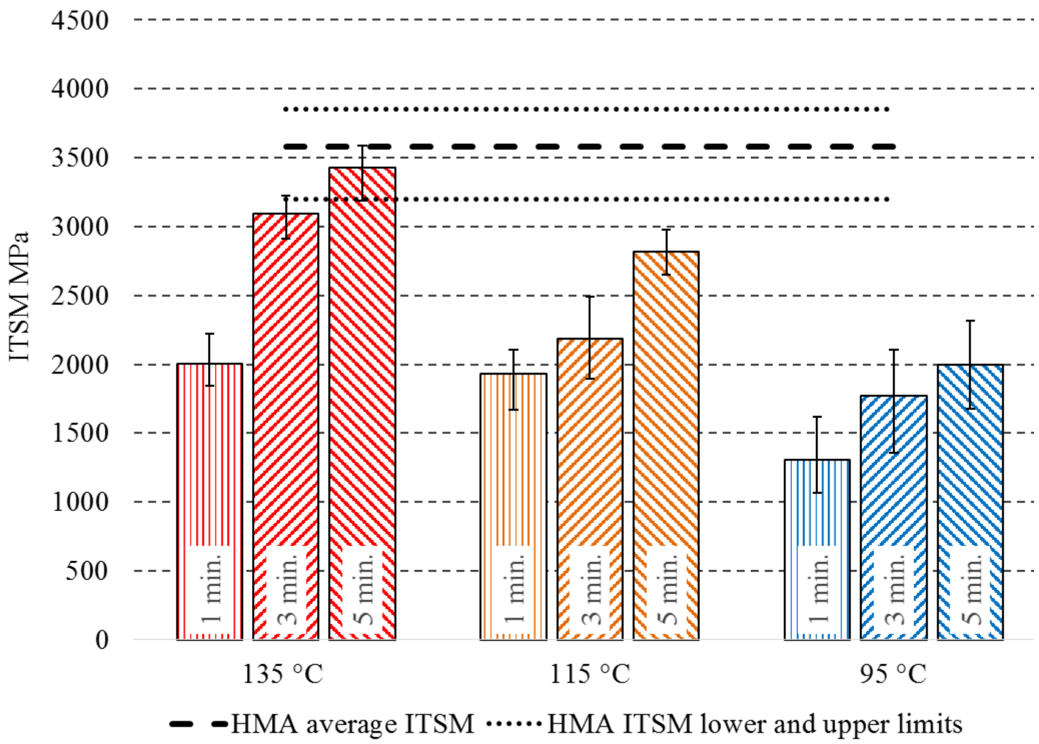


Figure 5-8. ITSM results

5.3.3 Relations between ITSM, DoB and Pen

Based on this evidence, stiffness measurements were used to quantify the DoB between RAP binder and the rejuvenator. The DoB is calculated as a percentage of the ITSM of the control mix using the following equation:

$$DoB\% = {ITSM_{AC-50\%\,RAP}}/{ITSM_{Control}} \times 100\% \qquad \text{Eq. 5-9}$$

where $DoB\%$ is the percentage of the degree of blending between RAP and soft binders, $ITSM_{AC-50\%\,RAP}$ is the stiffness modulus of WMRA samples, and $ITSM_{Control}$ is the stiffness of the control HMA. If the WMRA mix shows stiffness similar to the control mix, then the DoB is 100%; lower WMRA stiffness means less blending. DoB results are presented in Figure 5-9. It can be seen that 86-96% DoB was achieved when mixing for 3 or 5 minutes at 135°C, while 79% DoB was achieved when mixing at 115°C for 5 minutes. Other production conditions showed 60% or less DoB. However, these results were calculated based on the average ITSM results. To better analyse the DoB results, variability in ITSM results has to be considered. Figure 5-8 shows that all stiffness measurements of WMRA samples produced at 135°C when mixing for 5 minutes fall in the range of the control mix stiffness, suggesting no statistically significant difference between the two results. This means that these production conditions can be considered as optimised since they produce a mixture with statistically similar performance to the control HMA and the assumption of 100% RAP binder participation is therefore valid.

To further validate the results of this analysis, the relationship between ITSM at 20°C and penetration grade was explored. Five binder grades between 33 and 188 dmm penetration were used and AC samples fabricated with the same mix proportions and air voids content, and with virgin materials only. The ITSM values of these samples were then determined for at least three specimens per binder grade following the same procedure. The results are presented in Figure 5-10. A power law equation was found to represent the relationship between binder grade and ITSM. Statistically speaking, R^2 equals 0.92, which indicates a reasonable correlation. This relationship was then used to back-calculate and estimate the effective binder grade for the WMRA samples.

Figure 5-11 shows the results of these back-calculated binder grades. It can be seen that the effective penetration of the samples produced at 135°C for 3 or 5 minutes of mixing reached that expected after full blending since the back-calculated binder grade is about 49 dmm, which matches the target binder grade. Furthermore, mixing at 115°C for 5 minutes also came relatively close with 55.6 dmm. The results for back-calculated penetration grade provide a means of expressing the relative influence of rejuvenator and RAP binders, complementing the DoB values. The advantage of this approach is that it is possible to estimate binder grade and DoB without the need to extract and recover binder, which should be time effective and an economical approach in comparison with the traditional binder extraction and recovery methods (Rodenzo and Julian, 2018). Furthermore, the results presented in Figure 5-11 show that the back-calculated binder grades for all samples were between 47 and 106 dmm, whereas the soft binder penetration was 188, which means that even when mixing at 95°, part of the RAP binder is mobilized and participating in the mix.

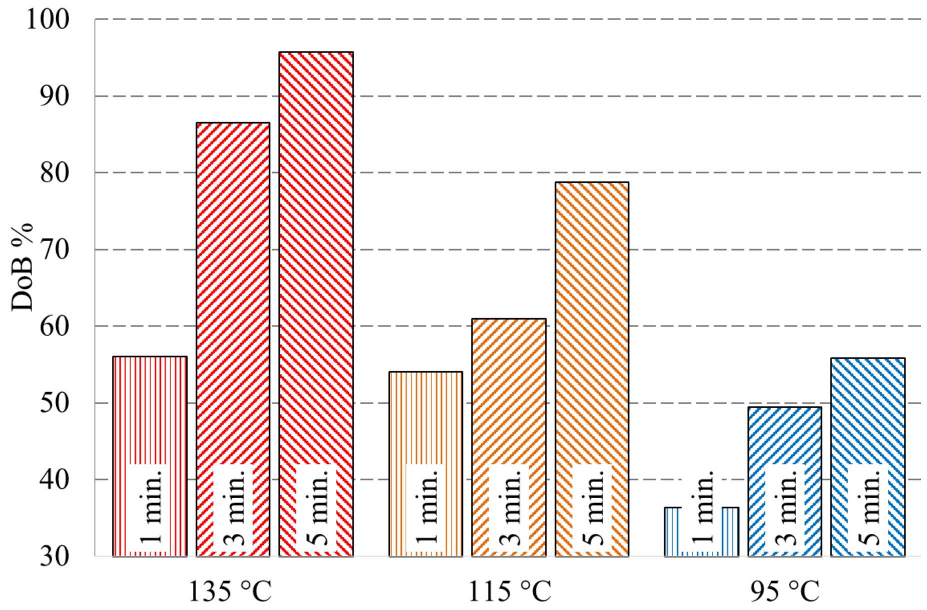


Figure 5-9. DoB calculation results

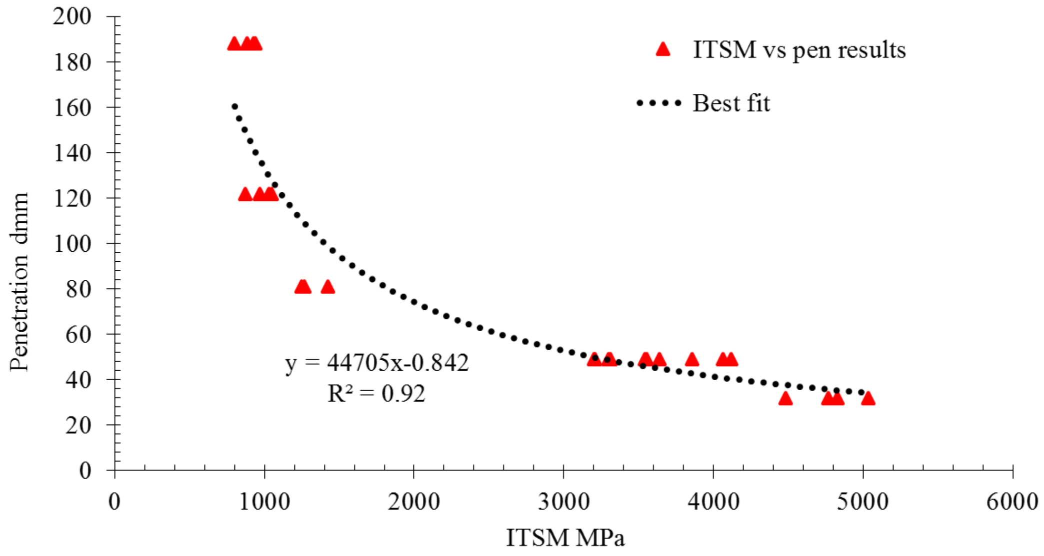


Figure 5-10. Penetration and ITSM relationship

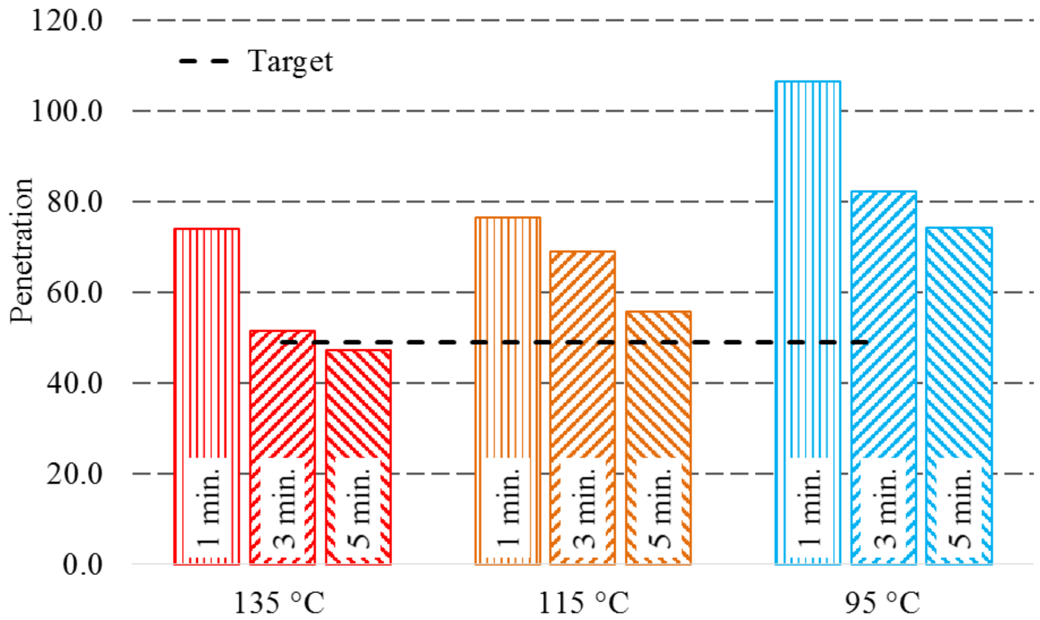


Figure 5-11. Back-calculated penetration

5.3.4 Indirect Tensile Strength (ITS)

The ITS test was performed in accordance with British Standard BS EN 12697-23 (BSI, 2003b) at a test temperature of 20°C. The ITS results are presented in Figure 5-12. The same trend can be identified; the higher the mixing time or temperature, the better the performance in terms of ITS. Mixing at 135°C for 5 minutes achieved the target ITS, which was the strength of the control mix, whereas mixing at 135°C for 3 minutes or at 115°C for 5 minutes resulted in 92% of the control ITS, approximately consistent with the ITSM results. However, as stated earlier, the ITSM test is more sensitive to binder than the ITS, because the low strain in the ITSM test occurs mainly in the binder, with only slight aggregate movement during load application. Also, considering the ITS in estimating DoB can lead to overestimation of this parameter. This evident in Figure 5-13, which demonstrates that the lowest DoB is about 60%, whereas the maximum can exceed 100%, which are unreasonable results. For this reason, the ITS results were not used to estimate DoB or back-calculate the blended binder grade but can be taken as an indicator of mixture performance to evaluate the fracture resistance of the WMRA mixtures.

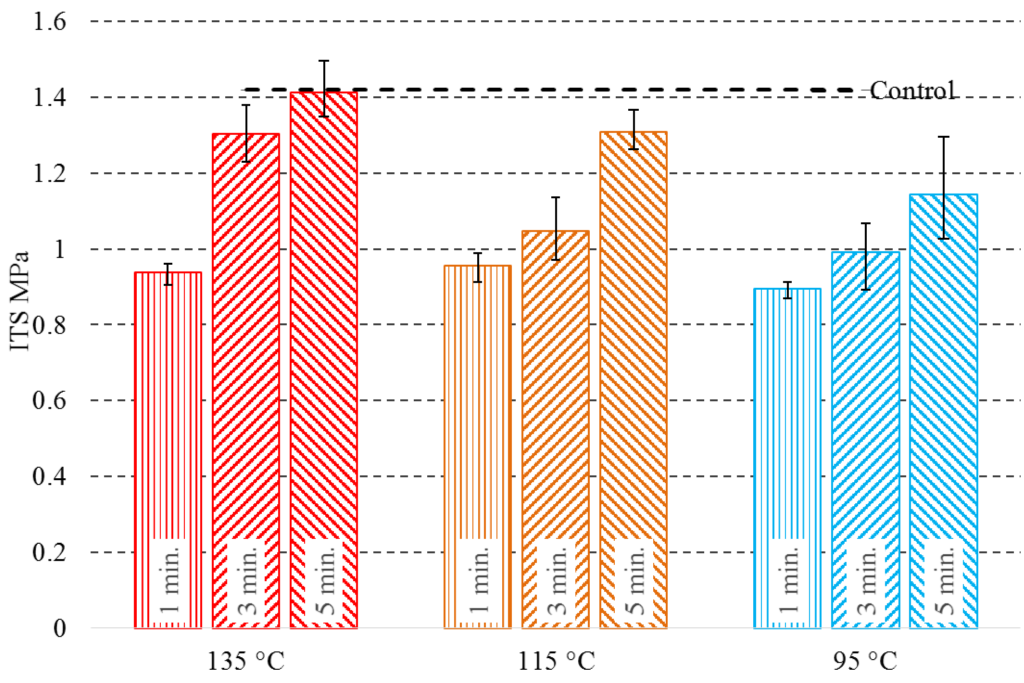


Figure 5-12. ITS results

5.4 Effect of WMA additives on DoB and Mix Performance

The previous sections discussed the DoB and its effect on the mechanical response of WMRA. The mechanism of DoB was investigated and analysed; it was understood that mixing temperature and time have a tremendous critical impact on DoB, which in turn influences mix volumetric properties and mechanical response. Hence, it is possible to study the possibility of whether WMA additives can improve DoB or not. If the additives can improve DoB, then that will be reflected on the DoB consequently mixture performance. Following this understanding, two WMRA mixtures were designed following the same design method but one containing Sasobit denoted by SWMRA, and the other containing Cecabase denoted by CWMRA. These mixes were manufactured at 135°C and mixed for five minutes. ITSM and ITS values of these mixes were then determined and compared to the control HMA and WMRA mixes, as shown in Figure 5-13 Figure 5-14, respectively. ITSM results show higher stiffness values for SWMRA than other mixtures; this result can be considered reasonable since this additive stiffness binder stiffness. Whereas CWMRA exhibited comparable ITSM to the control WMRA mixtures. The strength results, on the other hand, show generally comparable performance of all mixtures without any significant difference. These results show no clear evidence that the additives improved the blending. This means that the virgin binder grade is the main factor that impacts the blending with the aged binder in addition to the production conditions. This also shows the applicability to produced WMA with high RAP content by using a very soft binder as a rejuvenator. Hence, the primary role of the additives if incorporated is to enhance mix compactability as reported in the literature.

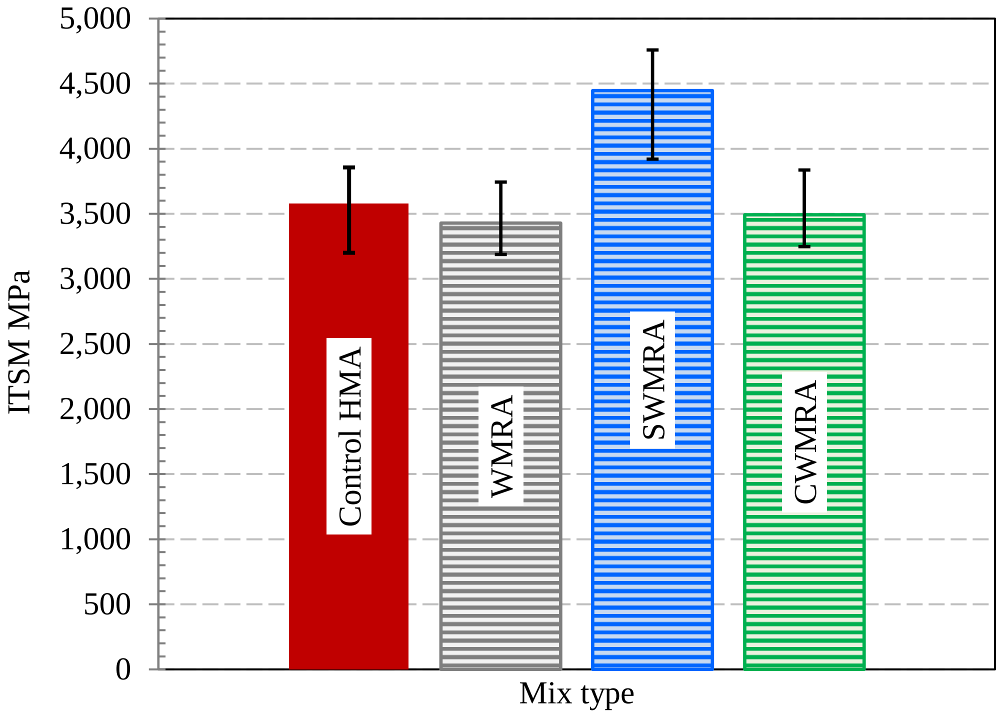


Figure 5-13. ITSM results of 50% RAP mixes produced using WMA additives

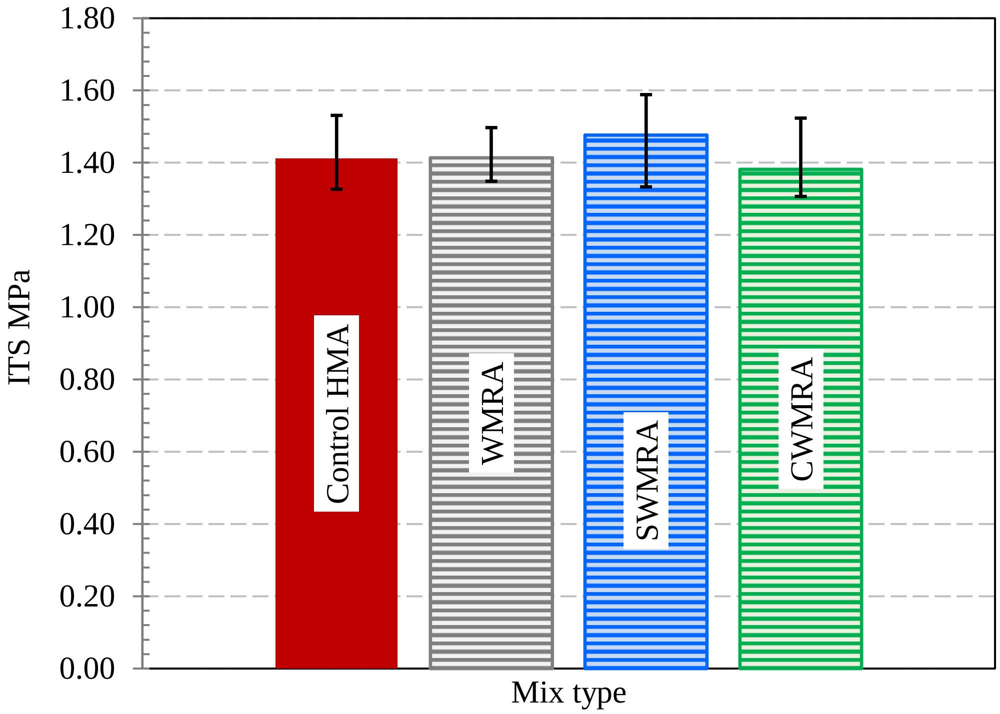


Figure 5-14. ITS results of 50% RAP mixes produced using WMA additives

5.5 Conclusions

This chapter investigated the design and analysis of asphalt mixtures containing a high percentage of RAP of 50% produced within the WMA production temperature range. The design stage involved characterisation of the RAP, which was used to design the ratios of the required virgin aggregate sizes and appropriate binder blending. Then the effect of the production conditions, mixing time and mixing temperature, was investigated. Performance of WMRA of several mixtures produced at different production conditions was evaluated by ITSM and ITS compared to the control HMA. A new analysis was then developed to understand and quantify DoB and to derive relationships between this factor and performance indicators. This approach is based on ITSM or ITS results and allows evaluating the DoB and prediction of the effective penetration grade of the recycled mixture binder without the need to extract and recover any binder. The results showed that the common assumption of full blending is rarely reasonable but could be practical if accurate design and processing conditions are used, even for asphalt mixes for surface courses incorporating high-content RAP and produced at WMA production temperatures. Based on the results of this chapter, the main conclusions can be summarised as follows:

1. WMA mixtures with high RAP content can be successfully produced with comparable stiffness and strength to traditional HMA if the production conditions are correctly designed.
2. The DoB was found to be a function of mixing time and temperature, and it can be estimated from stiffness or strength measurements. The author

believes that ITSM measurements provide higher accuracy since this characteristic has been proved to be more sensitive to binder stiffness.

3. The hypothesis of full blending is rarely acceptable with the common mixing conditions guaranteeing asphalt plant productivity; however, if these are optimised, it could be plausible even within the warm mix region. For instance, in this study, the ITSM results showed that full blending is a plausible assumption (DOB 96%) for WMRA containing 50% RAP mixed at 135°C only if five minutes were allowed for the mixing.

4. On the other hand, the black rock phenomenon is unlikely to happen in the warm region. In fact, results showed that even when mixing at temperatures as low as 95°C and for 1-minute mixing time, the RAP binder was still partially mobilized with the DoB for this condition being 36%. This is probably justified from the fact that at this temperature, only the outer layer of RAP binder became active, and there was insufficient heat energy to activate the entire thickness. However, increasing mixing time improved this value due to the increased mechanical contact when mixing for an extended period.

5. The effect of production conditions and virgin binder grade on DoB and performance is more pronounced than that of WMA additives. Incorporation of WMA additives showed no evidence that the additives can enhance blending between RAP and virgin binders. Hence, the role of the additives is to enhance mix workability and compactability.

Chapter 6: Evaluation of the Mechanical Performance of WMA and WMRA Mixes

6.1 Introduction

This chapter investigates primary performance indicators of the mixtures designed in this study in comparison to the control HMA. The selected indicators were permanent deformation studied at different high temperatures (≥30°), and fatigue cracking studied at different low to intermediate temperatures (≤20°C). Dynamic modulus test results are also reported and discussed in this chapter, and dynamic modulus master curves were constructed for all studied mixtures. Furthermore, the durability of WMA and WMRA was investigated, and the effects of long-term ageing and moisture damage on mix performance were analysed.

6.2 Materials and Sample Manufacturing

Based on the results presented in chapters four and five, HMA was mixed at 155 °C and compacted at 145°C, SWMA with 2% Sasobit was mixed and compacted at 135-125°C, CWMA with 0.4% Cecabase was mixed and compacted at 130-120°C, and WMRA with 50% RAP and 188 pen virgin binder grade was mixed and compacted at 135-125°C. The target air voids of all these mixtures was 5%, and the binder content was 5.1%. The compaction method was by a roller compactor, as explained earlier. After that the manufactured slabs were demoulded, cylindrical and trapezoidal samples were produced by coring and cutting to the required sizes.

6.3 Dynamic Modulus (E*)

6.3.1 Background

Asphalt is a viscoelastic material that exhibits time (or loading frequency)-dependent and temperature-dependent properties. Due to the viscoelastic nature of this material, a time lag appears between the applied stress and the resulting strain. In this case, the dynamic modulus can be defined as the stiffness of a material in the linear viscoelastic zone that is calculated by the division of the peak stress amplitude by the peak of the resulting strain amplitude (Al-Khateeb et al., 2006). The dynamic modulus is one of the essential fundamental properties of asphaltic materials used in the MEPDG to calculate pavement response under different temperatures and loading frequencies and predict pavement performance (Kim et al., 2011b). This modulus has been widely used in the US as a simple performance indicator. Reduced modulus values at low temperatures can be interpreted as giving improved fatigue cracking performance (Witczak, 2005), while increased asphalt modulus values at high temperatures can be lead to improved rutting resistance (Witczak, 2007).

The dynamic modulus can be determined by two different methods. Firstly, it can be predicted from mix design proportions by different prediction models including the law of mixtures (Al-Khateeb et al., 2006), or using mix volumetric properties and binder viscosity (NCHRP, 2004a). Secondly, it can be directly measured in the laboratory by performing dynamic modulus testing under different temperatures and loading frequencies. There are different test setups to run the test, such as in uniaxial mode, triaxial mode, indirect tension, and two point-bending (2PB) mode. The mathematical formulation behind these testing methods is as follows:

$$E^* = \frac{\sigma_0}{\varepsilon_0} = \frac{\sigma \times \sin(\omega \times t)}{\varepsilon \times \sin(\omega \times t - \delta)} \qquad \text{Eq. 6-1}$$

where σ_o, ε_o are the peak stress and strain amplitude in a sinusoidal dynamic modulus test, ω is the loading frequency, and δ is the phase angle between the stress and strain. To characterise material behaviour under different conditions, this test should be performed at different temperatures and loading frequencies. Then, these data can be processed by the application of the Time-Temperature Superposition Principle (TTSP) to form one continuous master curve at a reference temperature (Airey, 1997). By this principle, shift factors are used to shift modulus data measured at different temperatures and frequencies to form one continuous curve with a reference temperature. Consequently, the constructed curve exhibits a material response to a broader range of loading frequencies but at the reference temperature. In fact, one of the main benefits of TTSP is to analyse material behaviour at extended loading times and different temperatures from a few measurements.

Construction of master curves requires several steps. Firstly, |E*| data is collected at different temperatures and frequencies. Secondly, a reference temperature should be determined; the measured |E*| data at this temperature are not shifted. Thirdly, |E*| data at other temperatures are shifted horizontally by temperature shift factors (a_T) to form one continuous line aligned with the reference temperature. As a rule of thumb, the data for the temperatures lower than the reference one are shifted to the right whereas the data for higher temperatures are shifted to the left, as illustrated in Figure 6-1. Lastly, a mathematical function is fitted to the curve resulting from the shifting process. The most frequently used function is the sigmoidal model used in the MEPDG (NCHRP, 2004a), as follows:

$$\log(|E^*|) = \delta + \frac{\alpha}{1+e^{(\beta+\gamma\times \log f_r)}}$$ Eq. 6-2

where δ is the minimum |E*| value, α is the difference between maximum and minimum values of |E*|, β and γ are shape parameters, and f_r is the reduced frequency, which can be calculated as follows:

$$f_r = f \times a_T$$ Eq. 6-3

where f is the loading frequency, and a_T is the time-temperature shift factor. The shift factor determines the amount of shifting at each temperature; it can be used to express the material dependency on temperatures (Apeagyei et al., 2012). This parameter can be calculated as a function of temperature using some frequently used functions such as Williams Landel and Ferry (WLF), as follows:

$$\log(a_T) = \frac{-C_1\times(T-T_{ref})}{C2+(T-T_{ref})}$$ Eq. 6-4

where $c_1$and $c_2$are regression constants, and T_{ref} is the reference temperature.

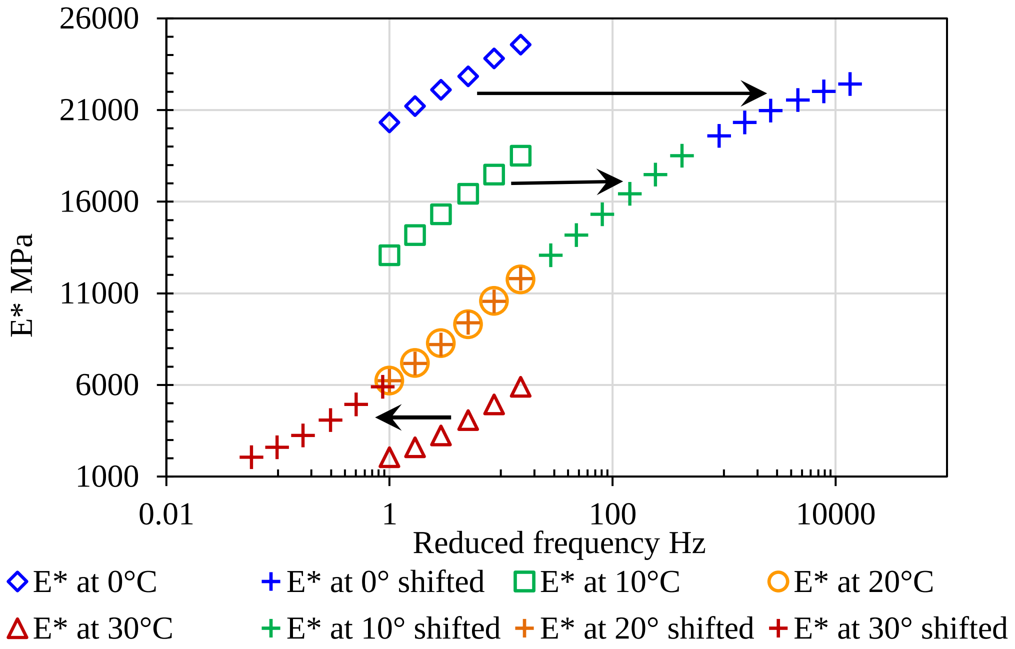


Figure 6-1. Construction of E* master curve; reference temperature is 20°C

6.3.2 E* Measurement by Two-Point Bending

In this study the dynamic modulus was measured directly by applying a sinusoidal strain with a maximum amplitude of 50 microstrains using a two-point bending machine in accordance with BS EN 12697-24 (BSI, 2004a) annex B. The testing setup is shown in Figure 6-2. This machine runs in control strain mode only and can control the temperature between 0 and 30°C. The test was performed at four temperatures 0, 10, 20, and 30°C and under six logarithmically spaced loading frequencies of 1, 1.7, 3.1, 5.2, 8.7, and 15 Hz; starting from the lowest temperature and highest frequency to avoid damaging the specimens during testing. Prior to each test, all samples were conditioned for at least four hours at the specified testing temperature in the machine cabinet before running the test.

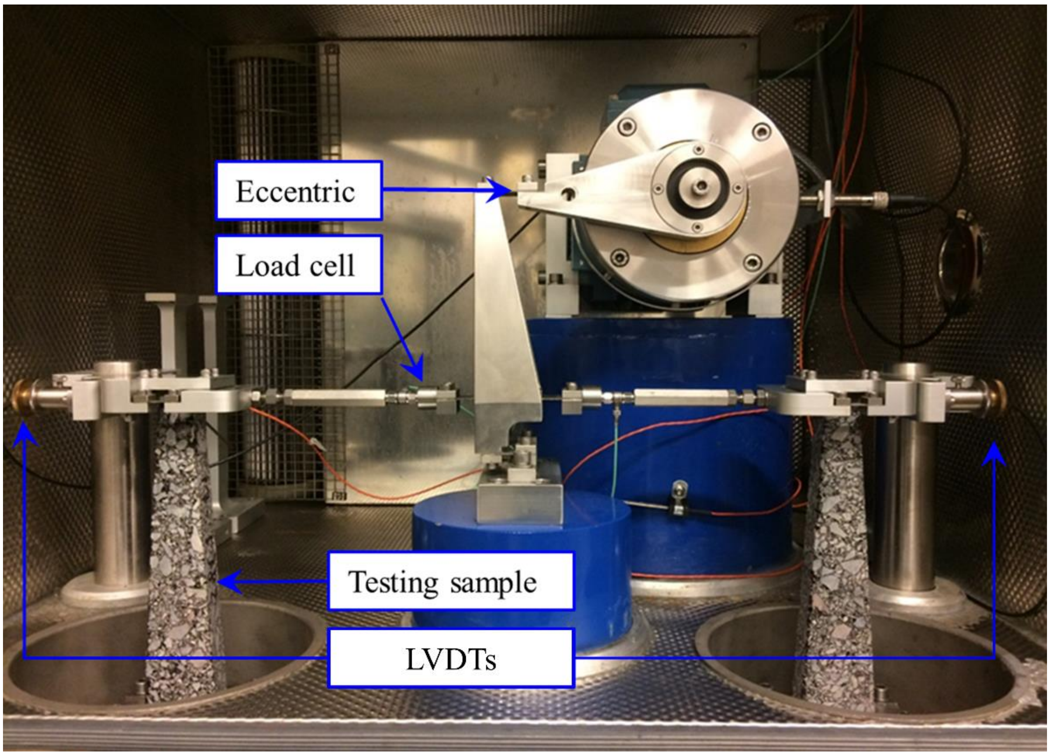


Figure 6-2. Two-point bending machine

6.3.3 Results and Discussion

Following the prescribed testing setup, E* of the control HMA, SWMA, CWMA and WMRA mixtures was determined. Then the sigmoidal function presented in Eq. 6-2 was fitted to the raw data. The optimisation function, Solver, embedded in Excel software, was utilised to calculate model parameters. The E* master curve and raw data of each mix are shown in Figures 6-3 to 6-6; Figure 6-7 presents a comparative chart of E* master curves of all studied mixes. This figure can be used to understand material response at different loading conditions and predict asphalt performance concerning rutting and fatigue cracking.

At low temperatures (high frequencies), it seems that SWMA has a higher dynamic stiffness than other mixtures, which is expected due to the stiffening effect of this material. However, this result can be interpreted as an increased low-temperature cracking potential due to the high stiffness of this mix at low temperatures. Other mixes seem to perform similarly at low temperatures. At intermediate temperatures, the same behaviour is encountered except that WMRA has a distinctly lower stiffness than other mixes, which could be an indicator of improved fatigue cracking performance. At high temperatures, SWMA is expected to perform better than other mixtures regarding permanent deformation resistance. HMA and CWMA are expected to perform comparably since they exhibit almost the same characteristics at high temperatures, whereas WMRA has more rutting potential that other mixes.

Moreover, the impact of WMA additives and RAP on the elasticity of asphalt can be assessed by the phase angle master curves, as shown in Figure 6-8. It can be seen that at low temperatures, the phase angles almost coincide into one curve with no significant differences. However, at intermediate and high temperatures, SWMA exhibits distinctly lower phase angles than other mixes, indicating an increased elasticity of this mix. Other mixes do not show apparent differences in the phase angles. Accordingly, it cannot be concluded that the presence of Cecabase or RAP has altered the elasticity of asphalt. However, this analysis is based on asphalt response results within the linear viscoelastic zone. The binder testing results in chapter three showed the necessity to characterise materials beyond the limit of linear viscoelasticity to better understand their behaviour.

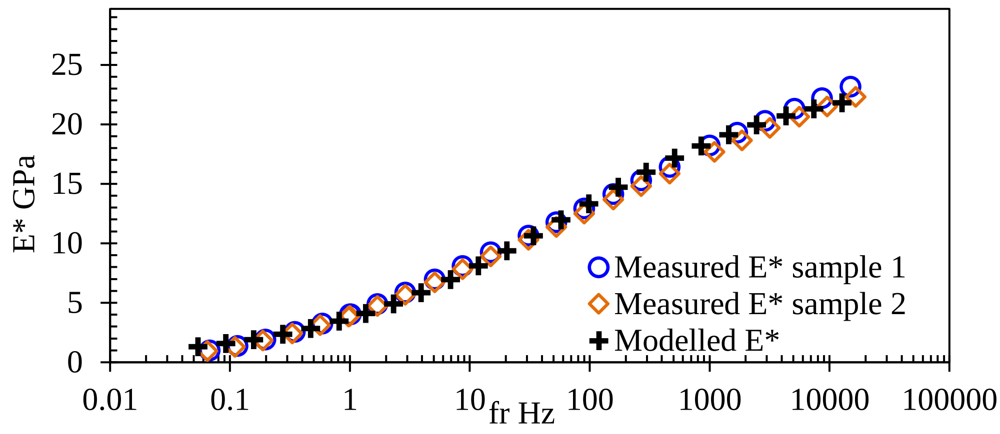


Figure 6-3. E* master curve of HMA

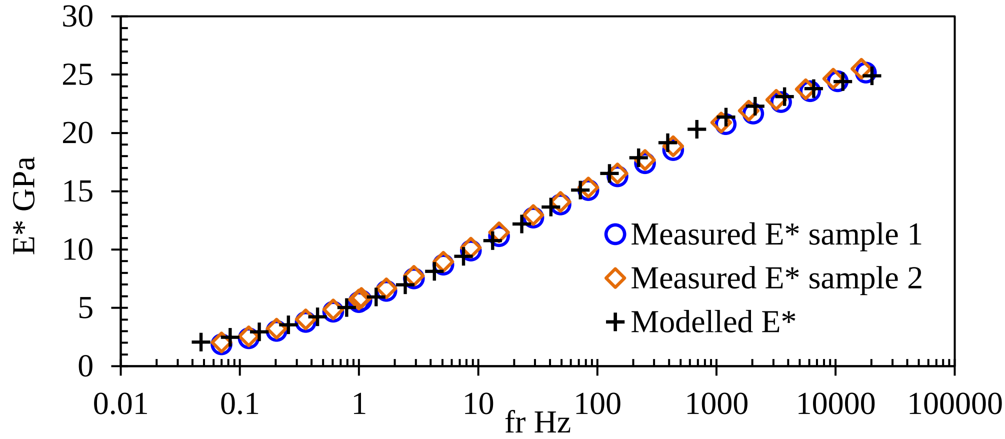


Figure 6-4. E* master curve of SWMA

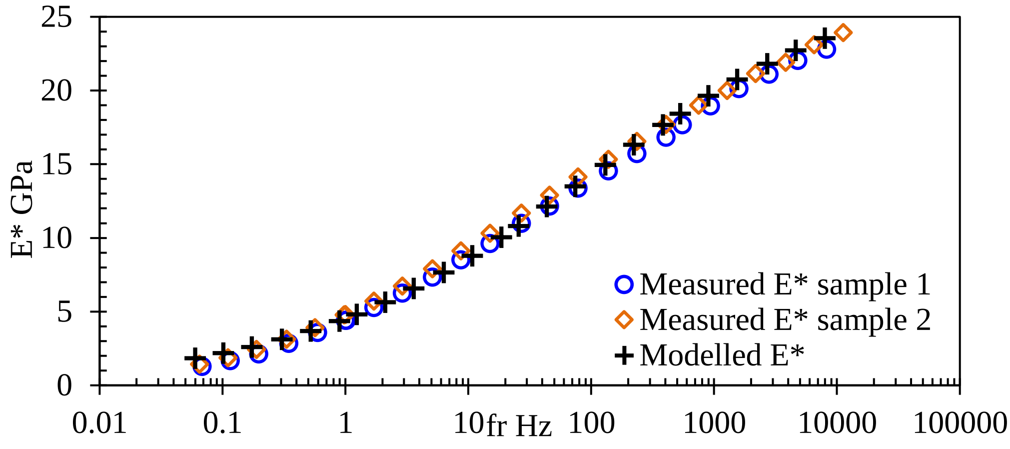


Figure 6-5. E* master curve of CWMA

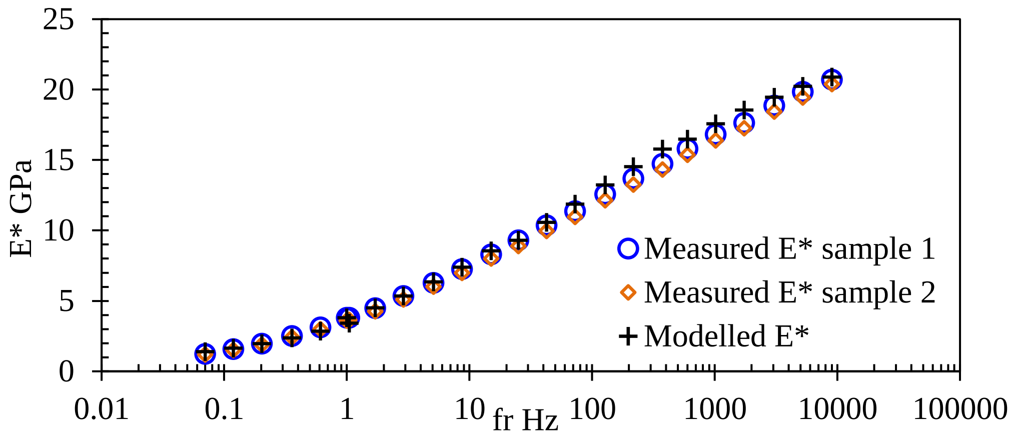


Figure 6-6. E* master curve of WMRA

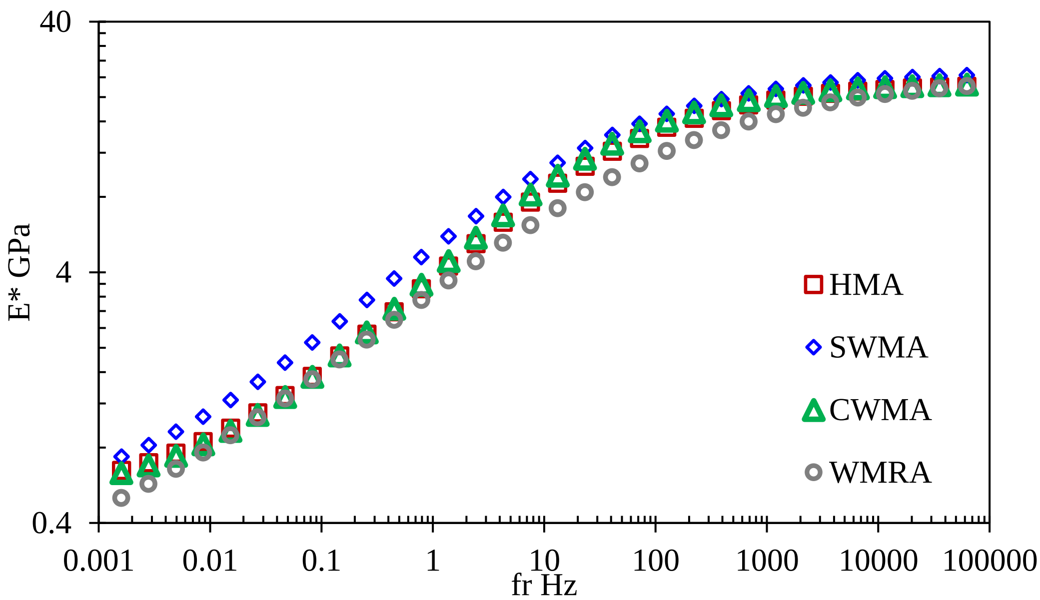


Figure 6-7. E* master curves of all mixes

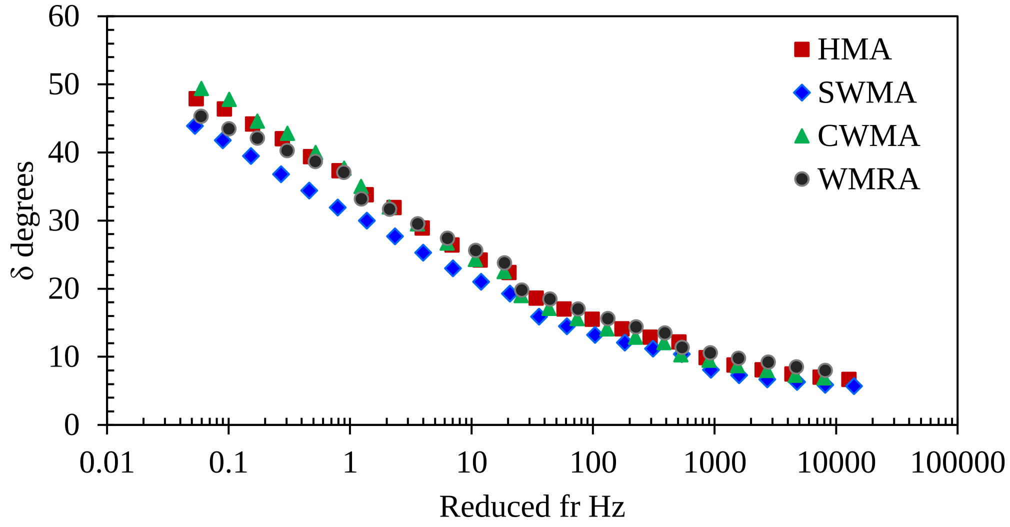


Figure 6-8. Phase angle master curves of the studied mixes

6.4 Resistance to Permanent Deformation

6.4.1 Background

Asphalt pavements exhibit a plastic strain under the effects of traffic loading and environmental conditions. Accumulation of permanent strains over time leads to the formation of rutting distress, which is one of the major failure modes of flexible pavements. Asphalt can exhibit permanent deformations due to two mechanisms, consolidation or densification of asphalt, and permanent flow of asphalt under the effect of the shear stress or as a result of inadequate shear strength of asphalt (NCHRP, 2004b). Consequently, this distress can be affected by several factors including, aggregate skeleton, air voids content, binder properties, traffic load, volume, and speed.

In this study, permanent deformation susceptibility of the studied mixtures was evaluated in the laboratory, so most of the factors that have an impact on rutting were kept under control. Consequently, any change in material performance should be due to the effect of the studied variables, which in this case were the presence of WMA additives, RAP, and mixing temperatures. In this way, the effect of other variables can be kept constant, and impacts of the studied variables on permanent deformation performance can be investigated and analysed.

6.4.2 Permanent Deformation Evaluation by Repeated Load Axial Test

In this study, permanent deformation performance was evaluated by conducting unconfined the Repeated Load Axial Test (RLAT) in accordance with British Standard DD226 (BSI, 1996) using a NAT machine, as illustrated in Figure 6-9. In this test, a square wave of compression stress is applied to a test specimen sandwiched between standard steel plates at a specific constant temperature. The applied load form and resulting material response are illustrated in Figure 6-10. The total load cycle time is two seconds, one second of loading followed by one second of a rest period. During the loading cycle, the material exhibits an instant strain due to the elastic element of asphalt, and a delayed strain due to the viscous element of asphalt. During the rest period cycle, an elastic recovery takes place as soon as the load is removed, followed by a delayed recovery due to the viscous part of asphalt. The remaining strain at the end of the rest cycle is a plastic strain and represents the permanent damage that the material has undergone due to the applied load. The accumulation rate of the plastic strain over the number of load applications is a key factor in evaluating material performance regarding rutting. The cumulative plastic axial strain in an RLAT test can be calculated as follows:

$$\varepsilon_n = \frac{h_o - h_n}{h_o} \qquad \text{Eq. 6-5}$$

where ε_n is the cumulative axial strain after n load applications, h_o is the initial height of the specimen, and h_n is the height after n load applications.

In the UK, the RLAT is performed at a testing temperature of 30°C, and 1800 load applications. This is because the highest air temperature in this country is generally less than 30°C. At this temperature, most of the paving grade binders may perform similarly because bitumen stiffness is still high. However, considering different climatic conditions, and for the sake of examining the investigated mixture performance under diverse environmental conditions, the RLAT was conducted at three high temperatures of 40, 50, and 60°C. The number of load applications was doubled to 3600. Other testing conditions were 100 kPa axial stress, 10 minutes of sample conditioning at 10 kPa, and at least at four hours of specimen conditioning at the required testing temperature prior to testing.

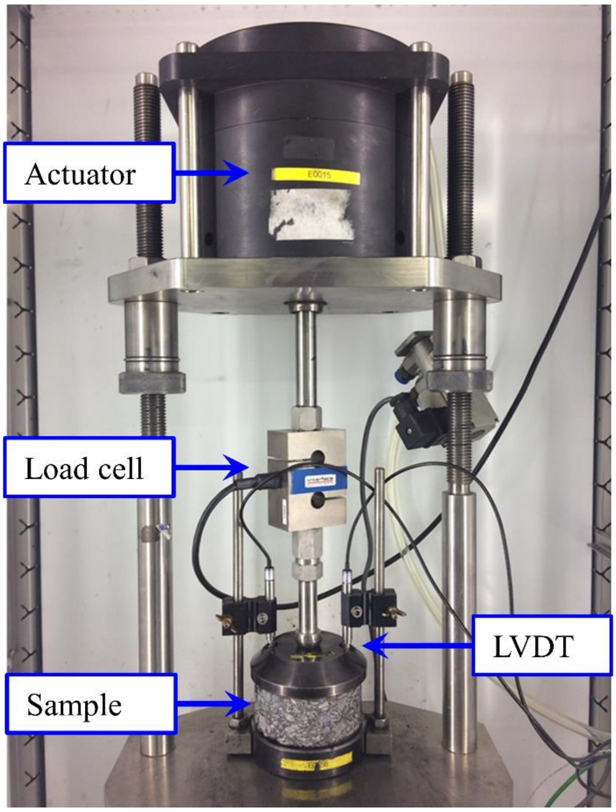


Figure 6-9. RLAT setup using NAT machine

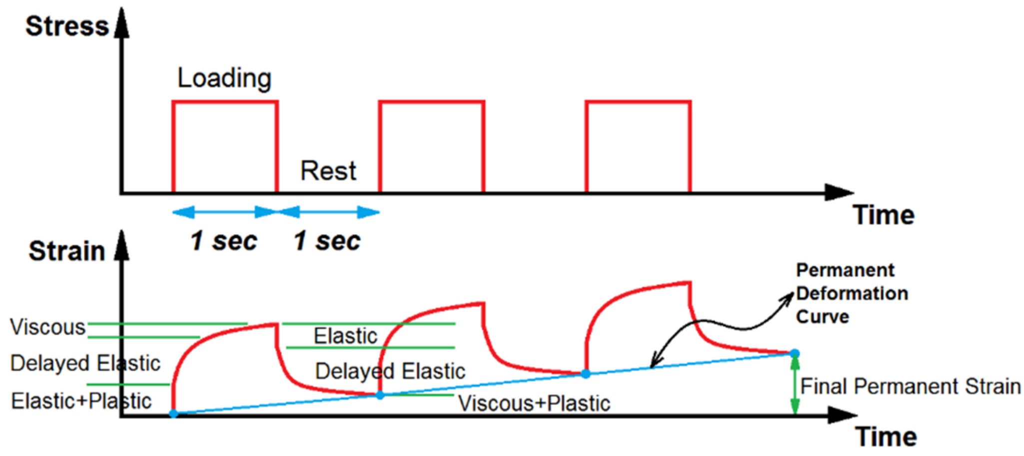


Figure 6-10. Load form and strain accumulation during RLAT (Jalali, 2016)

6.4.3 Results and Discussion

Figure 6-11 presents typical RLAT results at a test temperature of 60°C. In this figure, the axial strain is plotted as a function of the number of load applications. This allows detection of the phases that the material exhibits during the test. The first distinct phase is the rapid increase in the axial strain at the beginning of the test, which happens due to the densification and reorientation of aggregate particles under the effect of the applied load. The second phase represents a near-constant increase in the axial strain. This phase represents the steady-state response of the material, and the macro axial strain of this phase is mainly due to the micro-shear and compressive strains that the material undergoes during the test. The third phase or the tertiary stage appears when the material exhibits rapid axial strains after the steady-state stage. This phase indicates that the material is about to completely collapse under loading. None of the mixtures in Figure 6-11 reached the tertiary phase. However, the WMRA exhibited higher axial strain than other mixes, which means this material may reach the tertiary phase soon.

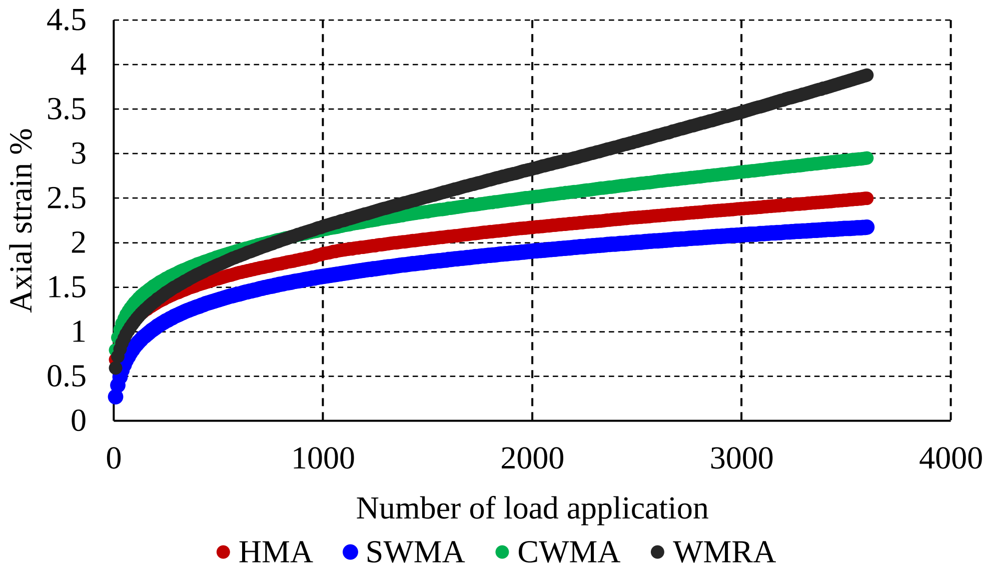


Figure 6-11. Typical RLAT results (at 60°C)

Figure 6-12, Figure 6-13, and Figure 6-14 present RLAT results of all mixtures at testing temperatures of 40, 50 and 60°C respectively. At 40°C, SWMA and CWMA performed comparably to HMA with very slightly higher strain than the control mix. However, WMRA showed distinctly lower axial strain than other mixes. At a test temperature of 50°C, bitumen stiffness becomes lower than at 40°C, and more difference in RLAT results can be detected, as shown in Figure 6-13. This figure shows that the bitumen stiffening effect of Sasobit is reflected in RLAT results as SWMA exhibited lower axial strain than HMA. On the other hand, CWMA showed relatively large plastic strain, about 28% higher than HMA. This result can be attributed to the effect of Cecabase on bitumen rheology and shows that this additive reduces bitumen shear modulus. Furthermore, these results are quite consistent with the MSCR results reported in chapter three. Moreover, WMRA also showed a clearly higher strain than other mixes. Despite Figure 6-13 indicating that the average RLAT result for WMRA is similar to CWMA, the former mix had higher variability in the results than the latter mix. This indicates that WMRA has more rutting potential than CWMA. The same trend of results can generally be spotted when analysing RLAT data at 60°C. But at this temperature, WMRA exhibited distinctly higher axial strain with much larger variability in the results than all other mixtures.

To explain the above observations, RLAT results at all testing temperatures were grouped in one chart, as shown in Figure 6-15. This chart shows that SWMA performed better than HMA when the test temperature increased from 40 to 60°C. The chart also shows that CWMA performed worse than HMA, especially at high temperatures. These results were attributed to the effect of WMA additives on bitumen rheology. However, Figure 6-15 indicates that WMRA performance at 40°C was better than HMA, whereas its performance at 50 and 60°C was worse. This result can be explained by the blending quality between the soft and RAP binders. The DoB of this mix was 96% based on ITSM results, as presented in chapter five, which means there is a chance that a small amount of the soft binder was still not fully blended in the mix. Even if the amount is as little as 4%, this small amount can represent points of weakness in the mix matrix, and these weak points are where the damage starts leading to damaging of the entire mix. Furthermore, to explain the improved RLAT performance of WMRA at 40°C, the Superpave rutting parameter $G^*/\sin\delta$ of the soft binder was considered in the analysis. The soft binder has a critical high temperature of 43.7°C, which is the temperature at which the binder exhibits $G^*/\sin\delta$ equal to 1kPa when tested unaged, as shown in Figure 6-16. This means that up to 40°C, this binder may not exhibit high axial strains since this temperature is below its critical high temperature. However, when the temperature reaches 50 or 60°C, the stiffness of the binder becomes very low, and at this state, this binder exhibits extremely high strains which cause large plastic deformations in the mix. This interpretation explains the substantial permanent deformation results of WMRA at 50 and 60°C.

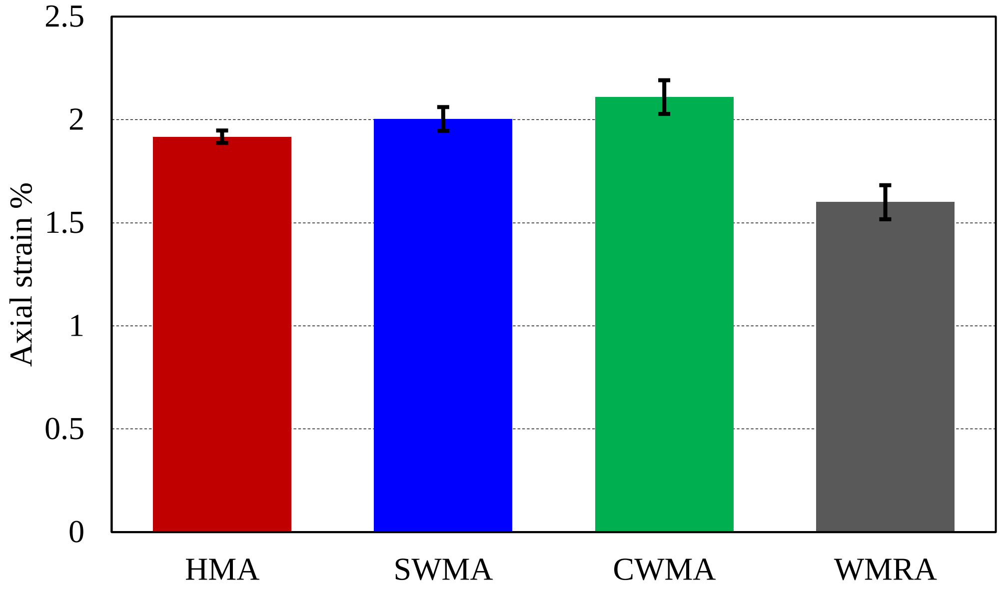


Figure 6-12. RLAT results at 40°C and 3600 load application (error bars represent the minimum and maximum values)

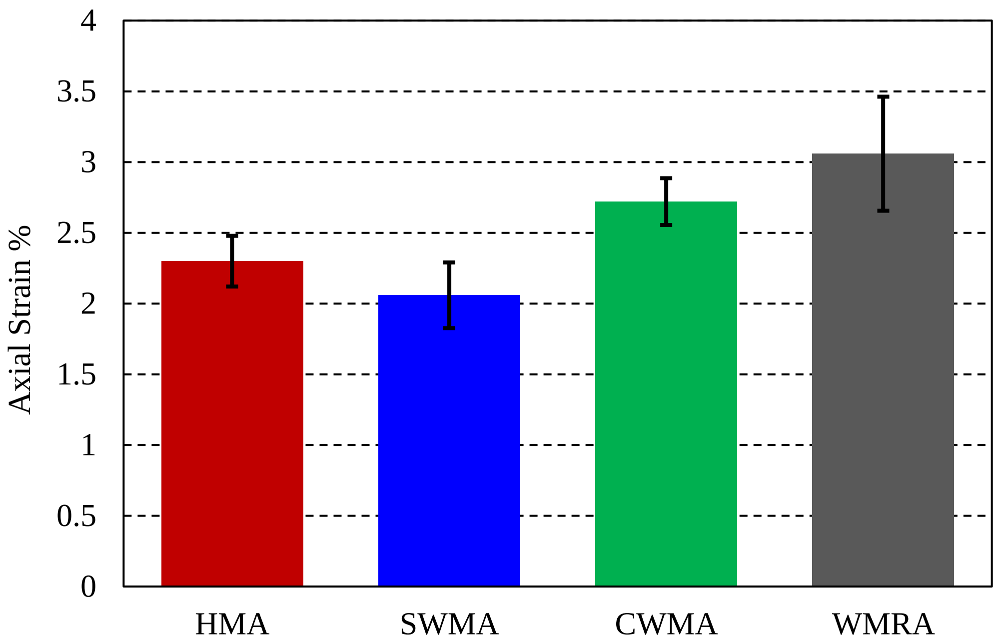


Figure 6-13. RLAT results at 50°C and 3600 load application

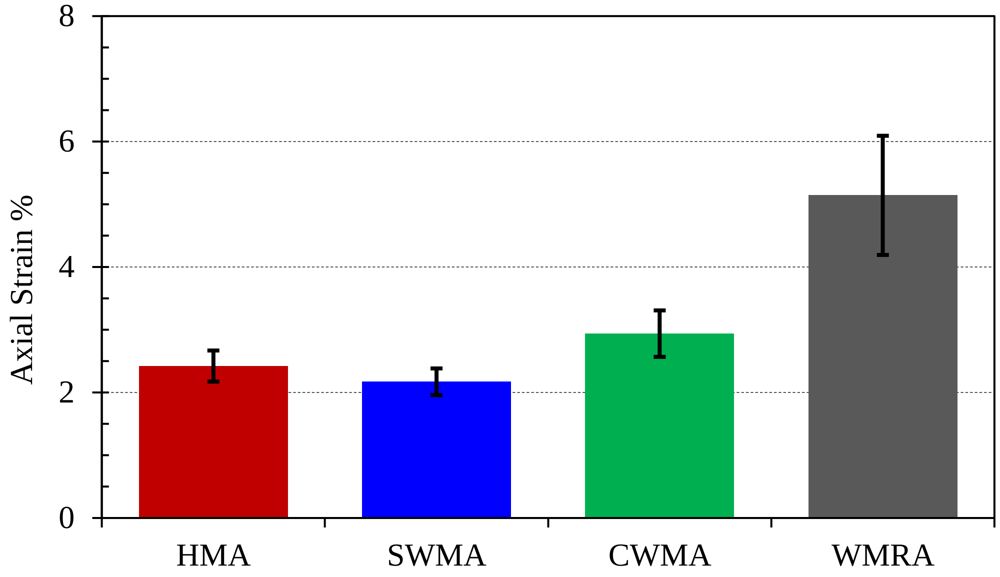


Figure 6-14. RLAT results at 60°C and 3600 load application

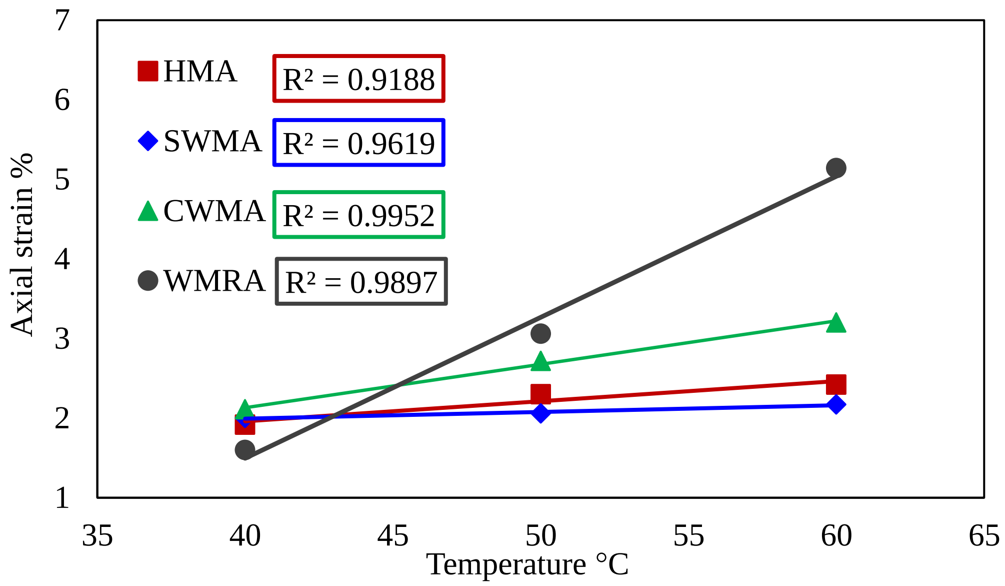


Figure 6-15. RLAT results of all mixtures at 40, 50, and 60°C

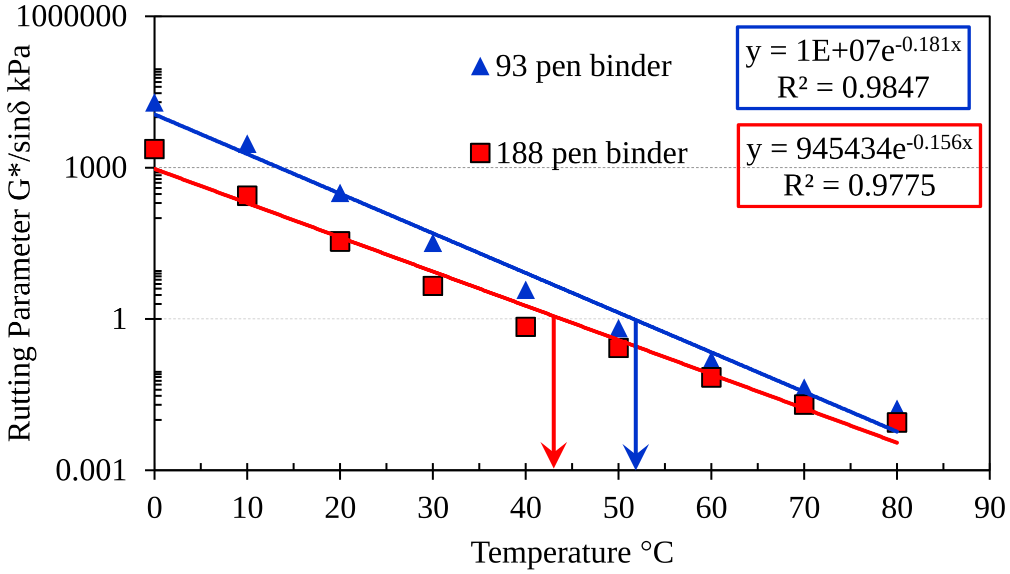


Figure 6-16. Superpave rutting parameters of 188 and 933 dmm pen binders

Despite WMRA performing better than HMA at 40°C and its performance at 50°C still being considered acceptable since the total axial strain was about 3%, an attempt was conducted in this study to improve WMRA performance concerning permanent deformation. Based on the understanding of the nature of the permanent deformation problem discussed in the previous paragraphs, two methods were followed to reduce the rutting potential of WMRA. First, two WMRA mixtures were produced using Sasobit and Cecabase under the same mix design and production conditions as WMRA without additives. The assumption of incorporating these additives was to validate whether they can improve the binder blending, consequently reducing rutting potential. However, this time, the effect of the additives was evaluated based on the RLAT test rather than ITSM and ITS since this test seems to discriminate excellently between mixes. The second method was to incorporate harder virgin binder than the one calculated based on the blending chart. The concept behind this method was to use a sufficiently soft binder grade to be used as a rejuvenator, but which at the same time can better resist the expected stresses. Moreover, in some European countries such as Germany, Poland and Slovenia, the softest binder grade allowed as RAP binder rejuvenator is 70/100 (Brosseaud, 2010). This limit seems to be quite reasonable to avoid excessive plastic strain problems in the cases of incomplete binder blending. Accordingly, four mixtures were considered, one WMRA with Sasobit and 188 dmm penetration virgin binder (denoted by SWMRA-188), one WMRA with Cecabase and 188 dmm binder (CWMRA-188), one was WMRA with Sasobit and 93 dmm binder (SWMRA-93) and one WMRA with Cecabase and 93 dmm binder (CWMRA-93). These mixtures were designed by the same design method followed in chapter five with the same binder content and volumetric properties. However, these mixes were tested at 50 and 60°C only since WMRA already performed excellently at 40°C.

Figure 6-17 and Figure 6-18 present RLAT results at 50 and 60°C, respectively. These results show that the incorporation of Sasobit significantly reduced the axial strain of mixtures containing RAP. At 50°C, the plastic strain of WMRA decreased from 3.1% to 2.3% (SWMRA-188). The same trend can be observed at 60°C; the axial strain decreased from 5.1% to 3.5%. It seems that the incorporation of Sasobit can alleviate the negative effect of remaining unblended soft binder, and there are two mechanisms that can justify this performance improvement. Firstly, by the stiffening effect of this additive; Sasobit stiffened the soft binder and increased its complex shear modulus, which led to reduced axial strain. Secondly, Sasobit might have improved the blending quality by reducing the aged binder viscosity during asphalt mixing, which leads to additional stiffening of the soft binder and eventually reduced permanent deformation. Cecabase, on the other hand, seems to impact RLAT results insignificantly. At 50°C, the average axial strain of CWMRA-188 was marginally higher than that of WMRA, whereas it was slightly lower at a test temperature of 60°C. This result indicates that Cecabase might have improved binder blending, but it does not provide conclusive evidence that this material can improve DoB.

On the other hand, RLAT results of the mixtures produced using the 93 dmm pen binder are also shown in Figure 6-17 and Figure 6-18. These figures demonstrate that using the 93 dmm pen binder as a rejuvenator has significantly reduced the amount of the plastic strain in the WMRA mixtures. At a testing temperature of 50°C, both WMRA mixtures produced using Sasobit and Cecabase outperformed HMA with less plastic deformation in the case of using Sasobit due to the stiffening effect of this additive. This result coincides with the critical high temperature of the 93 dmm pen binder, which was 51.4°C, as shown in Figure 6-16. At a testing temperature of 60°C, SWMRA-93 performed comparably to HMA and CWMRA-93 exhibited slightly increased axial strain. However, both of these mixtures should outperform HMA at 60°C since the theoretical resulting binder grade from blending the RAP and 93 pen binders should be 36 dmm whereas HMA binder grade was 49 dmm. This means that if the 100% blending case was achieved, then SWMRA-93 and CWMRA-93 should outperform HMA when tested at 60°C.

The above results provide conclusive evidence that permanent deformation of WMRA is significantly affected by the characteristics of the binders used in the mix, and the grade of the soft binder plays a vital role in shaping the performance of the final product. The results also indicate that careful mix design proportions and selection of appropriate rejuvenator grade are critical factors to assure production of recycled asphalt mixtures with acceptable performance. Furthermore, these results indicate that the RLAT is a reliable test to examine and evaluate permanent deformation performance and bitumen blending quality when a soft binder grade is used as a rejuvenator in mixtures containing high RAP content.

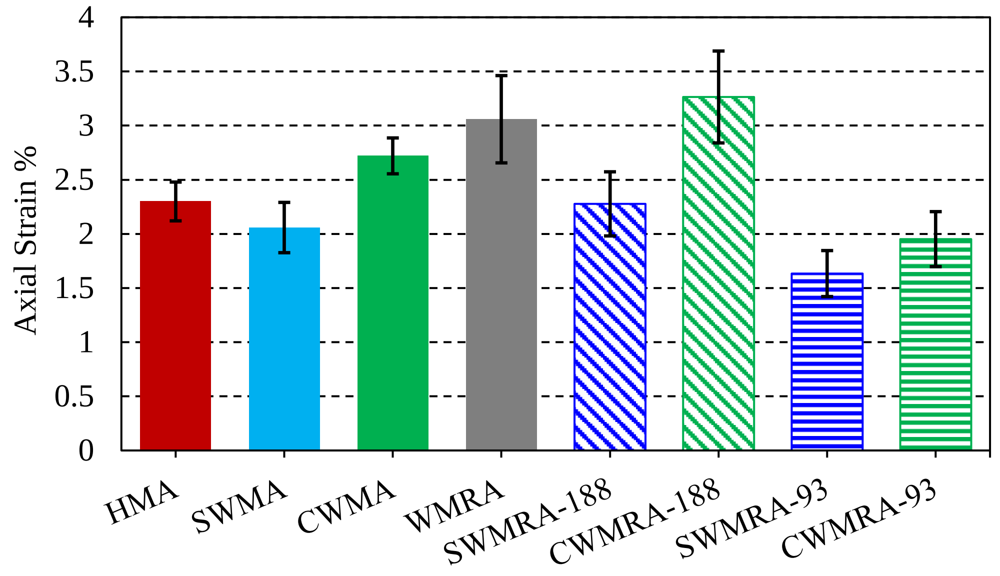


Figure 6-17. RLAT results at 50°C

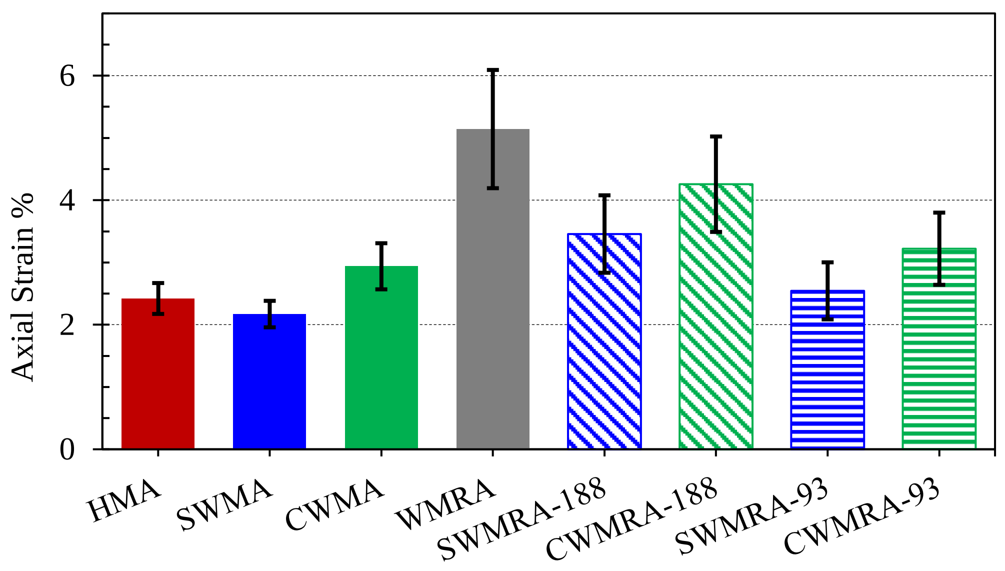


Figure 6-18. RLAT results at 60°C

6.5 Resistance to Fatigue Cracking

6.5.1 Background

Fatigue cracking is the process of cracking of asphalt due to tensile strains induced by traffic loading and environmental changes (Read, 1996). It is one of the primary failure modes of asphalt roads that leads to degradation of pavement layers (Di Benedetto et al., 2004). It is also one of the significant factors that causes degradation of serviceability and structural capacity of asphalt pavements. Fatigue cracking is a complex phenomenon that can appear due to several factors such as insufficient asphalt stiffness, poor pavement structure, improper compaction, or weak resistance of asphalt to the initiation and propagation of cracks. Resistance to this distress is a crucial factor to improve pavement performance and minimise pavement maintenance.

Fatigue performance of HMA has been extensively studied in the literature, and different methods have been developed to research fatigue behaviour of asphalt. However, fatigue performance of WMA and WMRA is still an ambiguous area, as more contributing factors are involved such as the reduction of production temperatures which reduces binder ageing during asphalt mixing, incorporation of WMA additives which changes bitumen rheological properties, and the presence of RAP which leads to the presence of an aged and brittle binder that may make asphalt more susceptible to cracking. Understanding the effects of these factors can lead to a better design against that kind of failure in WMA and WMRA mixtures.

6.5.2 Fatigue Cracking by Two-Point Bending

Due to the particular importance of fatigue cracking failure of asphalt, it has been extensively investigated, and different testing configurations have been developed in the literature. These include uniaxial tension-compression, indirect tension, two, three, and four-point bending tests, or even more complex loading forms such as torsion and torsion with deformation (Di Benedetto et al., 2004, Thom, 2014), Figure 6-19 presents some of the currently used fatigue testing methods. Each one of these configurations have different properties such as the direction and waveform type of the applied load, test mode, and specimen geometry; each has advantages and disadvantages. The indirect tension mode causes permanent deformation during testing and underestimates fatigue life; field stress and strain conditions are not adequately simulated in the axial loading configuration; the bending methods are usually not cost-effective, they are time-consuming and require specific equipment (Read, 1996). Therefore, ranking of fatigue life of the same material could be different from one test to another.

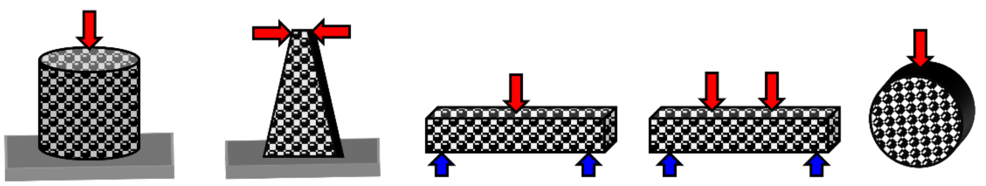


Figure 6-19. Asphalt fatigue cracking testing configurations

In this study, the 2-point bending configuration was adopted. This test has some advantages, such as that the fatigue life determined from this test can be used in pavement design using specific shift factors (Read, 1996). During this test, a sinusoidal deformation of a certain magnitude is applied to the top of the sample, which concentrates the strain at approximately one-third of the cantilever beam height, as illustrated in Figure 6-20. The required stress to achieve the applied deformation is also recorded by load cells. As a result of the applied loading, microcracks are formed within the sample, and the effect of these cracks on the sample response can be realised by a reduction in the dynamic modulus. A typical way to present fatigue damage is to plot normalised E* values against the number of load applications, as shown in Figure 6-20. This figure shows that the fatigue damage curve passes through four distinct stages, as follows (Di Benedetto et al., 1996, Rowe and Bouldin, 2000)

I. Immediate sharp reduction in the modulus that has been attributed to the internal heating of the sample due to sinusoidal loading. The increased temperature during this stage in addition to the formation of microcracks may be the reasons behind this sharp reduction in the modulus.

II. Steady-state reduction in the modulus. This stage appears after the sample has reached a thermal equilibrium, which makes the reduction in the modulus purely due to the formation of microcracks.

III. Distinct reduction in the modulus mainly due to joining of the microcracks and formation of larger cracks.

IV. Specimen “breakdown” (or its close approach).

Accordingly, to adequately characterise material behaviour, the fatigue test should continue until the material’s modulus reduces to about 20% of its initial value. In this case, all of the four stages can be captured and analysed.

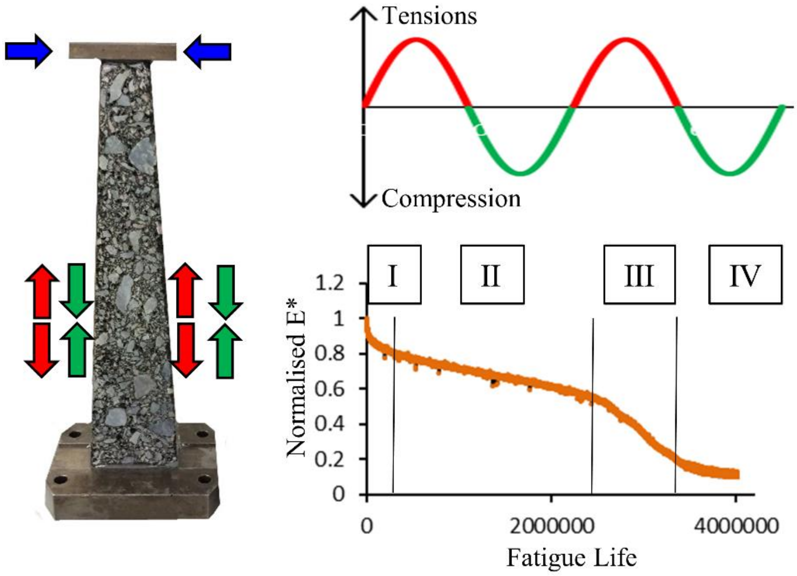


Figure 6-20. Two-point bending typical loading and material response

6.5.3 Fatigue Cracking Testing Conditions

In this study, the fatigue cracking test was conducted using the 2-point bending machine presented in Figure 2-2 in accordance with BS EN 12697-24 Annex A (BSI, 2004a). The testing conditions were as follows:

1. Testing temperatures were 0, 10, 20°C (some mixtures were only tested at 20°C due to raw material (RAP) and machine availability).

2. Controlled-strain mode with four strain levels between 80-320 microstrain (με) equally spaced on a log scale. These strain levels were target values; the actual applied strain levels during testing were close to these values. This is because the strain can be applied by mechanically adjusting the eccentric (Figure 6-2) to the required deformation rather than electronically entering this value. Also, the strain significantly depends on the actual dimensions of each sample, which cannot be exactly the same due to the

cutting variation from one sample to another. This makes the strain level vary slightly from one test to another.

3. Loading frequency of 15 Hz.

4. Two samples were tested at each strain level

5. The tests were stopped when the modulus reached about 20-30% of its initial value since the modulus of some samples did not drop below 30%.

Table 6-1 shows the mixtures considered in the two-point bending fatigue test. Mixtures SWMRA-93 and CWMRA-93 were considered in the fatigue cracking evaluation due to the improved permanent deformation performance of these mixes.

Table 6-1. Asphalt Mixture type and testing temperatures

Mix	Testing Temperature °C			Frequency Hz	Strain level με
HMA	0	10	20	15	80 - 320
SWMA	✓	✓	✓	✓	✓
CWMA	✓	✓	✓	✓	✓
WMRA	✓	✓	✓	✓	✓
SWMRA-93			✓	✓	✓
CWMRA-93			✓	✓	✓

6.5.4 Fatigue Cracking Analysis Methods

Different methods to characterise fatigue life are given in the literature. Some of these methods are valid for controlled stress mode and some for controlled strain mode. Some methods, such as the ones related to the dissipated energy, have been reported to be valid for both modes of loading. In this study, four methods were applied to analyse fatigue test data, also to investigate the validity of these methods to characterise fatigue life of WMA and WMRA mixtures.

6.5.4.1 Phenomenological Analysis

The phenomenological approach is one of the widely applied methods to determine fatigue life of asphalt. In this approach, the failure is defined as the number of load applications causing 50% reduction in the stiffness of the material (Tayebali et al., 1992); i.e. until the load required to achieve the applied strain has dropped to 50% of its initial values (Hunter and Read, 2015). Figure 6-21 presents an example of fatigue life determination using this method.

6.5.4.2 Normalised Stiffness Analysis

This is a standard analysis method to determine the fatigue life of asphalt samples subjected to sinusoidal flexural bending load (Astm, 2010). In this analysis, the stiffness is normalised to the initial stiffness using the following equation:

$$E_n = E_i \times N_i / (E_0 \times N_0) \qquad \text{Eq. 6-6}$$

where E_n is the normalised stiffness, E_i, E_0 are the modulus at the ith load cycle and the initial modulus respectively, and N_i, N_0 are the ith cycle and the number of load applications applied to measure the initial modulus. When plotting the E_n against the number of load applications, the failure point can be determined as the peak of the resulting curve, as illustrated in Figure 6-21.

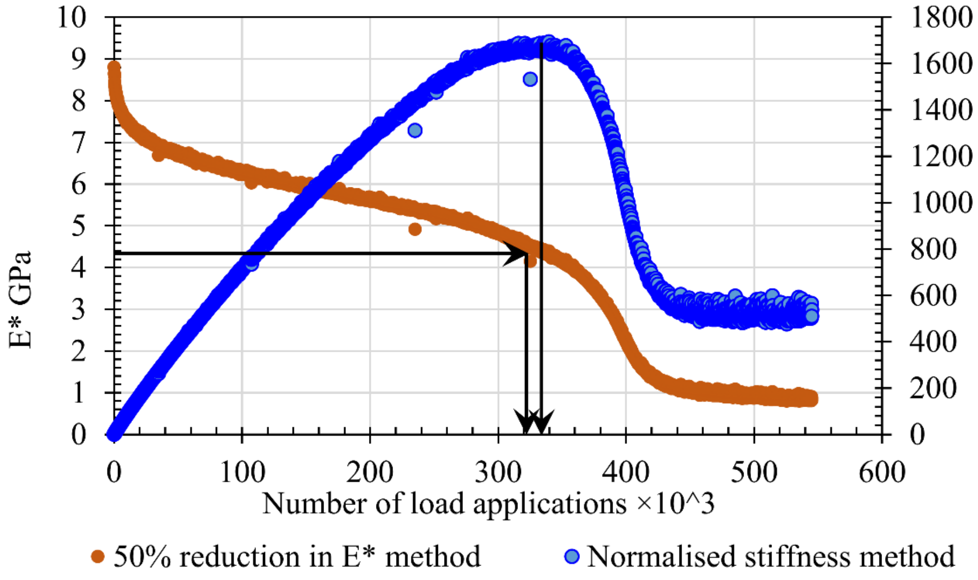


Figure 6-21. Fatigue life determination by modulus reduction and normalisation methods

6.5.4.3 Cumulative Dissipated Energy (CDE)

The dissipated energy approach has been widely used in different studies to characterise fatigue life of asphalt (Van Dijk and Visser, 1977, Hopman et al., 1989, Rowe, 1996, Ghuzlan and Carpenter, 2000, Shen et al., 2011, Maggiore et al., 2014, Subhy et al., 2017). These studies have demonstrated that this approach can be successfully used to characterise fatigue life of asphalt independently of the mode of loading. The concept behind this approach is that during the loading phase in a fatigue test, the loading energy is stored in the material, during the unloading phase some of the stored energy is dissipated, and some of it is lost due to fatigue damage. The concept of dissipated energy is graphically presented in Figure 6-22. The dissipated energy is defined as the area inside the hysteresis loop and can be calculated as follows:

$$W_i = \pi \times \sigma_i \times \varepsilon_i \times sin\delta_i \quad \text{Eq. 6-7}$$

where $W_i, \sigma_i, \varepsilon_i, \delta_i$ are the dissipated energy, stress, strain, and phase angle in cycle i, respectively. The dissipated energy relationship includes all the fundamental rheological properties of viscoelastic materials, which means that fatigue analysis based on this approach should be fundamental and independent of the mode of loading. In a fatigue test, if the applied stress or strain does not damage the material, then the stress-strain relationship follows the path of the intact material shown in Figure 6-22. However, if the applied loading or deformation damages the material then the rheological properties of the sample change due to damage, and the damage can be recognised by a reduction in the modulus and an increase in the phase angle (Masad et al., 2008), as shown in Figure 6-22. In this case, the material's ability to dissipate energy decreases proportionally with the fatigue damage level in the material. This mechanism continues until the material approaches failure at which the ability of the material to dissipate energy significantly drops, as shown in Figure 6-23.

Based on the definition of the dissipated energy, it has been suggested to use CDE to failure to determine fatigue life (Hopman et al., 1989, Tayebali et al., 1992), as follows:

$$CDE_n = \sum_{i=1}^{n} W_n \qquad \text{Eq. 6-8}$$

$$CDE_f = A \times (N_f)^Z \qquad \text{Eq. 6-9}$$

where CDE_n is the cumulative dissipated energy to the n^{th} cycle, CDE_f is the cumulative dissipated energy to failure, A and Z are constants, and N_f is the number of load applications to failure. N_f can be graphically determined by plotting CDE against N, the point at which the CDE deviates from a fitted straight line being defined as N_f. Figure 6-24 shows an example of determining the number of applications to failure by this method.

6.5.4.4 Energy Ratio (ER)

This method was suggested by Hopman et al. (1989) to determine fatigue life, which was defined as the number of load applications until the formation of a sharp crack. ER can be calculated as follows:

$$ER_n = n \times W_o / W_n \quad \text{Eq. 6-10}$$

where $\mathrm{ER_n}$ is the energy ratio at the $\mathrm{n^{th}}$ cycle, n is the cycle number, $\mathrm{W_o}$ is the dissipated energy at the first cycle, and $\mathrm{W_n}$ is the dissipated energy at the $\mathrm{n^{th}}$ cycle. Inserting Eq. 6-7 into Eq. 6-10 leads to the following:

$$\mathrm{ER_n} = \mathrm{n} \times \pi \times \sigma_o \times \varepsilon_o \times sin\delta_o / (\pi \times \sigma_i \times \varepsilon_i \times sin\delta_i) \quad \text{Eq. 6-11}$$

Since this test was conducted in a constant strain mode, then this equation can be rewritten as follows:

$$\mathrm{ER_n} = \mathrm{n} \times \frac{\mathrm{E_o}}{\mathrm{E_i}} \times \frac{sin\delta_o}{sin\delta_i} \quad \text{Eq. 6-12}$$

$\mathrm{E_o}$ is a constant material property, which means it is not going to change the shape of the ER curve when plotted against the number of load applications; also the change in phase angle during the test is marginal in comparison with the modulus (Rowe, 1996). This means that equation Eq. 6-12 can be rewritten as follows:

$$\mathrm{ER_n} = {}^{\mathrm{n}}/_{\mathrm{E_i}} \quad \text{Eq. 6-13}$$

When plotting ER against the number of load applications, the failure zone can be determined as the point of the sudden increase of ER, as illustrated in Figure 6-24.

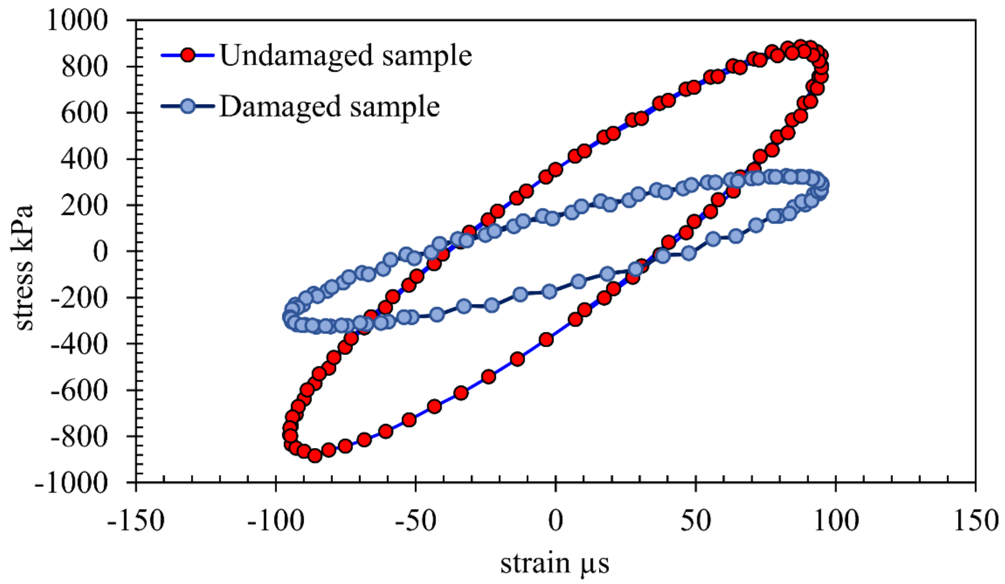


Figure 6-22. Hysteresis loops of an asphalt sample before and after fatigue damage

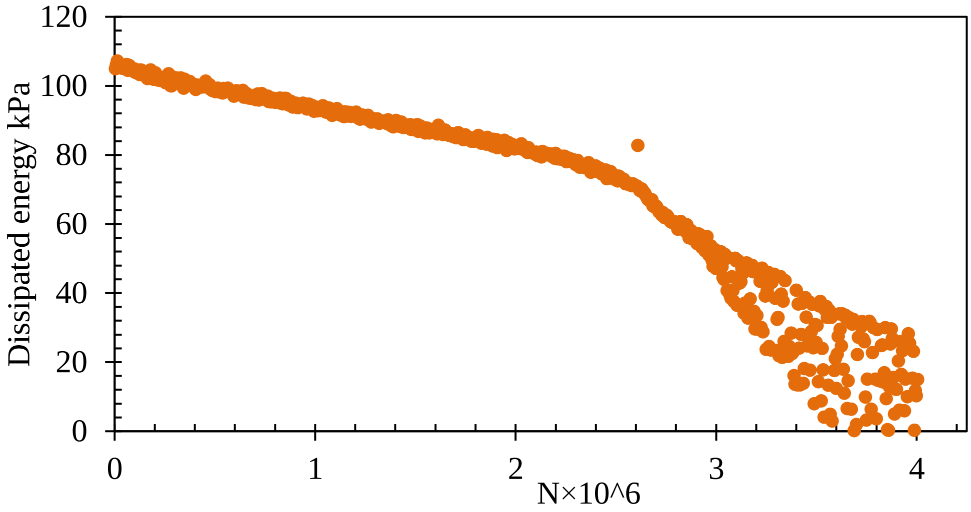


Figure 6-23. Dissipated energy reduction due to fatigue damage

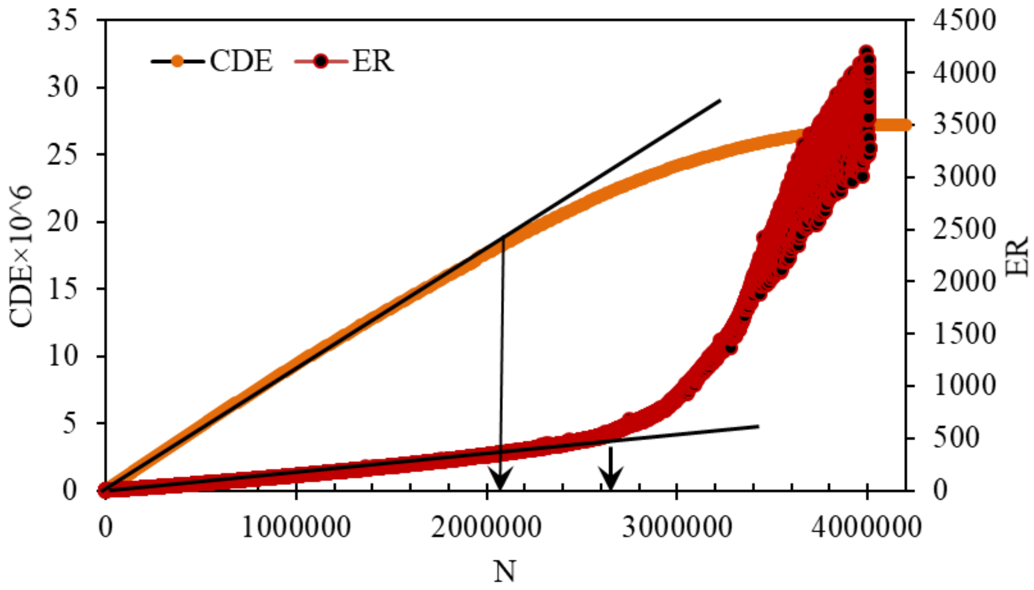


Figure 6-24. Failure definition of CDE and ER methods

6.5.4.5 Discussion on Fatigue Analysis Methods

It was previously explained that if there is sufficient fatigue data, the typical fatigue curve's four stages can be easily identified. Since in this study, the data were collected down to about 20% reduction in E*, then identifying the stage where a sharp crack is formatted was possible. Accordingly, the method that calculates the closest number of applications to the fatigue life of sharp crack formation can be considered accurate and can be selected to analyse fatigue data.

Following this understanding, the results of fatigue life determined based on the four methods were plotted on the fatigue curve of each studied mix, as shown in Figures 6-25 to 6-30. It can be seen that the 50% reduction in E* method captures the start of the stage of sharp crack formation accurately. The ER method can also capture this stage, but in about 90% of the fatigue data, this resulted in slightly lower fatigue life than the 50% reduction method. The En and CDE approaches typically resulted in a fatigue life lower than other methods with CDE being the lowest of all in several results.

Accordingly, it can be concluded that the 50% reduction method, despite the criticism found in the literature on this method, is still reliable and can accurately determine fatigue life up to the stage of sharp crack formation. Therefore, it was adopted in this study and used to construct fatigue lines of the studied mixtures.

6.5.5 Fatigue Cracking Performance Analysis

6.5.5.1 Analysis of Fatigue Cracking at 20°C and 100 με

Figures 6-25- to 6-30 present fatigue degradation curves at a test temperature of 20°C and a strain level of 100 με. The results show that reducing the production temperatures of asphalt alongside the presence of WMA additives without or with RAP can alter fatigue life significantly. Also, these factors can alter the failure mechanism of the sample as well. For example, the presence of Sasobit reduced fatigue life by about 23% in comparison with HMA. But it also altered the rate of the sharp crack formation. This is evident in Figure 6-26 by the sharp drop in the modulus after it reached 50% of its initial value. In contrast, the rate of the sharp crack formation in the case of HMA was much slower as can be seen in Figure 6-25. On the other hand, Cecabase reduced fatigue life to about 40% lower than the control mix. The presence of this additive seems to be affecting the fatigue damage rate as the CWMA showed the lowest fatigue life and smallest steady-state modulus reduction amongst all the mixtures. This result might be due to the effect of the additive on bitumen rheology; this additive reduced G* of bitumen, but it also reduced creep recovery of the bitumen which may lead to reduced damage healing capability of asphalt mixtures containing Cecabase.

Despite the presence of RAP making asphalt more susceptible to fatigue cracking (Mallick et al., 2008, Howard et al., 2013, Kim et al., 2018), the combination of RAP and the soft binder significantly improved fatigue life of the WMRA mixture as shown in Figure 6-28 and Figure 6-31. The fatigue life of WMRA was about three times that of HMA. It seems that the presence of the soft binder significantly extended the steady-state which prolonged the fatigue life of this mixture. This result can only be explained by the incorporation of the soft binder, which improves fatigue damage resistance of WMRA. The same results can be seen when WMA additives and the 93 pen binder are incorporated to produce SWMRA and CWMRA mixtures, as shown in Figure 6-29, Figure 6-30 and Figure 6-31. The presence of the 93-pen binder increased the stiffness of these mixtures in comparison with the WMRA prepared with the 188-pen binder. Sasobit further increased the modulus due to the stiffening effect of this additive. Cecabase slightly reduced the modulus and increased fatigue damage rate during the steady-state stage of the CWMRA mixture. However, this additive seems to be altering the failure mechanism of asphalt containing RAP; the presence of RAP brittle binder can be detected by the steep, sharp crack formation phase. In the case of Cecabase incorporation, the stage of sharp crack formation was noticeably extended and flattened compared to WMRA and SWMRA, which showed steeper crack formation stages. This observation can be interpreted as implying that the crack propagation rate in asphalt mixtures containing Cecabase may be slower than in other mixtures.

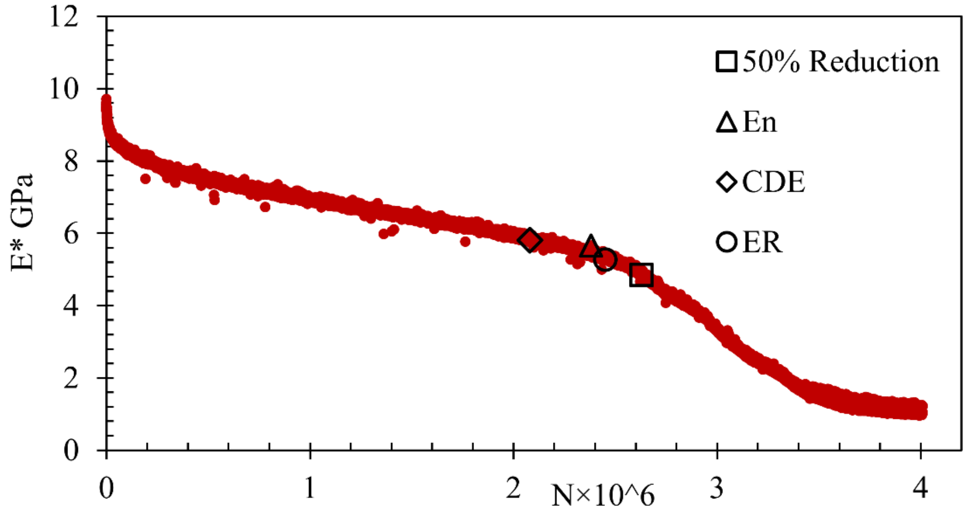


Figure 6-25. HMA fatigue life determination, at 20°C and 100 με

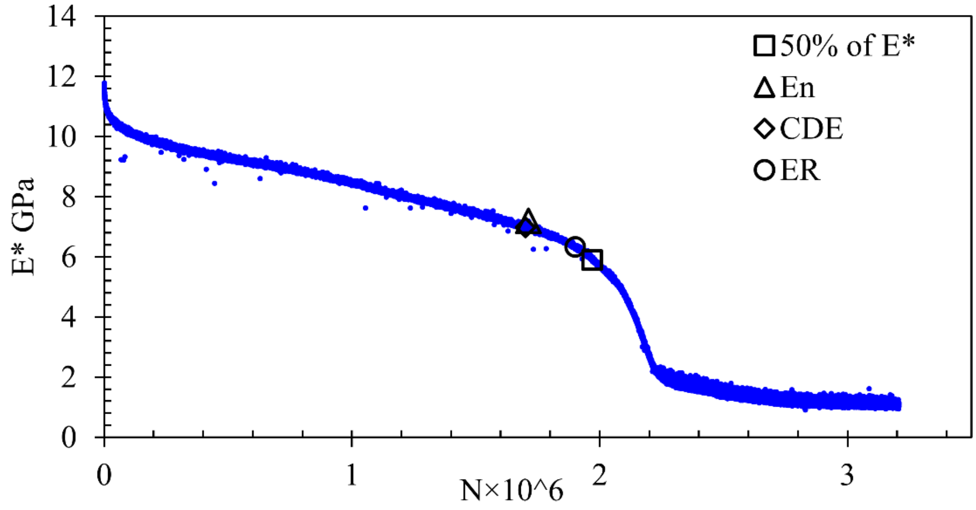


Figure 6-26. SWMA fatigue life determination, at 20°C and 100 με

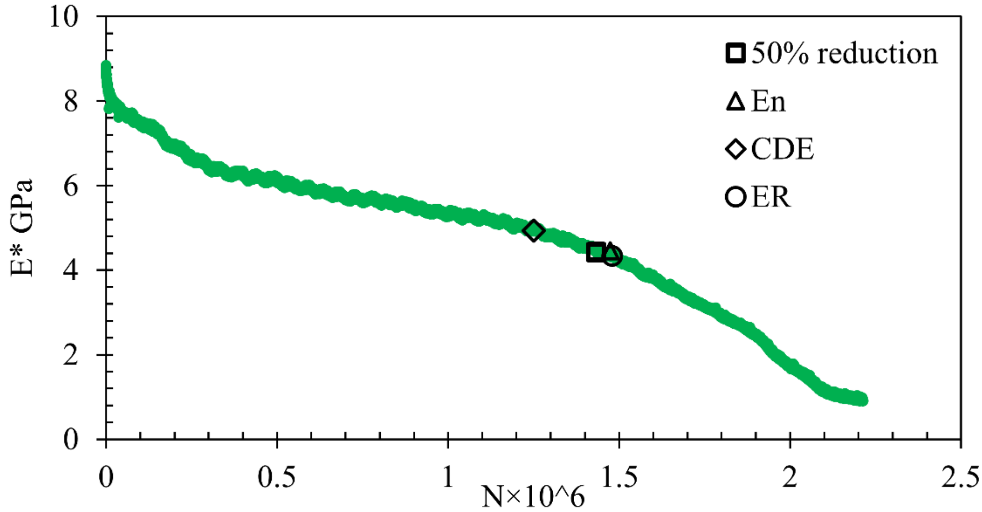


Figure 6-27. CWMA fatigue life determination, at 20°C and 100 με

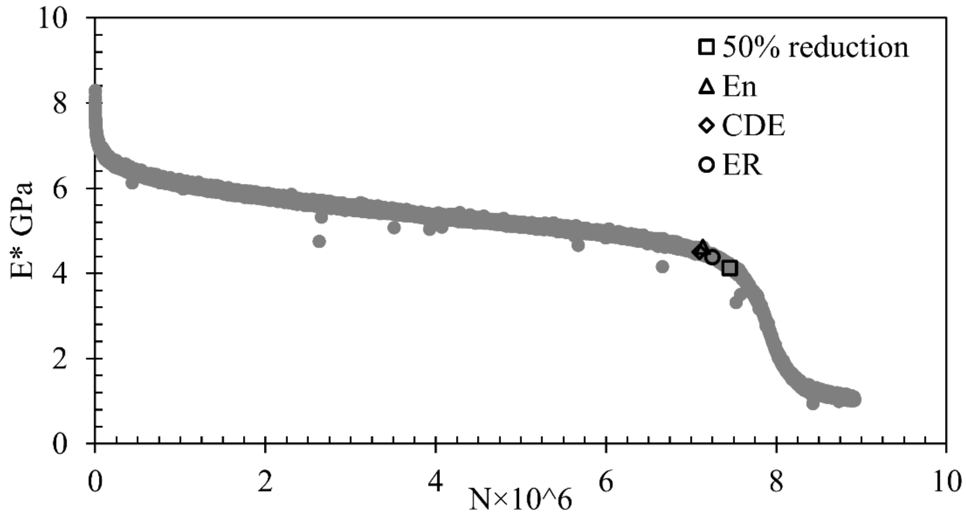


Figure 6-28. WMRA fatigue life determination, at 20°C and 100 με

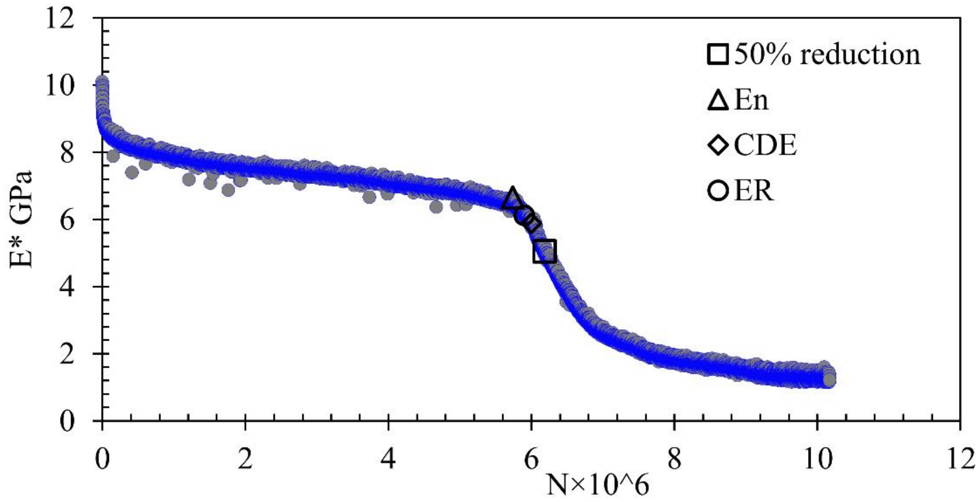


Figure 6-29. SWMRA fatigue life determination, at 20°C and 100 με

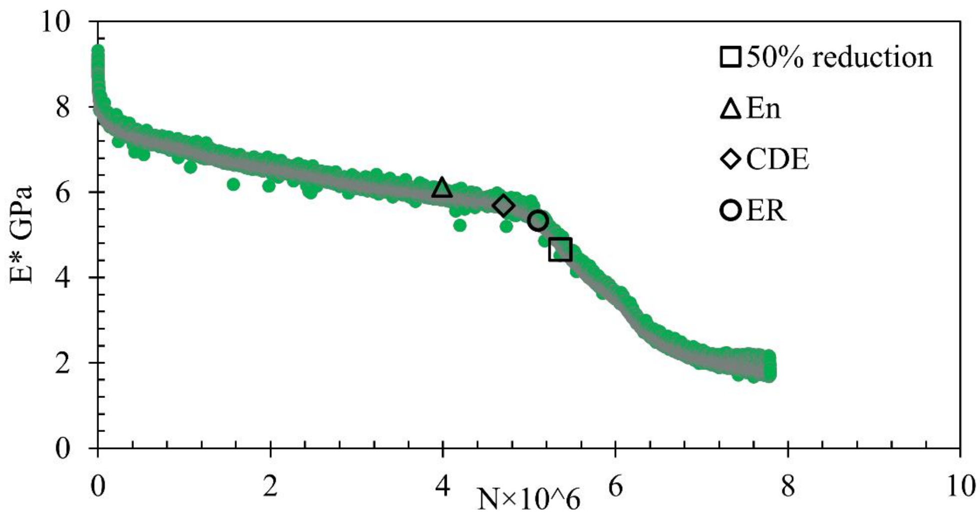


Figure 6-30. CWMRA fatigue life determination, at 20°C and ~100 με

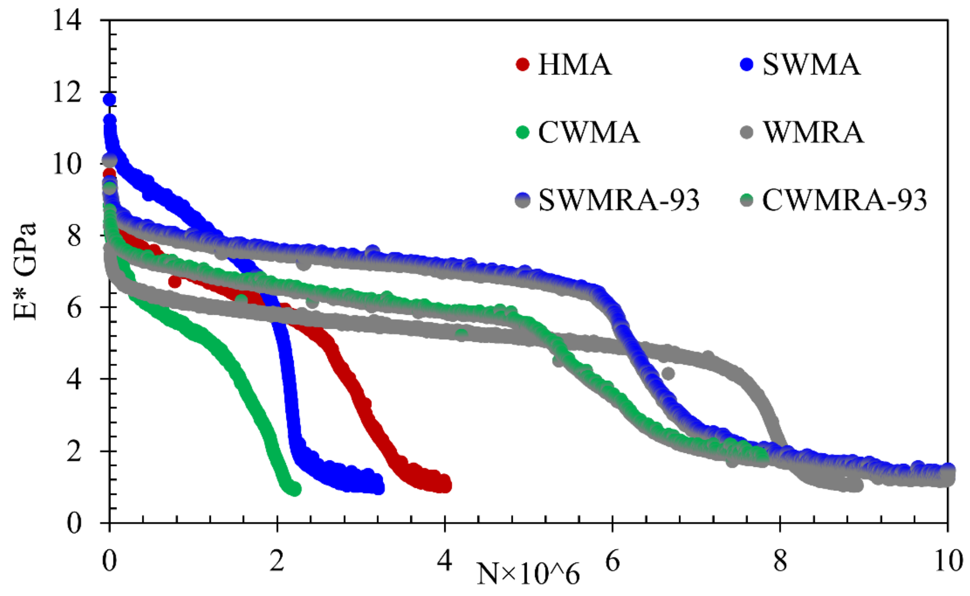


Figure 6-31. Fatigue curves of all mixtures at 20°C and 100 με

6.5.5.2 Construction of Fatigue Lines

The previous analysis was based on one strain level. To investigate fatigue performance at different strain levels, it is necessary to construct fatigue lines. In the conventional fatigue cracking analysis, 50% reduction in modulus, plotting the number of applications to failure against the applied strain levels, shows that there is a logarithmic, linear relationship between these two measures. Accordingly, fatigue life can be expressed as a function of strain using the following power equation:

$$N_f = f_1 \times {\varepsilon_o}^{f_2} \qquad \text{Eq. 6-14}$$

where N_f is the number of load applications to form a sharp fatigue crack, ε_o is the applied strain level, and $f_{1,}f_2$ are material constants can be calculated by a fitting process. Using this equation, fatigue lines of the studied mixtures were constructed; Table 6-2 shows fitted model parameters and coefficient of regression (R^2) values of the fitting process. This table indicates that R^2 values equal at least 0.96 for mixtures that did not contain RAP, and between 0.91-0.96 for mixtures containing

RAP. This larger variation in the fatigue results can be explained by the inhomogeneity in the properties of RAP even within the same source of RAP (Lo Presti et al., 2016). Figure 6-32 presents SWMA and CWMA results in comparison with HMA, Figure 6-33 presents the results of mixtures containing RAP compared with the control mix. In these figures, fatigue life was plotted on the Y-axis in order to directly implement the fitted models in pavement design. The first figure indicates that at relatively low strain levels, HMA outperforms mixtures prepared with Cecabase and Sasobit. At high strain levels, the fitted lines show that WMA mixtures may perform better than the control mix, with crossover in the fatigue lines at a strain level of approximately 180 με. However, the relatively slight difference in these results seems to be minor, indicating comparable performance of WMA and HMA.

The results presented in Figure 6-33 demonstrate that the grade of the rejuvenator binder plays a critical role in fatigue performance of asphalt. This is because using 188 or 93 pen binders as rejuvenators effectively improved fatigue life of WMRA, SWMRA and CWMRA mixtures. Fatigue life of WMRA was about three times that of the HMA. CWMRA fatigue life was slightly lower than that of WMRA. This can be explained by two reasons; either due to using a stiffer binder in this mix (93-pen), or because of the effect of Cecabase which can lead to reducing fatigue life of asphalt as was noticed in Figure 6-31. SWMRA apparently outperformed WMRA at low strain levels, but the presence of Sasobit made the mixture more sensitive to the applied strain as this mixture's fatigue life was lower than WMRA at high strain levels such as 320 με.

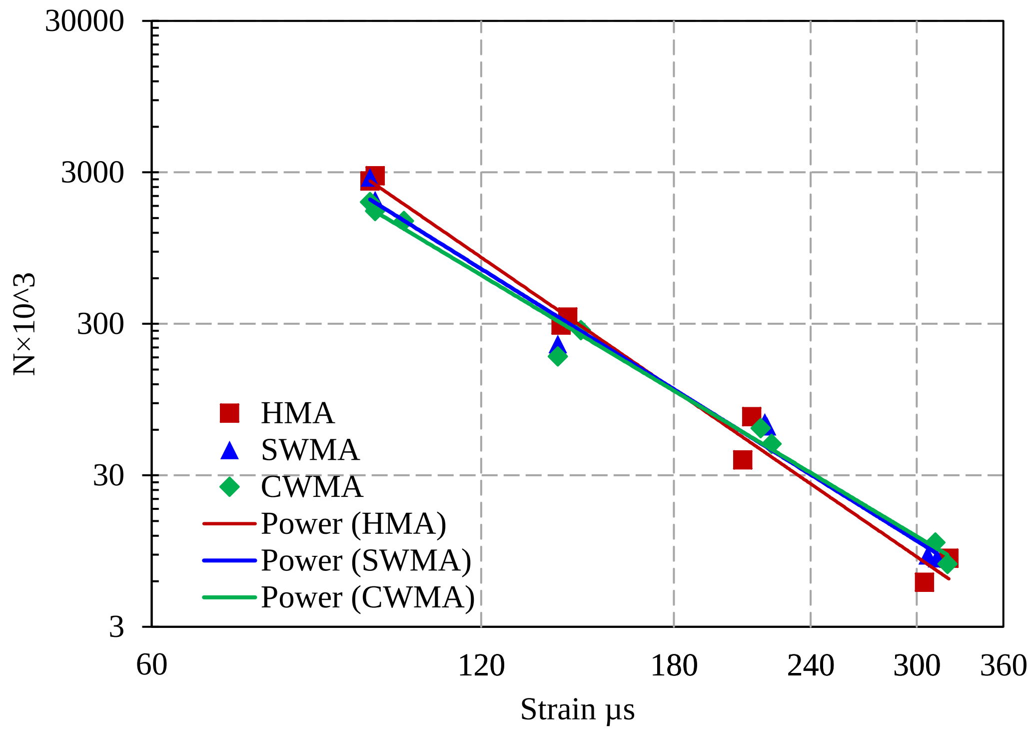


Figure 6-32. Fatigue lines of HMA, SWMA, and CWMA at 20°C

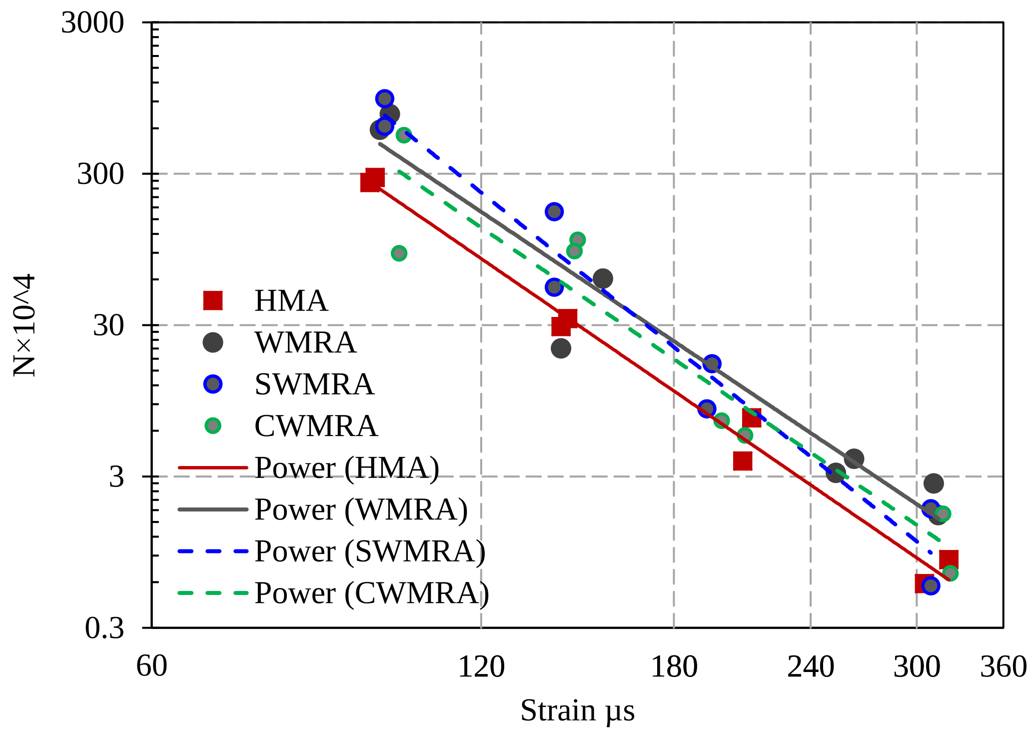


Figure 6-33. Fatigue lines of HMA, WMRA, SWMRA, and CWMRA at 20°C

6.5.5.3 Effect of Testing Temperatures on Fatigue Performance

Fatigue cracking damage is a traffic dependent and temperature-dependent process. Definitely, increasing traffic volume leads to accelerating fatigue cracking damage. The effect of temperature on fatigue damage can be understood by the relationship between the temperature and material stiffness. At low temperatures, asphalt stiffness is high, which leads to decreasing strain levels induced by traffic. Conversely, at high temperatures, asphalt stiffness is low, which leads to increasing strain levels. If the strain level is constant, however, what is the effect of the temperature on fatigue cracking damage? In several WMA related studies, the effect of temperature on fatigue performance has not been considered (Xiao et al., 2009, Lu and Saleh, 2016, Raab et al., 2017). Other studies considered testing at different fatigue temperatures, usually 10, 20 (Maggiore, 2014, Dondi et al., 2013), these studies demonstrated that temperature or mix stiffness has an important effect on fatigue performance; and it was recommended to take the effect of this factor into consideration in the structural design of asphalt pavements (Almeida et al., 2018).

Accordingly, to investigate the relationship between test temperature and fatigue performance of WMA and WMRA mixtures, further fatigue tests were conducted at two additional temperatures of 0 and 10°C using the two-point bending machine at the same strain levels. The 50% reduction in modulus analysis method was used to calculate fatigue life to failure. Equation 6-14 was fitted to the results; Table 6-2 shows the fitted model parameters and the coefficients of regression. This table shows all R^2 results are more than 0.95, except WMRA tested at 0°C, which was 0.91, indicating higher variability in the properties of this mix most likely due to RAP incorporation.

Figure 6-34 and Figure 6-35 present fatigue cracking results at testing temperatures of 10 and 0°C, respectively. Both of these figures indicate that CWMA performance is fairly comparable to HMA. At a testing temperature of 10°C, the HMA fatigue line was very slightly higher than that of CWMA; whereas at 0°C the lines were almost identical indicating similar performance of these mixtures. SWMA performance was also similar to HMA at 10°C and low strain levels, but at high strain levels, HMA slightly outperformed SWMA. At 0°C, SWMA noticeably outperformed HMA at low strain levels, but at strain levels above 200 με, HMA performed better than this mixture. This fatigue performance behaviour of SWMA can probably be explained by the effects of this additive on bitumen rheology. Obviously, this additive increased mixture stiffness, and that seems to be improving fatigue resistance at low strain levels. However, this mixture also became more sensitive to strain as at high strain levels fatigue life became less than that of HMA.

WMRA significantly outperformed HMA at a test temperature of 10°C. At a test temperature of 0°C, however, fatigue performance of this mixture becomes more sensitive to strain; improved performance at low strain and degraded performance at high strain levels. In fact, fatigue life of WMRA was quite comparable to that of SWMA, as shown in Figure 6-35. This result can be interpreted as indicating that these mixtures may more prone to fatigue cracking or low temperature cracking when the strain level exceeds the strain tolerance that the material can stand, in this case approximately 220 με. But this is an unlikely scenario because generally tensile strain levels in a typical asphalt pavement are about 30-200 με (Hunter et al., 2015).

Table 6-2. Fatigue model parameters and coefficients of regression

Mix	Temperature °C	f_1	f_2	R^2
HMA	20	2.00E+16	-4.967	0.98
SWMA	20	2.00E+15	-4.512	0.97
CWMA	20	6.00E+14	-4.334	0.98
WMRA	20	2.00E+16	-4.852	0.94
SWMRA	20	2.00E+18	-5.792	0.96
CWMRA	20	2.00E+16	-4.938	0.91
HMA	10	4.00E+14	-4.395	0.98
SWMA	10	3.00E+15	-4.898	0.96
CWMA	10	5.00E+14	-4.508	0.97
WMRA	10	3.00E+17	-5.489	0.96
SWMRA	10	NA*	NA	NA
CWMRA	10	NA	NA	NA
HMA	0	3.00E+14	-4.426	0.97
SWMA	0	4.00E+16	-5.383	0.97
CWMA	0	5.00E+14	-4.525	0.95
WMRA	0	1.00E+17	-5.595	0.91
SWMRA	0	NA	NA	NA
CWMRA	0	NA	NA	NA

* Not Available

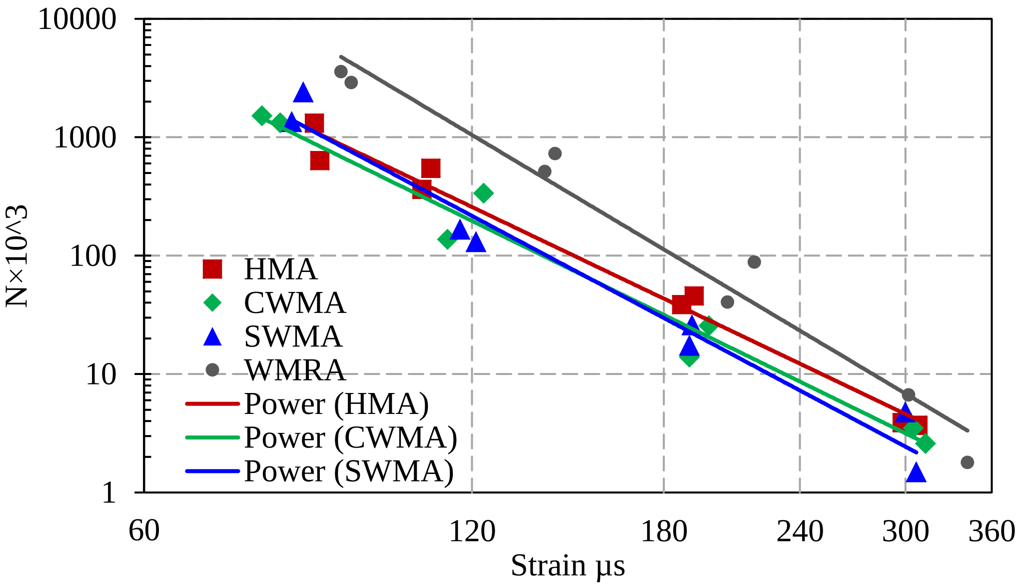


Figure 6-34. Fatigue cracking results at 10°C

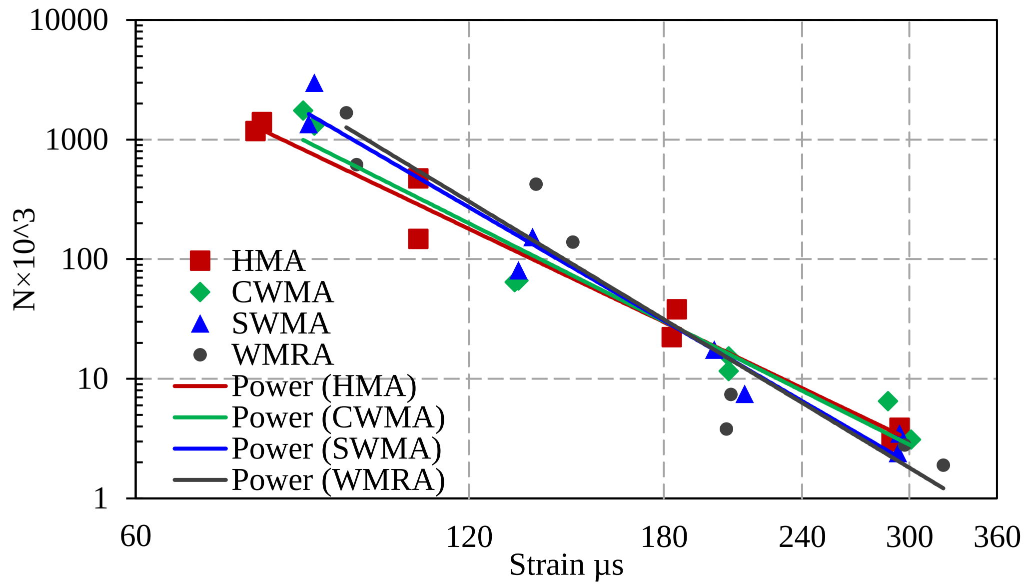


Figure 6-35. Fatigue cracking results at 0°C

6.6 Durability of WMA

Asphalt pavements are subjected to repeated traffic loading and environmental effects such as oxidative ageing, daily and seasonal temperature variations, sunlight, and water during their in-service life. That kind of continuous exposure to these factors can substantially affect the capability of this material to resist the damage imposed by these factors. It also requires the asphalt to maintain an acceptable level of strength to resist or delay the damage due to these factors. Accordingly, asphalt durability can be defined as the ability of this material to maintain satisfactory performance under the effects of traffic loading and environmental impacts during its service life (Nicholls et al., 2008).

The consequences of asphalt durability are critical from an economic perspective. If the asphalt has limited durability, its condition can quickly deteriorate, and different distress modes can appear shortly after the start of service life. This leads to a rise in the road user costs due to the poorer condition of the road surface and an increase in the road maintenance cost due to the additional maintenance cycles required to maintain asphalt condition to acceptable requirements (Varveri et al., 2016). Therefore, asphalt should have satisfactory durability in order to reduce road user and maintenance costs and to minimise the negative economic impacts of this property.

Asphalt durability can be assessed by evaluating its performance in terms of several requirements such as stiffness, resistance to fatigue and rutting or other requirements related to the location of the asphalt layer such as noise level and skid resistance in the case of surface layers (Nicholls et al., 2008). Most of these performance indicators are affected by two critical factors that can significantly affect durability of asphalt, namely oxidative ageing and moisture induced damage (Airey, 2007, Hunter et al., 2015, Varveri et al., 2016). Asphalt ageing is a complex process and happens mainly due to bitumen oxidation over time which leads to embrittlement and loss of flexibility of this material causing load-related and environment related cracks (Lesueur and Youtcheff, 2013). Moisture damage is also a complex phenomenon that is affected by several factors such as aggregate chemistry, percentage and connectivity of air voids; the main impact of moisture damage is the loss of adhesion between bitumen and aggregate leading to stripping of bitumen from the aggregate surface (Hunter et al., 2015).

Accordingly, durability of selected mixtures developed in this study was evaluated with respect to long term ageing and moisture damage effect on the mechanical performance of the mixtures; since they were reported to be some of the main factors that cause mistrust in the durability of WMA and WMRA mixtures and can reduce the implementation of these mixtures (Lopes et al., 2014)

6.6.1 Long Term Ageing of Asphalt

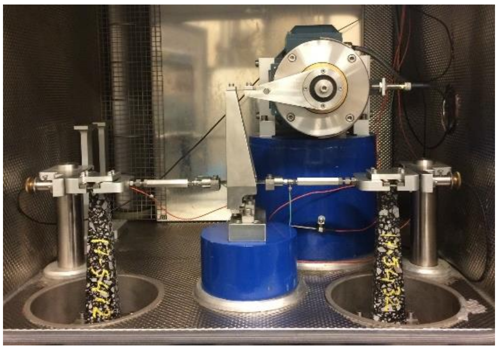
Different methods exist in literature to simulate long-term ageing of asphalt, such as subjecting compacted asphalt specimens to high temperatures for a specific time or high temperatures associated with air pressure. The aim of these methods is to assess the performance of the material after a certain period of in-service time, usually between 5-15 years. In this study, the Superpave Long Term Oven Ageing (LTOA) method developed under SHRP-A-379 project was adopted (Harrigan et al., 1994). This method requires ageing of compacted asphalt samples in a draft oven at a temperature of 85±1°C for 120±0.5 hours. The number of oven ageing days has been investigated and validated against field ageing of asphalt in some studies. Bell et al. (1994) demonstrated that eight days of oven ageing at 85°C was equivalent to 18 years of field ageing based on modulus values. Monismith et al. (1994) showed that the field ageing of asphalt depends on climatic conditions. In a wet-not freeze climate, four days of oven ageing at 85°C is equivalent to 15 years; whereas, in dry-freeze weather, four days of ageing is equivalent to about seven years. Since the weather in the UK is generally wet-limited freeze cycles, then, it can be concluded that five days of oven ageing at 85°C can be considered equivalent to approximately ten to fifteen years of field ageing.

It must be stated here that this ageing procedure requires short term ageing of asphalt before the LTOA. However, due to the reduced ageing of WMA and WMRA because of the lowered production temperatures of these mixtures, which has been considered as a critical factor that increases rutting potential of that kind of asphalt, only a limited short-term asphalt ageing was allowed in this study. The short term ageing procedure was to heat the bitumen in an oven for three hours at the designated mixing temperature, mix aggregate and binder at the required mixing temperature, place the asphalt in metal moulds preheated to the compaction temperatures, wait until the asphalt meets the compaction temperature, compact the asphalt to the required density and leave the material twenty four hours to cool down to the ambient temperature. In this procedure, the effect of the actual production conditions of the studied mixtures can be reflected in the performance of these mixes; also the effect of the ageing retarding capability of Sasobit and Cecabase can be considered in the durability analysis of WMA produced by these mixtures.

To apply the selected LTOA procedure, two setups were developed. One to age trap samples, as shown in Figure 6-36 A; the specimen was sandwiched between two holed plates fixed with eight bolts to prevent any deformation during the LTOA process. The other was for cylindrical samples, as shown in Figure 6-36 B; the core sample was constrained by a thin holed plate to avoid any deformation during sample ageing. At the end of the LTOA process, all aged samples were passed through careful visual inspection to ensure no damage or deformation had occurred during the ageing process.

A

B



C

Figure 6-36. LTOA setup for trapezoidal and cylindrical specimens; A, a trap sample sandwiched between two holed plates, B, ageing of trap samples in an oven, and C, testing of trap samples after LTOA process in the two-point bending

6.6.1.1 Ageing Impact on E* of Asphalt

To investigate ageing impact on the dynamic modulus of the studied mixes, the E* test was performed on LTOA samples using the two-point bending machine. The impact of ageing was analysed by the difference in the E* and δ at different temperatures and frequencies before and after the ageing process, and ageing indices were developed using the following equation.

$$Ageing\ Index = Indicator_{before\ ageing}\ /Indicator_{after\ LTOA} \quad \text{Eq. 6-15}$$

Ageing index results calculated by E* are presented in Figure 6-37 and Figure 6-38. The first figure indicates that at low temperatures, the effect of ageing is marginal as the increase in E* is about 10% only. However, in the second figure, which shows E* results at a test temperature of 30°C, it can be seen that the ageing index is between 1.3-1.65. Also, both figures demonstrate that Cecabase and Sasobit can retard the ageing process since the increase in E* is about 50%, whereas E* increased by about 65% in the case of HMA. These results confirm the conclusion made in chapter three that these additives can retard the bitumen ageing process.

On the other hand, ageing of WMRA was relatively slower than that of HMA. This result was slightly unexpected because of the presence of the soft binder, which was expected to age more than other binders. However, this result can probably be explained by the following reasons. First, most of the soft binder reacted with the RAP binder, which means the chemical composition of the soft binder was changed, and the susceptibility of the binder to ageing was changed as well. Second, any soft binder that remained unblended in the mix after mixing is basically a minor amount probably present as microparticles surrounded by other components of asphalt. This means that it is unlikely to age the remaining soft binder since it is difficult to force the oxygen to reach these particles.

Furthermore, Figure 6-39 presents ageing index based on phase angle. This figure shows that δ generally decreased after ageing, which agrees with the impact of ageing on asphalt rheology. However, comparing the changes in δ and E* indicates that the ageing impact is more pronounced on E* than δ since the reduction in the phase angle was about 9-18% only. This means that assessment of ageing should be made based on E* since it is the parameter most affected by the ageing phenomenon.

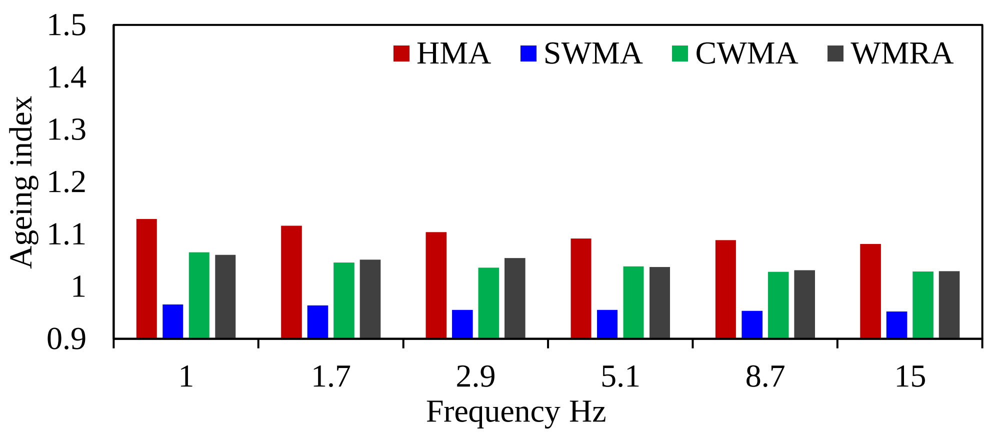


Figure 6-37. Ageing index by E* at 0°C

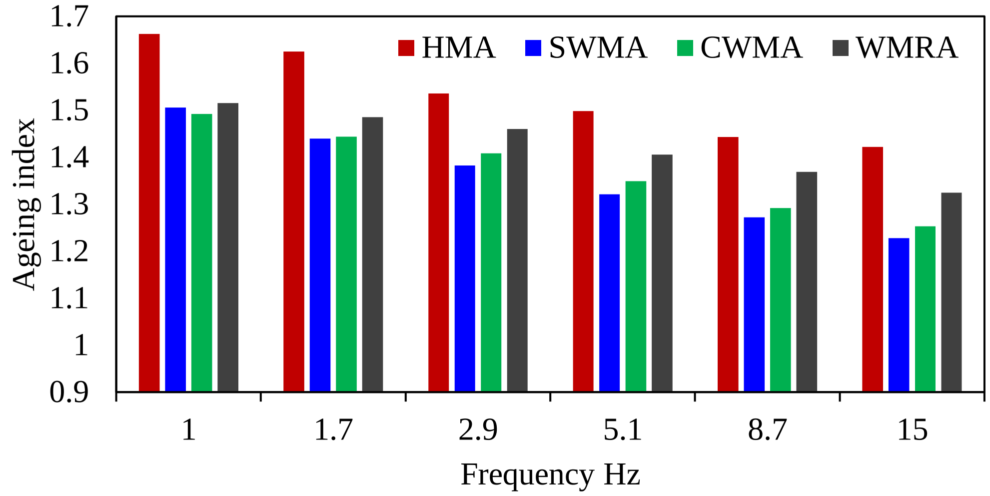


Figure 6-38. Ageing index by E* at 30°C

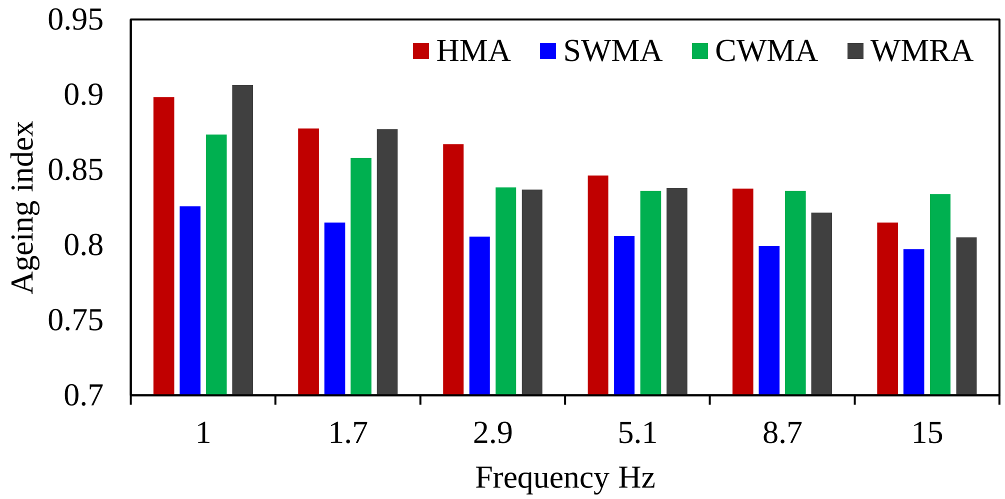


Figure 6-39 Ageing index by δ at 30°C

6.6.1.2 Ageing Impact on Fatigue Cracking of Asphalt

The effect of ageing on fatigue cracking of asphalt can be considered as one of the main factors that affects durability of asphalt pavements. This is because as the material becomes stiffer over time, it also loses its flexibility and strain recovery properties as well. Ageing impact on fatigue cracking resistance was evaluated in this study by analysing fatigue lines of the studied mixes before and after the LTOA process. The fatigue cracking test was conducted at a test temperature of 20°C using the two-point bending machine following the same procedure explained earlier.

Fatigue lines after the LTOA process were constructed and compared to those before the ageing process, as shown in figures 6-40 to 6-44. Figure 6-40 indicates that fatigue performance of HMA slightly dropped after LTOA at low strain levels, whereas it was indistinguishable at high strain levels. SWMA fatigue performance was improved at strain levels lower than 170 με, and it dropped at strain levels larger than that, as shown in Figure 6-41. This means that SWMA became more sensitive to strain after ageing, which can be interpreted as an increase in the fatigue cracking potential over ageing if the strain level exceeds 170 με. This result can be explained by the combined stiffening effect of ageing and Sasobit, which made SWMA more sensitive to strain and reduced its strain flexibility. Figure 6-42 presents fatigue lines of CWMA before and after ageing; this figure demonstrates that the LTOA process did not change fatigue performance of that mix. This observation can be explained by the ageing retardation capability of Cecabase, which made CWMA fatigue performance almost constant over ageing of that mix. WMRA fatigue performance was also improved after the LTOA, but at strain levels more than 200 με the performance dropped. This means that this mixture became sensitive to the loading strain after the ageing process. The improvement in fatigue performance of this mix can probably be explained by the presence of the soft binder, and by an improvement in the cohesion of the binder phase due to the ageing process. This means that the performance of this kind of asphalt is very likely to improve under field ageing and binder oxidation processes over time.

To further analyse the ageing impact on fatigue performance and for the sake of presenting fatigue data in a clearer way than showing fatigue lines on log scale figures, a bar chart as shown in Figure 6-44 was developed. This figure presents the normalised fatigue life after LTOA compared to the one before the ageing process at three strain levels of 100, 150, 300 με. This figure indicates that fatigue life of HMA dropped about 32, 28, and 19% at strain levels of 100, 150, and 300 με respectively after the LTOA process. This means that this mixture became fatigue cracking susceptible due to the embrittlement and loss of strain tolerance after the LTOA process. SWMA fatigue performance increased by about 50% at a strain level of 100 με, whereas it was comparable to the performance before LTOA at 150 με and dropped about 48% at the high strain level. Despite the increase in the fatigue life of this mix, this result means that it became more sensitive to strain after ageing, which means it may not sustain large tensile strain in the field. Fatigue life of CWMA reduced about 18-25% at 100 and 150 με, but it dropped by about 29% when tested at 300 με. Moreover, the figure shows WMRA fatigue performance was significantly improved after the ageing process. At 100 με, fatigue life increased by about 450% whereas it increased by about 200% at 150 με. However, it dropped to about 50% at 300 με indicating the increased sensitivity of this mixture to high strain levels.

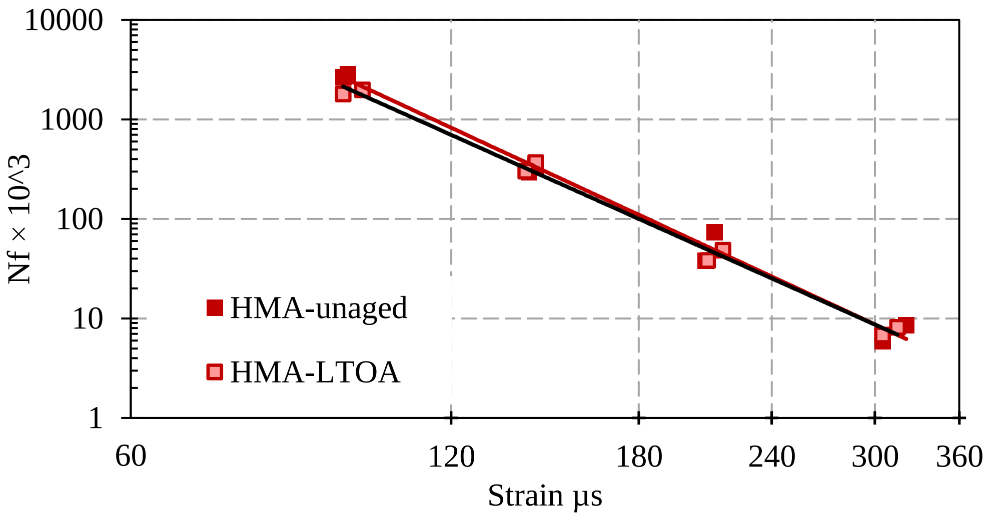


Figure 6-40. Fatigue lines of HMA before and after ageing

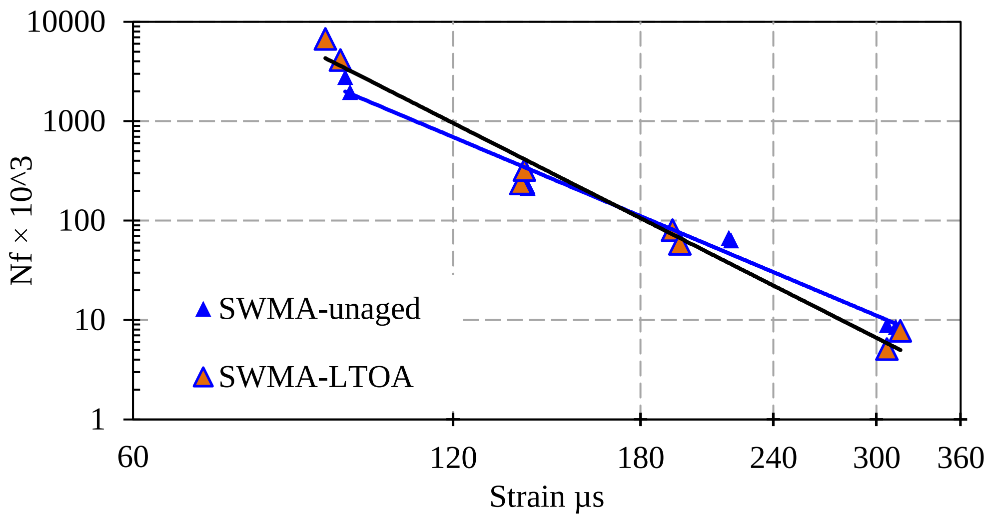


Figure 6-41. Fatigue lines of SWMA before and after ageing

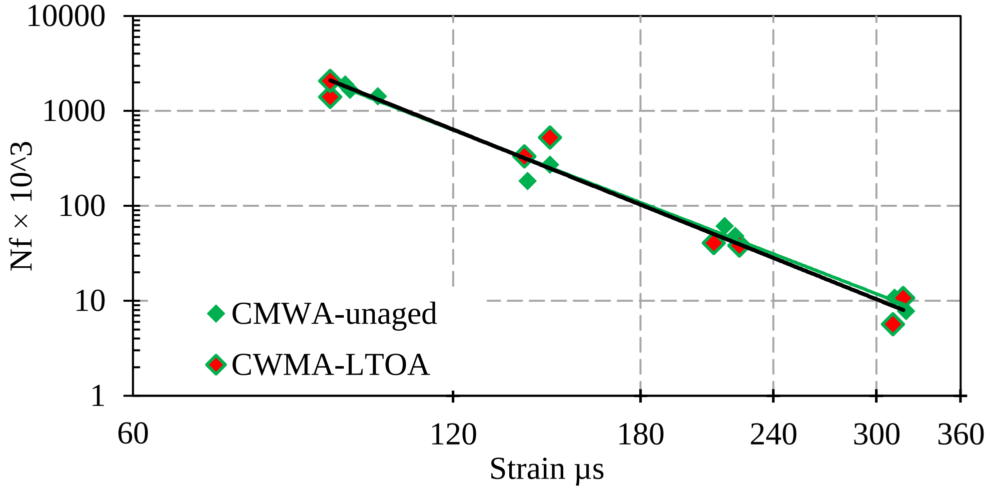


Figure 6-42. Fatigue lines of CWMA before and after ageing

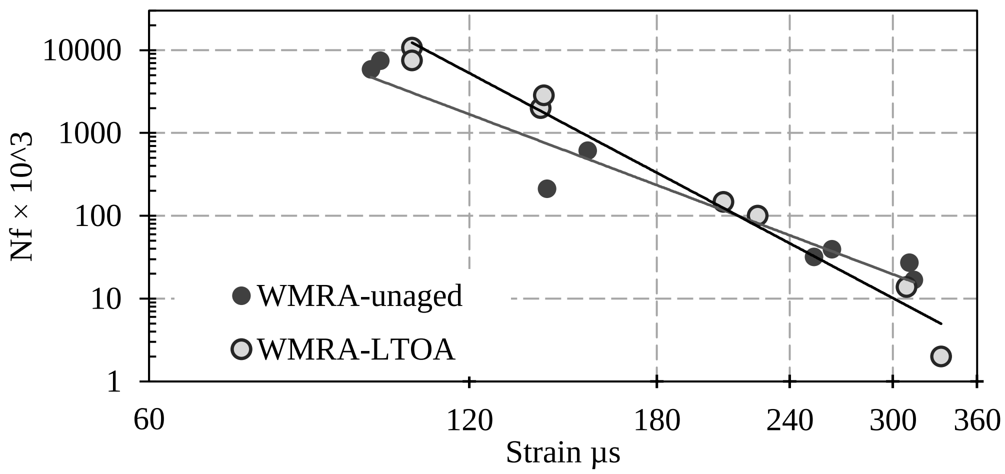


Figure 6-43. Fatigue lines of WMRA before and after ageing

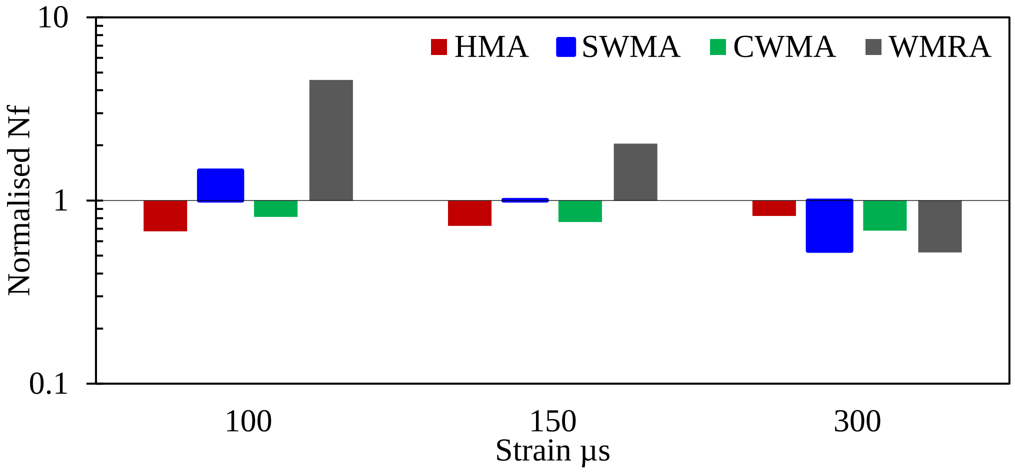


Figure 6-44. Normalised Nf (Nf LTOA / Nf unaged)

6.6.2 Moisture Damage Resistance (MDR)

Moisture damage resistance is one of the critical factors that leads to degradation of asphalt pavement condition over time. The effect of this factor on the integrity of asphalt has traditionally been investigated by monitoring the changes in the mechanical performance usually stiffness and strength before and after subjecting this material to particular moisture conditioning method (Airey and Choi, 2002). There are several moisture conditioning protocols that aim to simulate moisture damage under different environmental conditions. These include the Lottman conditioning procedure (Lottman, 1982), or the Modified Lottman procedure (AASHTO, 2018). In this study, water conditioning of asphalt was performed according to British Standard BS EN 12697-12 (BSI, 2003a). By this standard, ITS of at least six specimens is tested, three before conditioning and three after conditioning, then the influence of water damage on material response can be assessed using the following equations:

$$ITSMR = 100 \times ITSM_{wet}/ITSM_{dry} \qquad \text{Eq. 6-16}$$

$$ITSR = 100 \times ITS_{wet}/ITS_{dry} \qquad \text{Eq. 6-17}$$

where $ITSR$ is the indirect tensile strength ratio, and $ITSMR$ is the indirect tensile stiffness modulus ratio. These indicators represent the change in material stiffness and strength due to water damage occurring during the water conditioning process. The water conditioning procedure consisted of the following steps:

1. Place the specimen in a vacuum container and fill the container with distilled water that has a temperature of 20 ± 5 °C. Figure 6-45 A shows the system used for this task.
2. Gradually apply a vacuum required to exert a residual pressure of 6.7 ± 0.3 kPa.
3. Maintain the vacuum for 30 ± 5 minutes then slowly and gradually reduce the applied vacuum.
4. Place the sample in a water bath at a temperature of 40°C for 72 hours. Figure 6-45 B shows the water bath used in the moisture conditioning process.

Furthermore, based on field performance evaluation of some WMA projects in the US and comparative evaluation of lab conditioning and moisture damage procedures, NCHRP (Martin, 2014) proposed new criteria to evaluate MDR of that kind of asphalt. The evaluation procedure consists of two stages: first, by conducting resilient modulus, strength, and stripping inflection point tests before and after moisture conditioning; second, by the same performance indicators but this time the conditioning procedure consists of performing the LTOA process followed by the moisture conditioning cycle before performing the required tests. The evaluation criteria for both of the conditioning procedures are presented in Table 6-3.

Table 6-3. Moisture damage evaluation requirements for WMA (Martin, 2014)

Indicator	Water conditioning set	LTOA-water conditioning set
M_R	1379 MPa	≥3103 MPa
M_R ratio	≥70%	Unspecified
ITS	0.45 MPa	≥0.793 MPa
ITSR	≥70%	Unspecified
Stripping inflection point	≥3500 cycles	≥12000 cycle
Stripping slope	≤5.3 µm/cycle	≤1.4 µm/cycle

In this study, however, only stiffness and strength measurements were conducted to evaluate MDR of the studied mixtures. Since Table 6-3 does include any specification for stiffness measurements, it is suggested in this study to use 70% as a threshold to evaluate moisture susceptibility of WMA. This limit was suggested because both M_R and ITSM are measures of material modulus of elasticity, the only difference being that M_R is measured by dividing the peak stress by the peak strain of a cyclic applied load whereas ITSM is measured by dividing peak stress by the peak strain of a pulse load. Correspondingly, a 30% reduction in ITSM was considered as a limit to evaluate MDR of WMA mixtures. Figure 6-45 graphically presents the followed procedure to evaluate MDR of WMA and WMRA mixtures.

Following this procedure, MDR of WMA and WMRA mixture was assessed and compared to that of HMA. Figure 6-46, Figure 6-47, and Figure 6-48. present ITSMR, ITS, and ITSR results after one cycle of moisture conditioning. The first figure indicates that the reductions in ITSM of HMA and SWMA were quite comparable, 28 and 30.6% respectively. CWMA exhibited slightly more reduction in the stiffness, about 36%. This may indicate that CWMA may be more susceptible to moisture damage in terms of stiffness reduction. Moisture resistance of WMRA was the best as the reduction in the stiffness of this mix was less than 5%. This indicates that RAP incorporation can improve moisture damage resistance based on stiffness measurements. On the other hand, MDR results with respect to strength are presented in Figure 6-47 and Figure 6-48. These results indicate that all mixtures can have an acceptable MDR based on NCHRP criteria. The retained ITS after the moisture conditioning of these mixtures was significantly above the NCHRP threshold limit, 0.45MPa.

Also, ITSR results indicate that these mixtures are not moisture susceptible since they passed 70% minimum strength ratio. However, in terms of mixture ranking, HMA and SWMA durability was generally comparable based on all indicators. CWMA had slightly less durability than these mixes. WMRA exhibited the best durability out of all the mixes. This result agrees with many published studies that have shown that RAP incorporation can enhance MDR of asphalt (Shu et al., 2012, Lu and Saleh, 2016, Singh et al., 2017). However, these results may disagree with several other studies which showed that RAP incorporation degraded moisture damage resistance of asphalt. This could be explained by different contributing factors from one study to another, such as chemistry of aggregate used, binder properties, RAP properties, and mix volumetric properties.

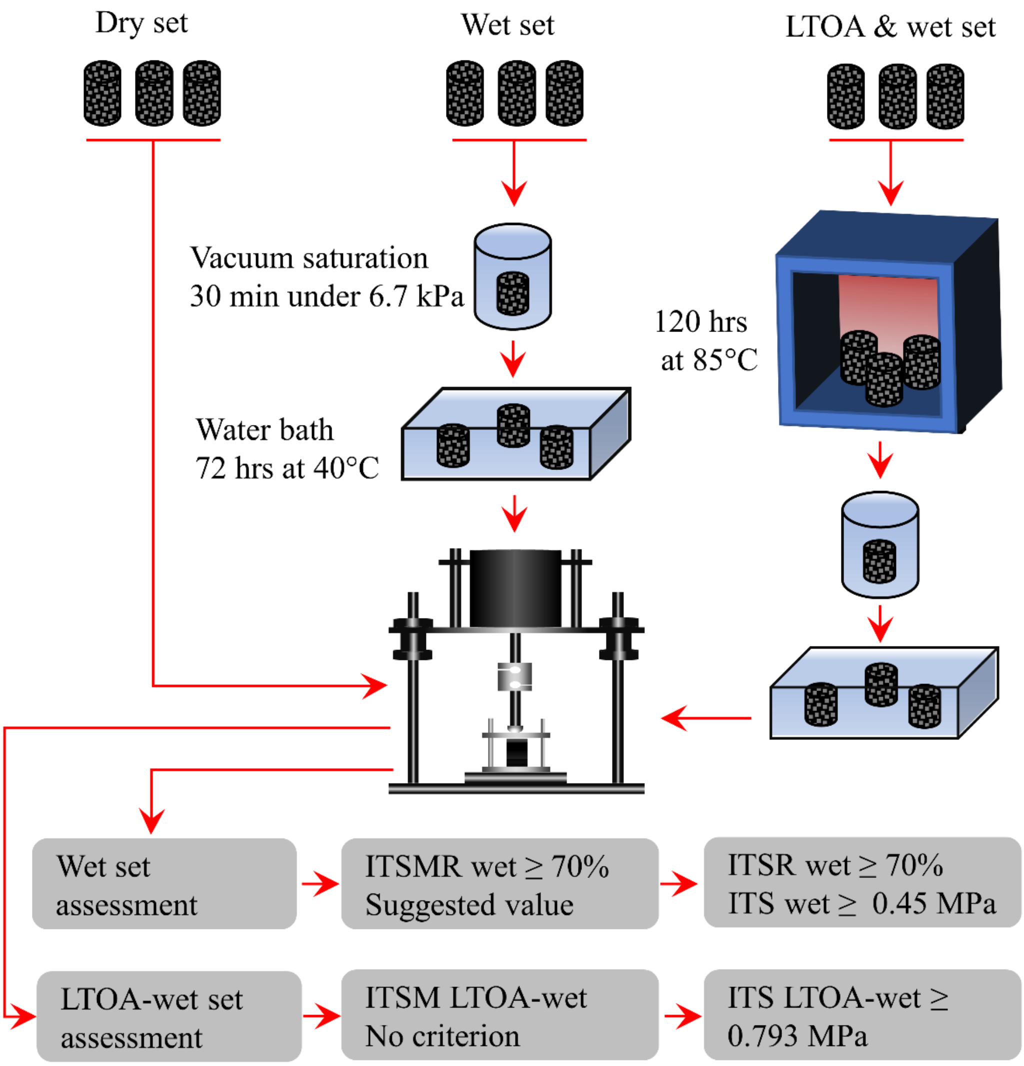


Figure 6-45. Evaluation procedure of MDR

Regarding mix durability after the combined LTOA and moisture conditioning effects, the results of this evaluation procedure are presented in Figure 6-49 and Figure 6-50. The first figure presents stiffness measurements of the unconditioned state and after LTOA and combined LTOA with 1, 2 and 3 Moisture Conditioning Cycles (MCC). The reason for conducting that number of MCCs was that after the first MMC, the ITSM did not drop significantly, so it was decided to conduct the second the third MMC and ITSM was measured after every cycle. The results of this step are presented in Figure 6-49, which indicates that LTOA significantly improved MDR of all mixes. Also, this figure shows that stiffness reduction after the first MCC was almost stabilised for CWMA; HMA showed similar results with slightly more reduction in the stiffness. However, despite SWMA and WMRA exhibiting the highest stiffness measurements after all MCCs, the stiffness of these mixes was dropping after every MMC. This may indicate that the long-term MDR of these mixes may also drop. Nevertheless, three MCCs can be considered as an aggressive conditioning protocol, so the performance of the mixes should be sufficient with respect to MDR. This conclusion can be confirmed by the results in Figure 6-50. This figure demonstrates that ITS values of all mixes are significantly above the NCHRP limit of 0.793 MPa. Bearing in mind that this limit was set to one MCC whereas the current results are after three MCCs, this indicates that these mixes should exhibit sufficient MDR in the field.

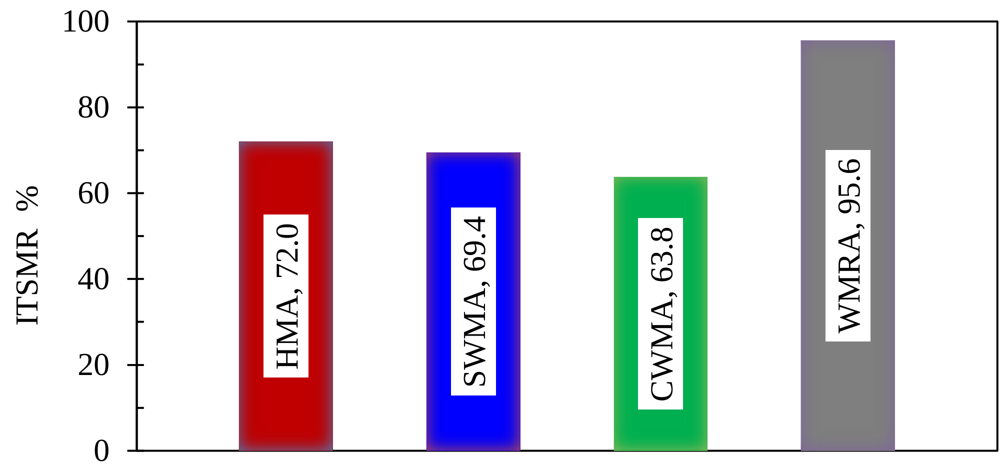


Figure 6-46. ITSMR results after moisture conditioning

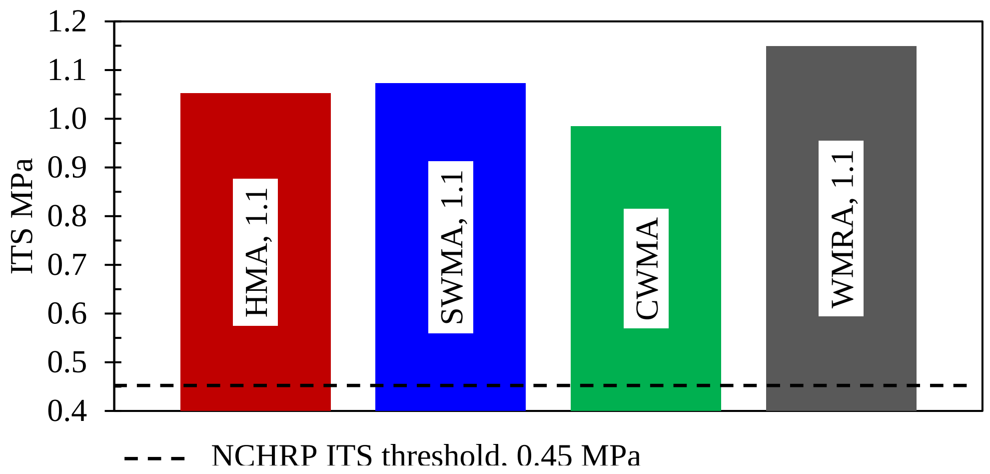


Figure 6-47. ITS results after moisture conditioning

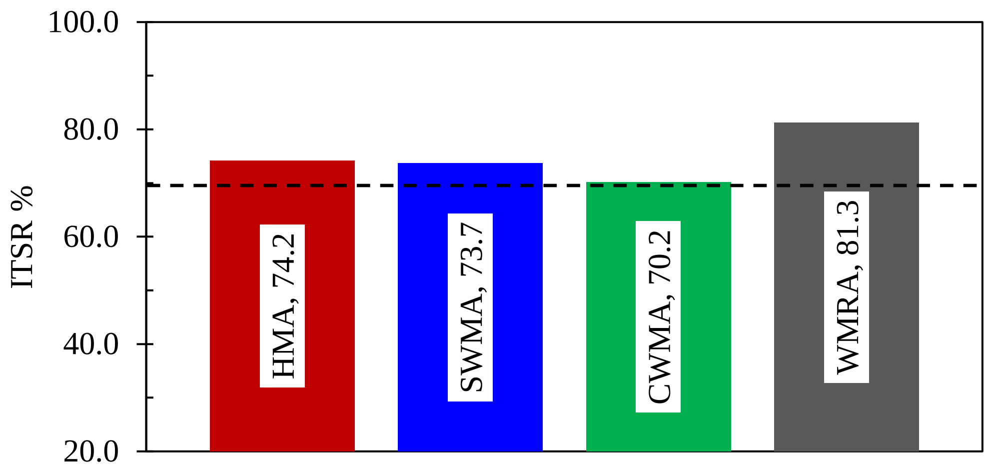


Figure 6-48. ITSR results after moisture conditioning

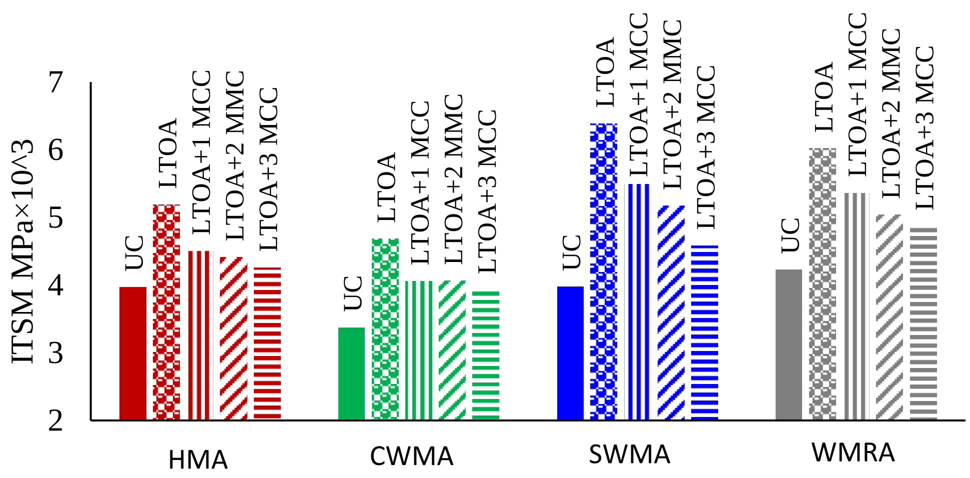


Figure 6-49. ITSRM results after LTOA and 1, 2, 3 MCCs

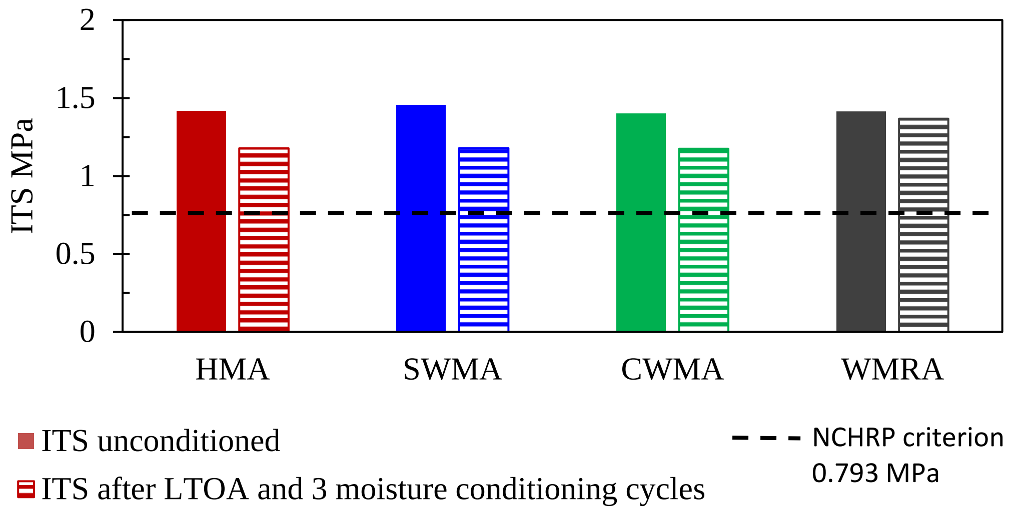


Figure 6-50. ITS measurements after combined LTOA and three MCCs

6.7 Summary and Conclusion

This chapter has presented a comprehensive performance evaluation of WMA and WMRA mixtures in comparison with HMA. Mechanical performance of these mixtures was evaluated by conducting rutting, dynamic modulus, and fatigue cracking tests at different temperatures. The durability with respect to long-term ageing and moisture damage impact on mechanical performance of the mixes was also investigated. LTOA was performed to study the ageing impact on fatigue cracking performance of the mixes, whereas MDR was studied by measuring ITSM and ITS before and after moisture conditioning and combined LTOA and moisture conditioning. Based on the results of the conducted tests, the following can be concluded:

6.7.1 Dynamic Modulus

1. Sasobit generally increased E* of SWMA, the increase was minor at low temperature but significant at intermediate and high temperatures. These

results suggested that Sasobit may improve rutting resistance but may reduce fatigue life due to the increased stiffness of SWMA.

2. Cecabase did not change E* of CWMA in comparison with HMA at all temperatures and loading frequencies. This meant that performance of this mixture may be comparable to the control mix. However, rutting and fatigue cracking measurements indicated that E* measurements cannot be used to interpret performance of CWMA.

3. E* results of WMRA indicated that this mix can outperform HMA in terms of fatigue since the presence of the soft binder reduced E* at intermediate temperatures. However, the reduced E* values at high temperatures showed that this mix may be susceptible to permanent deformation failure.

6.7.2 Permanent Deformation

1. Rutting performance of SWMA was noticeably better than HMA, especially at high temperatures, such as 50 and 60°C. This result can be explained by the effect of Sasobit on rheological properties of the base binder. Sasobit increased the complex shear modulus of bitumen, as concluded in chapter three; this effect improved permanent deformation performance of this kind of WMA. Moreover, reducing production temperature of asphalt from 155°C to 135°C did not compromise rutting performance of SWMA which means the reduced short-term ageing during the production process of SWMA does not have a negative impact on permanent deformation performance of that kind of WMA.

2. CWMA permanent deformation was slightly increased in comparison with HMA, and this result was clearly evident at test temperatures of 50 and

60°C. This result can be explained by three contributing factors. First, Cecabase slightly reduced the shear modulus of the base binder, which may have increased rutting potential of the mix. Second, the bitumen ageing retardation capability of Cecabase retarded bitumen stiffening during the mixing process, which also could have increased permanent deformation of CWMA. Third, reducing the production temperature of asphalt from 155 to 130°C also could have additional reduced bitumen ageing and eventually rutting resistance of CWMA. Nevertheless, these factors increased the plastic strain of CWMA about 12% more than that of HMA at a test temperature of 60°C which can be considered as a minor increase in the rutting potential of CWMA. Also, for cold regions such as the UK or most European countries, this mixture can perform sufficiently due to the low air temperatures of this region.

3. WMRA rutting performance was dependent on two factors, the grade of the soft binder and testing temperature. When a 188 dmm pen binder was used as a rejuvenator, WMRA performance was better at 40°C but worse than HMA at 50 and 60°C. However, when a 93 dmm pen binder was used as a rejuvenator alongside WMA additives, SWMRA and CWMRA performance were better at 40 and 50°C and reasonably comparable to HMA at 60°C. These results can be explained by the Superpave rutting parameter of the soft binders. The critical high temperature of the 188 dmm pen binder was about 42°; therefore, it performed sufficiently at 40°C, and the performance dropped at 50 and 60°C due to exceeding the threshold temperature of this binder. On the other hand, the critical high temperature of the 93 dmm pen binder was about 51°C; therefore, both SWMRA-93 and

CWMRA-93 performed acceptably at 40 and 50° but underperformed at 60°C due to exceeding the temperature threshold of the rejuvenator. Moreover, these results suggest that the full blending of RAP and soft binders was not entirely achieved, and even if a minor percentage of the soft binder was not fully blended, this percentage can be considered as a source of weakness that increases rutting potential of the mix. Furthermore, this conclusion indicates that RLAT is a very reliable test that can be used to study RAP binder and rejuvenator blending.

6.7.3 Fatigue Cracking

1. Four fatigue cracking analysis methods were implemented to characterise fatigue life of the studied mixes. Among these, traditional 50% stiffness reduction method was proven to be valid to predict the number of load applications till the formation of a sharp crack for HMA and WMA mixtures as well, therefore, this method can be utilised in pavement design problems to predict fatigue life of these mixtures.

2. At a temperature of 20°C, both Sasobit and Cecabase slightly reduced fatigue life of asphalt at low strain levels, but at higher stain levels these additives slightly increased fatigue life. At a temperature of 10°C, fatigue performance of these mixes became comparable at low strain levels, but HMA outperformed SWMA and CWMA at high strain levels. However, at a temperature of 0°C, Sasobit clearly improved fatigue life whereas Cecabase slightly increased it, but these additives reduced fatigue life at strain levels more than 180 $\mu\varepsilon$ indicating improved low temperature cracking performance of these mixtures.

3. With respect to WMRA, despite the fact that RAP can reduce fatigue life of asphalt, the inclusion of the soft binders 188 and 193 dmm as rejuvenators significantly improved fatigue life of this mix in comparison with the control HMA. This result can be explained by the reduced stiffness of these binders, which improved strain recovery and fatigue performance of WMA mixtures containing 50% RAP. This finding suggests that WMA with 50% RAP can be produced with superior fatigue cracking performance when soft binders are used as rejuvenators. However, caution should be taken when selecting the grade of the soft binder since it can compromise rutting performance of the mix.

6.7.4 Durability

The durability was investigated in terms of oxidative ageing, moisture resistance and combined ageing and moisture conditioning impacts on mechanical performance of the studied mixtures. The oxidative ageing was simulated by conducting LTOA of trap samples; the ageing procedure was to heat the trap specimens at 85°C for five days in an oven. Then the resistance to ageing was investigated by performing E* tests and comparing the results to the stiffness values before ageing. The results showed that ageing impact can be better characterised when studied at high temperatures and low frequencies; also, E* was affected by ageing more than δ; therefore, the former indicator should be used to study ageing. The results also demonstrated that both Sasobit and Cecabase retarded mix ageing, which agrees with what was concluded in chapter three that these additives can slow down the ageing process of bitumen. Furthermore, WMRA also showed retarded ageing in comparison with HMA. This result can probably be explained by the presence of the soft binder which reduced mix stiffness, and since the soft binder was entrapped in the mix, it was difficult to age that binder.

Ageing impact on fatigue performance was also investigated by doing two-point bending fatigue cracking testing. The results showed that fatigue life of SWMA improved after LTOA, but this mix became more sensitive to strain as the fatigue life dropped at high strain levels such as ≥180 $\mu\varepsilon$. CWMA fatigue performance dropped slightly after ageing. This result can be attributed to the effect of Cecabase on ageing; this additive significantly retarded mix ageing, which led to stabilising fatigue performance of that mix. WMRA fatigue performance was similar to SWMA, improved fatigue life at low strain and reduced fatigue life at high strain levels. This means that this mix became more sensitive to strain magnitude after ageing.

Moisture damage resistance (MDR) was studied according to British standards by vacuum saturation followed by moisture conditioning in a water bath at 40° for 72 hours. Then ITSM and ITS measures before and after moisture conditioning were utilised to assess MDR. The results were then compared to a recently published US standard to evaluate durability of WMA in order to assess MDR of the studied mixes. The results showed that HMA, SWMA, CWMA and WMRA mixtures passed the 70% ITSR requirement, and also passed the minimum retained strength of 0.45 MPa after the conditioning process. However, neither CWMA nor SWMA passed the 70% ITSMR limit suggested in this study. Furthermore, the analysis revealed that RAP incorporation can significantly enhance durability of WMA, and this result can probably be explained by the improved adhesion between RAP aggregate and binder due to ageing.

MDR was also investigated after combined LTOA and three MCCs. ITSM was monitored after LTOA and each cycle of moisture conditioning; ITS was measured at the end of this process. The results showed that stiffness of CWMA almost stabilised after the first MCC, HMA showed a similar trend but with slightly more drop in stiffness after each MCC. However, SWMA and WMRA exhibited more drop in stiffness after each MCC. Nevertheless, ITS results after the combined LTOA and the three MCCs demonstrated that these mixes were not moisture susceptible since the strength of these mixes passed the ITS threshold suggested by NCHRP, 0.793 MPa.

Chapter 7: Performance Modelling and Structural Design of WMA and WMRA

7.1 Introduction

Unlike many other engineering structures, asphalt pavements are subjected to complex repeated loading and variable environmental conditions during the service life of roads. The primary function of the pavement is to resist all of the applied loads while maintaining an acceptable level of serviceability with minimal maintenance. Accordingly, effectively distributing traffic loads under the prevailing environmental conditions and minimising construction and maintenance costs are the ultimate aims of the pavement structural design process (Hunter and Read, 2015).

Various pavement design methods can be found in the literature. AASHTO (1993) is one of the widely applied pavement design methods; by this method, the thickness of pavement layers can be calculated by determining a structural number of the pavement; the structural number represents the structural capacity of the pavement that is required to resist the design traffic volume for a given subgrade strength. This method has some advantages such as its simplicity in solving pavement design problems, and the inclusion of critical factors, including reliability, drainage, and moisture effect in the design process. However, the main asphalt pavement failure distress types, rutting and fatigue cracking, are not explicitly considered in the design process. Also, pavement stress and strain magnitudes, which are the fundamentals of any failure mechanism, are not considered in the design process.

In the UK, asphalt pavement design is usually conducted based on design guidelines published in the report LR1132 (Powell et al., 1984). The designer should first select a specific pavement foundation type based on the effective modulus of the foundation. Secondly, the thickness of the other layers, base, binder and surface can be determined based on the expected traffic volume and the properties of these layers. Lastly, the pavement is modelled as a multilayer elastic system, and the strain values at critical locations are calculated and compared against the permissible strains that are determined based on the expected traffic volume; if the calculated critical strain levels are lower than the permissible ones, then the design can be considered satisfactory. The critical strain levels considered in the design method are the horizontal tensile strain at the bottom of the bituminous or bound base layer and the compressive strain on top of the subgrade. These strain types are considered in order to ensure acceptable structural rutting and fatigue cracking performance. Although this method takes rutting and fatigue cracking distress indirectly into account, material performance with respect to these indicators is not included in the design process. Also, this method was developed based on material properties available in the UK, which means it is challenging to implement it in other regions (Thom, 2014).

The MEPDG method (NCHRP, 2004b) is probably the most advanced of these methods that explicitly take asphalt rutting and fatigue cracking performance models into account in the structural design process of asphalt pavements. In this method, an initial design is suggested based on the available material properties, then rutting, fatigue cracking and other distress types are quantified by accumulating damage in a timewise process; if the suggested design does not satisfy the required distress thresholds, then the design is revised until it satisfies the required criteria.

In this study, the MEPDG method was implemented to assess performance of the studied mixtures, since this method allows direct evaluation and quantification of rutting and fatigue cracking distress. This step is especially required to evaluate performance of WMA and WMRA mixtures because these mixtures neither perform similarly nor exhibit the same properties as conventional HMA mixtures (Zaumanis et al., 2018). This means that designing asphalt pavements based on material stiffness only or even by limiting the critical strains cannot guarantee satisfactory performance of WMA and WMRA. This is because calculating and limiting strains is only part of the design process; the other part is how the materials respond to the applied loads, and also how much damage the applied loads cause to the materials. This chapter investigates the applicability of modelling WMA and WMRA rutting and fatigue cracking performance and uses these models in pavement structural design problems by following the MEPDG approach.

7.2 Modelling of Pavement Performance

7.2.1 Rutting Modelling

In the MEPDG method, rutting is predicted by summing permanent deformation of every layer in the pavement using the following equation:

$$\mathrm{RD} = \sum_{i=1}^{number\ of\ sublayers} \varepsilon_p^i \times h_i \qquad \text{Eq. 7-1}$$

where RD is total rutting deformation, ε_p^i and h_i are the plastic strain and thickness of each sublayer. The plastic strain of the bituminous layers is calculated as follows:

$$\varepsilon_{p,i}^{as} = \varepsilon_{r,i} \times \beta_{r1} \times a_1 \times T^{\beta_{r2} a_2} \times N^{\beta_{r3} a_3} \times k_1 \qquad \text{Eq. 7-2}$$

where $\varepsilon_{p,i}^{as}$ and ε_r are the plastic and resilient strain in each asphaltic sublayer i, T is the temperature, N is the number of load repetitions, a_1, a_2, a_3 are regression coefficients, β_{r1} β_{r2} β_{r3} are field calibration factors, and k_1 is a depth parameter to account for confinement pressure in rutting prediction; this parameter can be calculated using the following equations:

$$k_1 = (C_1 + C_2 \times depth) \times 0.328196^{depth} \qquad \text{Eq. 7-3}$$

$$C_1 = -0.1039 \times h_{ac}^2 + 2.4868 \times h_{ac} - 17.342 \qquad \text{Eq. 7-4}$$

$$C_2 = 0.0172 \times h_{ac}^2 - 1.7331 \times h_{ac} + 27.428 \qquad \text{Eq. 7-5}$$

where depth is the distance from the surface to the computational point and h_{ac} is the total thickness of the asphalt layers – both in inches. It can be seen that the model presented in Eq. 7-2 depends on the elastic strain, temperature, number of load applications, and the effect of the confining pressure, which is quite logical since these factors are the main contributing factors to rutting distress. However, the design guide uses model coefficients and calibration factors that have been derived for typical materials in the US; neither of the regression coefficients nor the calibration factors have been derived for UK materials. To this end, a new approach is suggested in this study to determine the coefficients of the rutting model from RLAT results. The proposed approach is detailed as follows:

1. Another set of samples of each mix (two samples of each mix due to material availability) was tested in RLAT at a testing temperature of 30°C in order

to cover the temperature range where asphalt is susceptible to rutting distress.

2. The elastic strain in RLAT tests was determined using the following relationship:

$$\varepsilon = \frac{\sigma}{E} \qquad \text{Eq. 7-6}$$

where σ is the applied stress during RLAT tests and E is the elastic modulus determined from the E* master curve under the testing conditions.

3. Permanent strain model coefficients, a_1, a_2, a_3, were then determined by a fitting process using the optimisation function, Solver. The parameters were optimised by minimising the error between the observed and predicted strain values. The result of this step is presented in Table 7-1. This table shows that the parameters vary significantly from one mix to another, which reflects the way that each material corresponds to the applied stress. The table also shows that the responses of some mixtures such as SWMA are less dependent on the temperature; this is consistent with the results presented in Figure 6-15 which showed that SWMA plastic strain was the least affected by the testing temperature. Table 7-1 also shows that WMRA is the most affected by the number of load applications; this is also consistent with the behaviour of this mix since it exhibited the highest near-constant deformation rate in the second stage of permanent deformation. Also, it was the only mix that reached the tertiary stage, unlike other mixtures. In conclusion, the optimised parameters reported in Table 7-1 reflect the way that each material responds to the applied load; and these parameters generally agree with the RLAT results of each mix.

Table 7-1. Rutting model parameters

Mix	a1	a2	a3
HMA	20.64673	0.09896	0.20399
SWMA	48.86511	0.00010	0.17808
CWMA	29.21154	0.04754	0.20972
WMRA	0.076169	1.173454	0.395365

The determined coefficients were then used to predict the plastic strain in the RLAT tests; Figure 7-1, Figure 7-2, Figure 7-3, and Figure 7-4 present the measured and predicted results using this model. These figures indicate that the model can fit very well the behaviour of mixtures that are sensitive to the testing temperature, such as in the case of CWMA and WMRA. This model, however, may not fit so well the behaviour of mixtures that are less sensitive to the testing temperature such as the in case of SWMA.

4. Lastly, the predicted strain was converted into permanent deformation using Eq. 7-1.

Accumulating plastic deformations at the end of a specific time period, such as the end of each month leads to predicting total rutting at the end of the service life of the pavement.

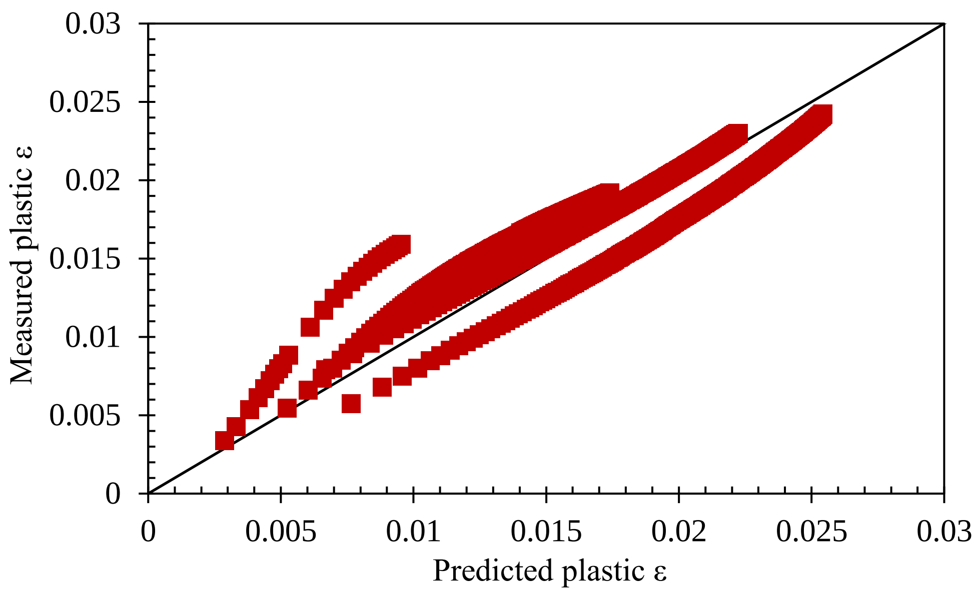


Figure 7-1. Measured and predicted RLAT results of HMA mix

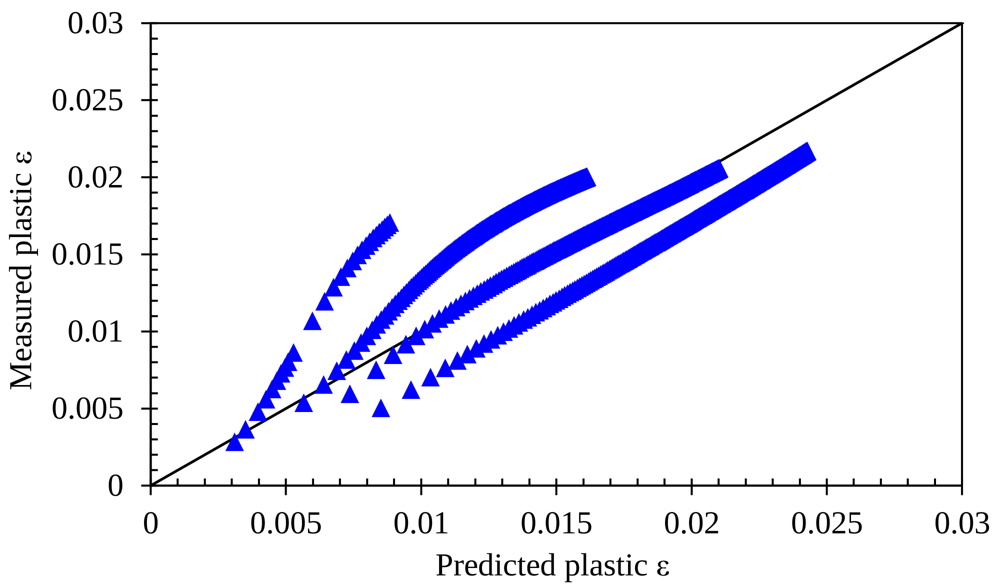


Figure 7-2. Measured and predicted RLAT results of SWMA mix

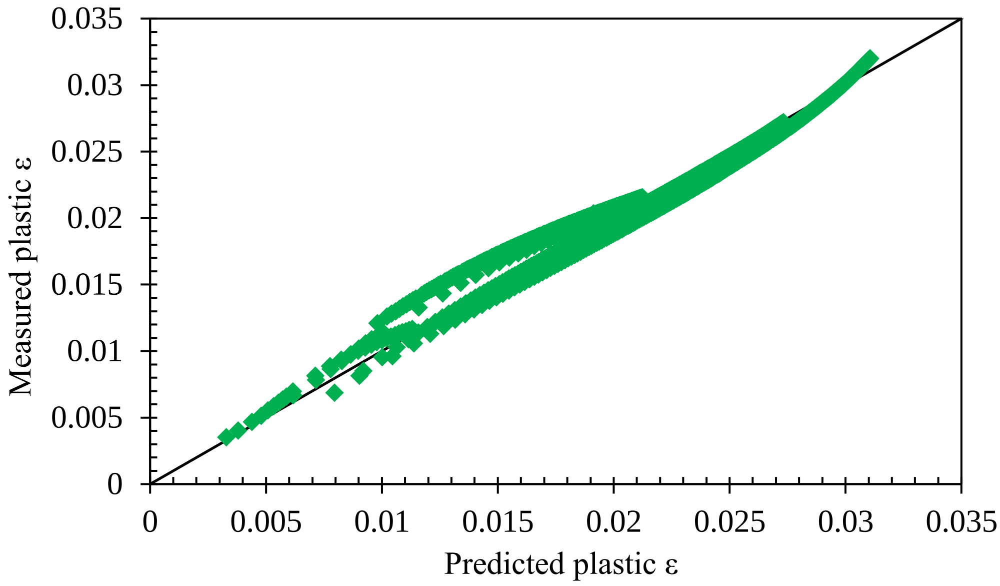


Figure 7-3. Measured and predicted RLAT results of CWMA mix

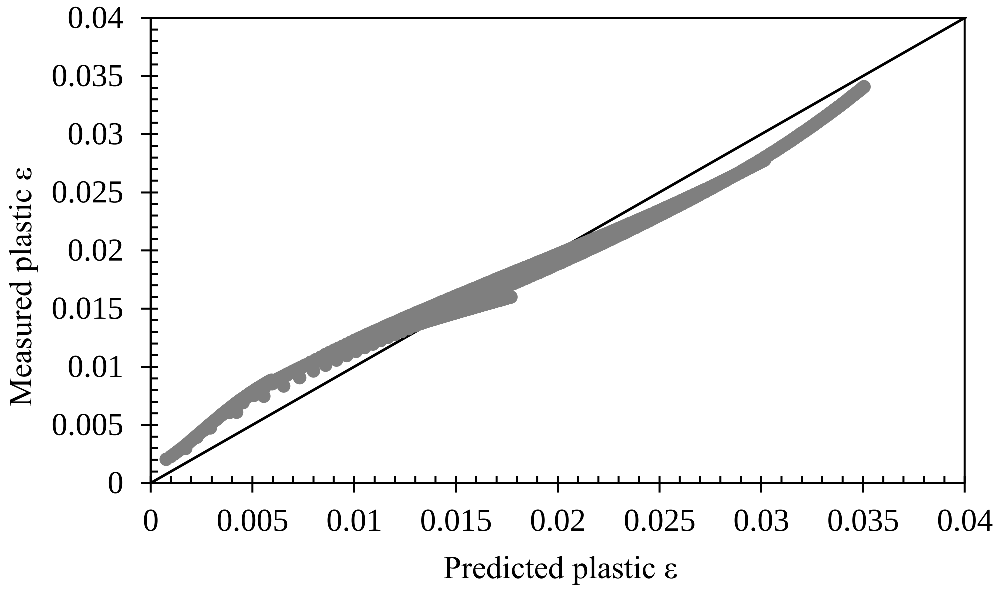


Figure 7-4. Measured and predicted RLAT results of WMRA mix

7.2.2 Fatigue Cracking Modelling

In the literature, fatigue cracking damage is often determined based on Miner's law (Huang, 2004) using the following equation:

$$FC_d = \sum_{i=1}^{l} \sum_{j=1}^{m} \frac{n_{i,j}}{Ni,j} \times 100\% \quad \text{Eq. 7-7}$$

where FC_d is the percentage of fatigue cracking damage, l is the number of time periods, m is the number of load groups, $n_{i,j}$ is the applied traffic volume in each period, and $N_{i,j}$ is the allowable number of load applications. In this study, a standard Equivalent Single Axle Load (ESAL) of 2×20 kN is considered in the analysis, which means equation Eq. 7-7 can be solved by merely dividing the expected ESALs by N. However, determining an accurate allowable number of load applications has a vital impact on the results of this analysis. If N is overestimated, then fatigue cracks will appear in the analysed pavement earlier than expected; if N is underestimated, then the pavement service life with respect to fatigue cracking will be longer than expected. In general, two models have been widely applied to determine asphalt fatigue life; the first is the Asphalt Institute's model (The Asphalt Institute, 1981), which is formulated as follows:

$$N = C \times k_1 \times (\frac{1}{E})^{k_2} \times (\frac{1}{\varepsilon_t})^{k_3} \quad \text{Eq. 7-8}$$

where N is the fatigue life in terms of number of load applications, k_1, k_2, k_3 are laboratory regression coefficients, C is a fatigue life laboratory-to-field shift factor, E is asphalt stiffness, and ε_t is the horizontal strain at the bottom (for bottom-up cracking) or surface (for top-down cracking) of an asphalt layer. The second model only relates fatigue life to the applied tensile strains, as follows:

$$N = C \times k_1(\varepsilon_t)^{k_2} \quad \text{Eq. 7-9}$$

where all abbreviations are as determined before. The difference between these models is that the former includes an asphalt stiffness effect on fatigue life, whereas the latter does not consider that factor. But the fatigue cracking results presented in the previous chapter proved that asphalt fatigue life is significantly affected by stiffness, evident in that the fatigue life of all mixtures generally dropped when tested at 0 and 10°C in comparison with those that resulted at 20°C. Accordingly, the model presented in Eq. 7-8 was implemented in this study. Model parameters were optimised for the studied mixtures using the optimisation function Solver. Table 7-2 lists the optimised coefficients and the coefficients of regression of the studied mixtures; it can be seen that R^2 values are at least 0.83 indicating an acceptable goodness of fit. Figure 7-5, Figure 7-6, Figure 7-7 and Figure 7-8 present the measured against the predicted fatigue life values. Despite the scattering of fatigue cracking data, these figures suggest that the predicted values have a reasonable correlation with the measured ones since both measures are almost symmetrically distributed around the equality line.

After determining the allowable number of load applications, fatigue damage can be determined in an incremental process. Basically, in each analysis period, the critical tensile strain is calculated based on the properties of the analysed pavement, then it is used to determine the allowable number of load applications using Eq. 7-8; fatigue damage for this period is then calculated using equation Eq. 7-7.

Table 7-2. Fatigue cracking model coefficients and coefficients of regression

Mix	K_1	K_2	K_3	R^2
HMA	8.801E-05	4.832	2.266	0.96
SWMA	1.042E-07	4.961	1.635	0.83
CWMA	1.042E-07	4.834	1.553	0.83
WMRA	1.48E-07	5.696	2.345	0.92

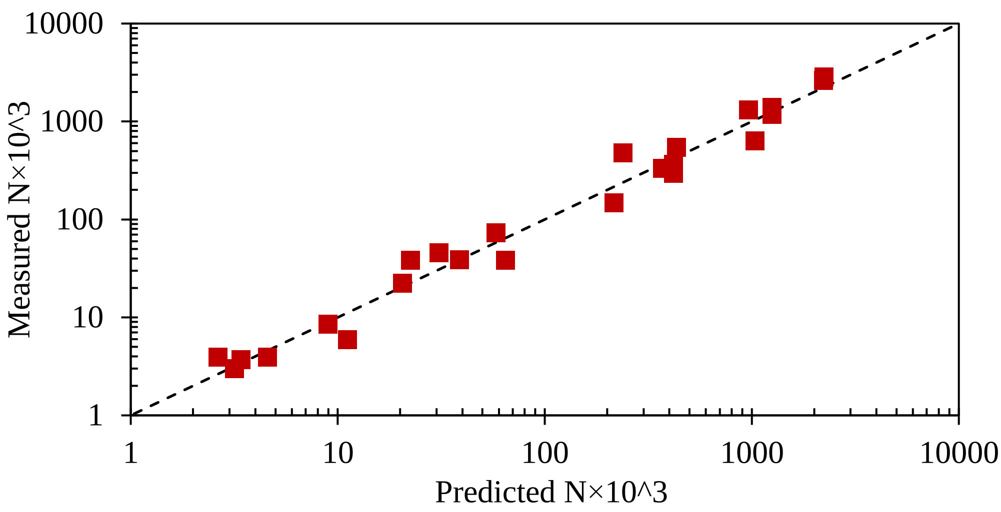


Figure 7-5. Measured versus predicted fatigue life of HMA

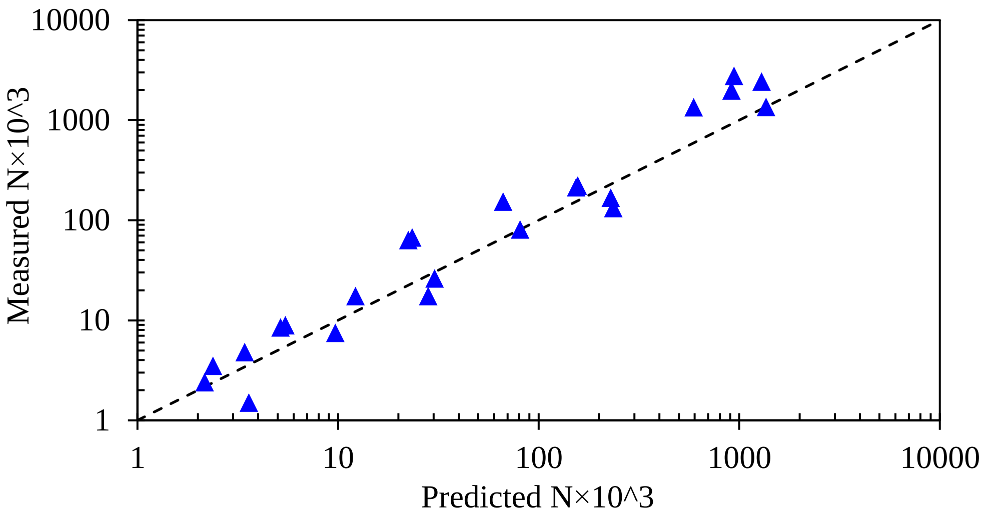


Figure 7-6. Measured versus predicted fatigue life of SWMA

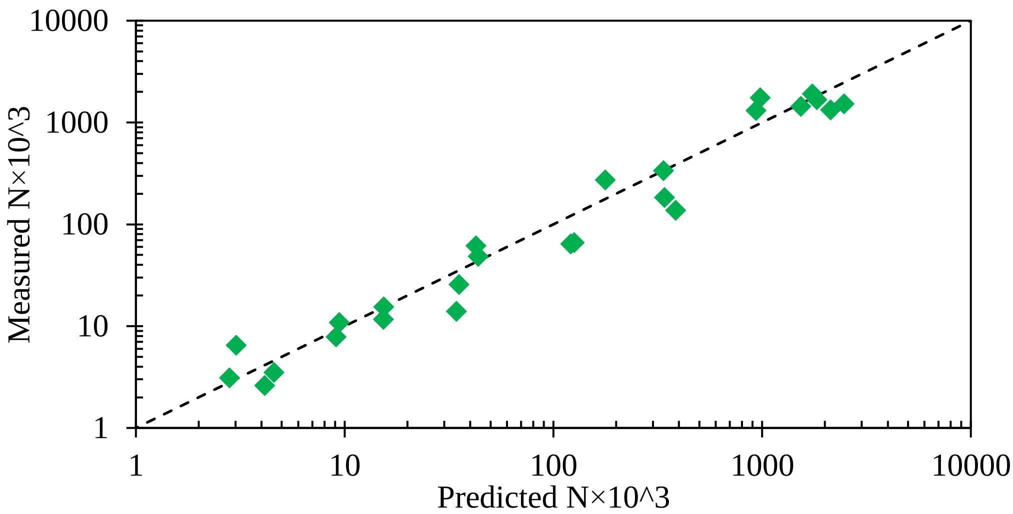


Figure 7-7. Measured versus predicted fatigue life of CWMA

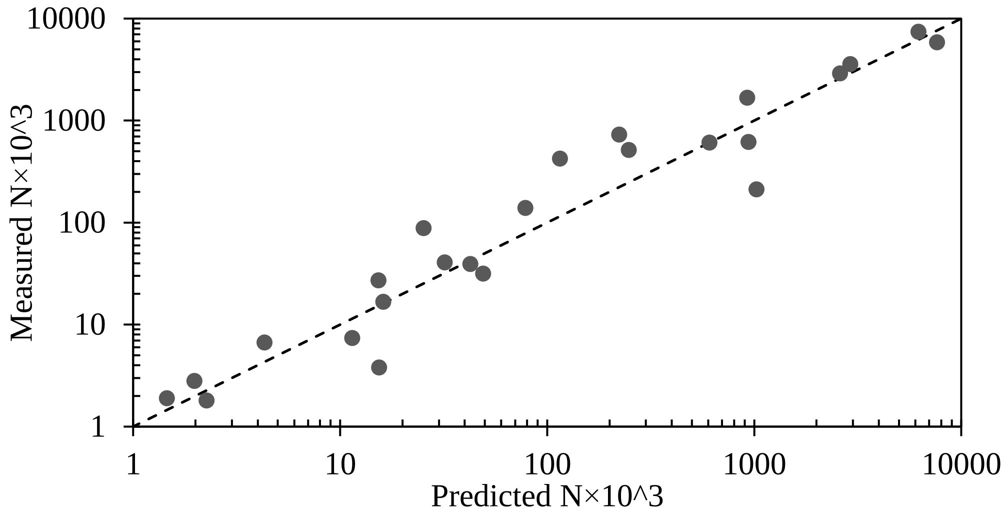


Figure 7-8. Measured versus predicted fatigue life of WMRA

7.2.3 Modelling of Pavement Temperature

Pavement temperature is one of the significant inputs to calculate the dynamic modulus of asphaltic materials (Kim et al., 2011b). The critical relationship between these parameters arises from the strong relationship between bitumen viscosity and temperature; the higher the pavement temperature, the lower the viscosity and the lower the dynamic modulus of asphalt and vice versa. This means that all pavement distress types are significantly affected by the pavement temperature, so it must be accurately determined if an accurate pavement performance prediction is required.

Pavement temperatures are generally calculated from air temperatures. This relationship depends on several factors, such as the geographical location of the pavement, climatic conditions, and thermal properties of asphalt. Several models can be found in the literature to predict pavement temperature from air temperature measurements. In this study, the models suggested by (Mohseni, 1998) and used in the Superpave mix design method were implemented, as follows:

$$\mathrm{LowTemp_{pav}} = -1.56 \times T_{air} - 0.004 \times Lat^2 + 6.26 \log_{10}(H + 25) \quad \text{Eq. 7-10}$$

$$\text{HighTemp}_{\text{pav}} = 54.32 + 0.78 \times T_{air} - 0.0025 \times Lat^2 - 15.14 \times \log_{10}(H + 25) \qquad \text{Eq. 7-11}$$

where $Low\text{Temp}_{\text{pav}}$ and $High\text{Temp}_{\text{pav}}$ are low and high pavement temperatures in °C, T_{air} air temperature °C, Lat is the latitude of the section in degrees, and H is the depth to pavement surface in mm. These models were originally developed to calculate bitumen performance grade (PG) in the Superpave mix design method by entering the lowest temperature and the average of the highest seven air temperatures. Nevertheless, they can be used to calculate pavement temperatures at different depths from air temperatures. These models result in two pavement temperatures from one air temperature, one is lower, and one is higher than the air temperature. If the air temperature is rising, then the pavement temperature is lower, and the pavement is absorbing energy in this case. If the air temperature is dropping, then the pavement temperature is higher, and the pavement is losing energy in this case. Accordingly, it can be assumed that in every month, the pavement temperature can be represented by three temperatures that can be calculated from the average monthly air temperature, one is the lower, one is the higher and the last one is the average of these temperatures.

On the other hand, air temperatures have to be modelled in order to predict pavement temperatures throughout the year. Since the air temperature periodically changes over time, then it is logical to assume that the air temperature can be modelled as a sine wave against time. To prove this assumption, air temperature data from Nottingham city were considered in this study; the mean monthly temperature data were downloaded from a UK weather forecasting website (UK

Forecast, 2019) and are presented in Figure 7-9. These data were then modelled using the following trigonometric function:

$$\mathrm{Temp}_{\mathrm{time}} = \mathrm{a} + \mathrm{b} \times \sin\left(2\pi \times \frac{mn}{12} + \theta\right) \qquad \text{Eq. 7-12}$$

where $\mathrm{Temp}_{\mathrm{time}}$ is the predicted temperature as a function of time, a and b are fitting coefficients, mn is the time in months, and θ is a shift angle to fit the sine wave to the temperature data. The optimisation function Solver was used to calculate the model parameters; the real data and the predicted temperatures are presented in Figure 7-9. It can be seen there is a very good correspondence between the measured and predicted data, which means this assumption can be considered valid; therefore, it was followed in this study to predict air temperatures. The predicted temperatures based on this method can then be converted into pavement temperatures using Eq. 7-10 and Eq. 7-11. Figure 7-10 presents the predicted air and pavement temperatures for one year. It must be clearly stated here that the suggested way to predict air and pavement temperatures is only an approximate method; if this method is to be used in real pavement design problems, then it must be calibrated and validated before any implementation.

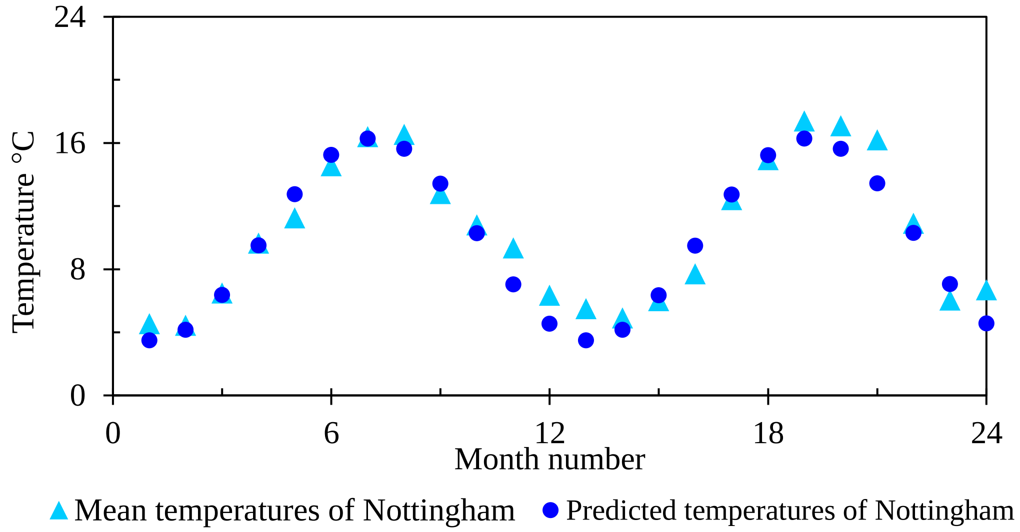


Figure 7-9. 2015 and 2016 mean monthly temperatures of Nottingham

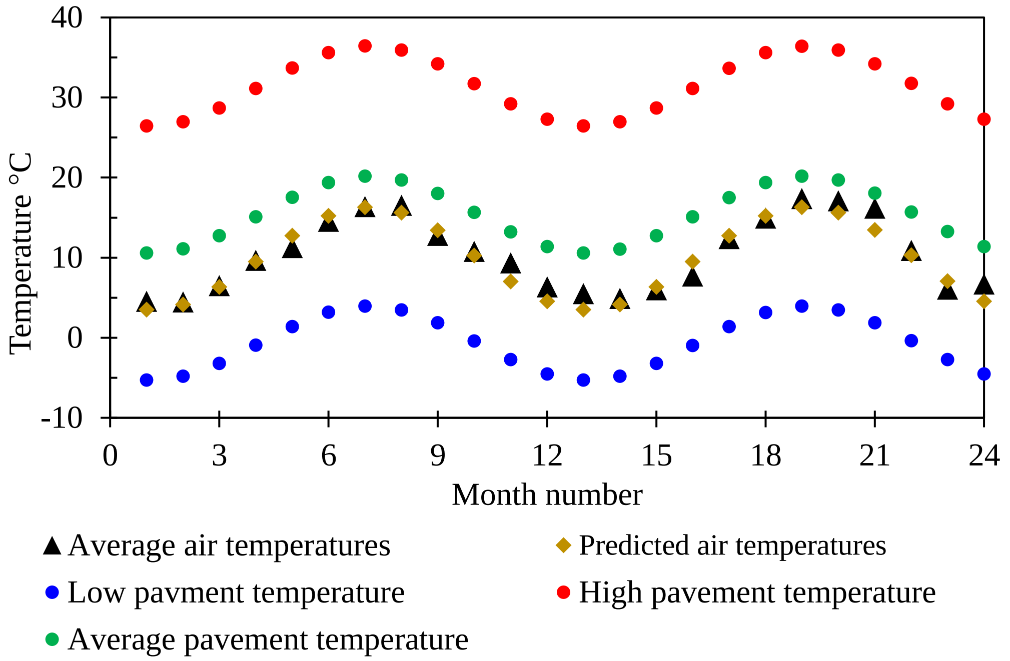


Figure 7-10. Air and pavement temperatures

7.3 Performance Prediction of a Typical Asphalt Pavement Structure

7.3.1 Pavement Structure Description

Typically, asphalt pavement structures in the UK consist of compacted subgrade, sub-base, bound base, and asphalt surface (Brown and Brunton, 1986), as illustrated in Figure 7-11. To analyse this system and predict its mechanical performance, the following assumptions were made:

1. All unbound layers are linear, elastic, and isotropic.
2. All asphalt layers are linear, viscoelastic, and isotopic.
3. All layers are fully bonded to ensure the continuity of stress and strain at layer interfaces.

Based on these assumptions, the suggested pavement structure was analysed; pavement properties are presented in Table 7-3. The foundation (sub-base and subgrade) of this structure corresponds to foundation class 2 in the UK, which is a typical foundation for roads that can convey medium or high traffic volumes (Nunn, 2004). For base and surface layers, four alternatives were considered in the analysis; these were the four primary mixtures investigated in this study. Both the surface and base layers were assumed to be constructed of the same mix and the same properties since the studied mixture specifications can fit surface or asphalt base properties in the UK. The thickness of these materials was kept to the minimum allowable thickness of these layers; in this case, the mechanical performances of the studied mixtures can be compared and evaluated. The traffic volume considered in this study was determined in accordance with the Design Manual for Roads and Bridges in the UK, volume 7 “traffic assessment” (Dft, 2006). Raw traffic data were downloaded from the Department of Transport website (Dft, 2018); the A52, a major road in the East Midlands that connects Nottingham with other cities, was considered in the analysis; processing the raw traffic data resulted in a monthly traffic volume of 162,500 of 4×20 kN standard wheel load, which accounts for thirty-nine million ESALs during 20 years of service. Furthermore, the monthly traffic volume was distributed with respect to pavement temperatures. Since the monthly pavement temperatures are divided into three parts, low, high, and average temperature, then the monthly traffic volume is also divided into three portions, 10%, 35%, and 55%. The first portion is assumed to pass at the low pavement temperature; the second at the high pavement temperature, and the last portion at the average pavement temperature. The aim of this assumption is to match the time that traffic volume is applied with the time that pavement temperature is at a particular value. This procedure should improve the damage prediction process because pavement damage, in this case, is predicted based on traffic damage and pavement temperatures when traffic is passing.

The analysis process is illustrated in Figure 7-12. The analysis starts with determining asphalt temperatures at a depth of analysis in order to calculate asphalt stiffness from the E* master curve of the mix. Table 7-4 presents the E* model parameters of the studied mixtures; these values were calculated by a regression process using Matlab; the rest of the inputs were as shown in Table 7-3. After that, these data were fed into Kenlayer software to determine pavement response or strain values at the middle of the asphalt for rutting prediction and at the bottom of the asphalt for fatigue cracking prediction. In this study, Kenlayer software was used in an innovative method to calculate pavement response by utilising it as a subroutine in Matlab. Basically, Matlab determines the analysis inputs, then it sends these data to Kenlayer to run the analysis, then it extracts pavement strains automatically from Kenlayer. The vertical compressive strain levels at the middle of the asphalt were used to predict total permanent deformation using equations 7-1 to 7-5. The horizontal tensile strain at the bottom of the asphalt layer was used to predict the bottom-up fatigue cracking damage by equations 7-8 and 7-9. The horizontal tensile strain at the edge of the contact area at the pavement surface was used to predict the top-down fatigue cracking damage using the same equations. However, Kenlayer resulted in a horizontal compressive strain at the pavement surface rather than tensile strain. This was due to the fact that this software cannot calculate pavement response accurately at depths less than about 50 mm (Huang, 2004); other pavement analysis programmes such as JULEA also tend to exhibit the same problem when calculating pavement response at depths less 20% of the contact radius (Khazanovich and Wang, 2007). To solve this problem, Thom (2014) suggested the following equation to approximate the tensile strain at the load edge at the pavement surface:

$$\varepsilon_{surface} = \frac{P}{E} \times \sqrt{6 \times (1 + \upsilon)} \quad \text{Eq. 7-13}$$

where $\varepsilon_{surface}$ is the shear strain at the edge of the applied load, P is traffic load, E is stiffness of asphalt layer, and υ is Poisson's ratio. This equation can give a reasonable approximation to the tensile strain at the surface with negligible computational effort, which makes it suitable for implementation in pavement analysis software. Having determined the critical tensile strain levels, the fatigue cracking damage at the bottom of the base and top of the surface could be calculated, and these measures were used to quantify fatigue cracking using the following equations (NCHRP, 2004b):

$$FC_{bottom} = \frac{1}{60} \times \frac{6000}{1 + e^{(-2c_2' + c_2' \times log(D_{bottom}))}} \quad \text{Eq. 7-14}$$

$$FC_{top} = 10.56 \times \frac{1000}{1 + e^{(7.0 - 3.5 \times log(D_{top}))}} \quad \text{Eq. 7-15}$$

$$c_2' = -2.40874 - 39.748 \times (1 + H_{ac})^{-2.856} \quad \text{Eq. 7-16}$$

where FC_{bottom} is the bottom-up cracking percentage of the lane area, FC_{top} is the length of top-down fatigue cracking per unit length of road.

7.3.2 Results and Discussion

The analysis results are split into two sections. One presents the analysis of pavement response to the applied load, which depends on stiffness and thickness of pavement layers. The other presents pavement performance results which depend on strain levels calculated from the pavement analysis and the way that each material responds to the strain; as follows:

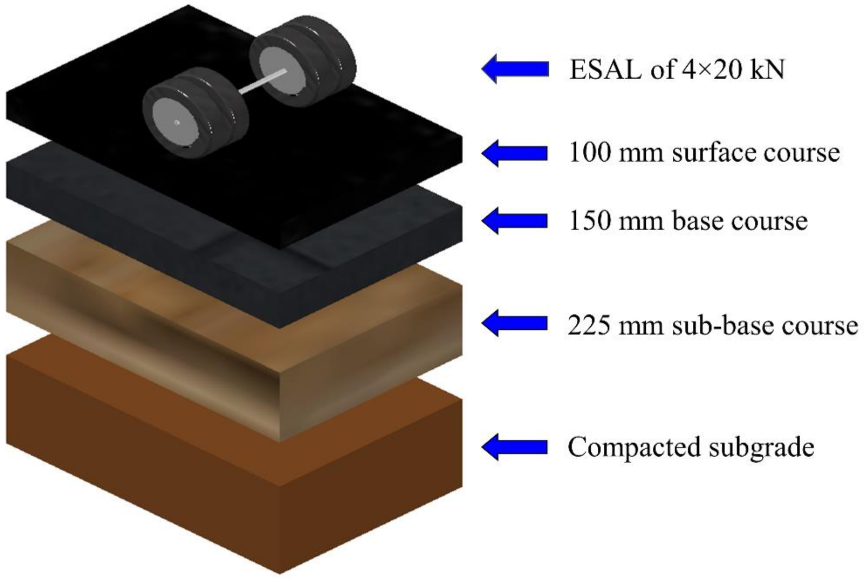


Figure 7-11. Pavement structure considered in the modelling process

Table 7-3. Pavement properties considered in the analysis

Material	Layer	Thickness mm	E MPa	Poisson's ratio
Alternative 1, HMA	Surface	100	From E* master curve (the surface and base layers of every alternative have the same dynamic modulus)	0.35
	Base	150		0.35
Alternative 2, SWMA	Surface	100		0.35
	Base	150		0.35
Alternative 3, CWMA	Surface	100		0.35
	Base	150		0.35
Alternative 4, WMRA	Surface	100		0.35
	Base	150		0.35
Sub-base		225	170	0.4
Compacted Sub-grade		-	50	0.4

Table 7-4. E* model parameters of the studied mixes

Mix \ parameter	δ	α	β	γ
HMA	2.7359	1.6806	-0.0009	0.7984
SWMA	2.8405	1.6092	-0.2437	0.7512
CWMA	2.7454	1.6703	-0.0701	0.8072
WMRA	2.6434	1.7593	-0.0521	0.6951

7.3.2.1 Analysis of Pavement Response

As mentioned earlier, pavement response depends on pavement properties, including pavement temperature, which has a critical impact on the overall response. This means that in hot weather strain levels in the pavement are larger than those expected in cold weather; this is because pavement stiffness in the hot weather is lower than that in the cold weather. For this reason, pavement response results were divided into two sets. One presents pavement response in the coldest weather in Nottingham, which occurs in January. The second set presents pavement response in the hottest weather in Nottingham, which occurs in July.

Figure 7-13 and Figure 7-14 present the horizontal tensile strain levels in both cold and hot weather. It can be seen that there is about five times increase in the strain levels when analysing pavement response in the hot weather compared to cold. Moreover, both figures show that SWMA exhibited the lowest tensile strain levels, which is reasonable since Sasobit increased bitumen and mix stiffness. HMA and CWMA responses were comparable in both conditions. WMRA exhibited the highest strain levels, especially at high temperatures.

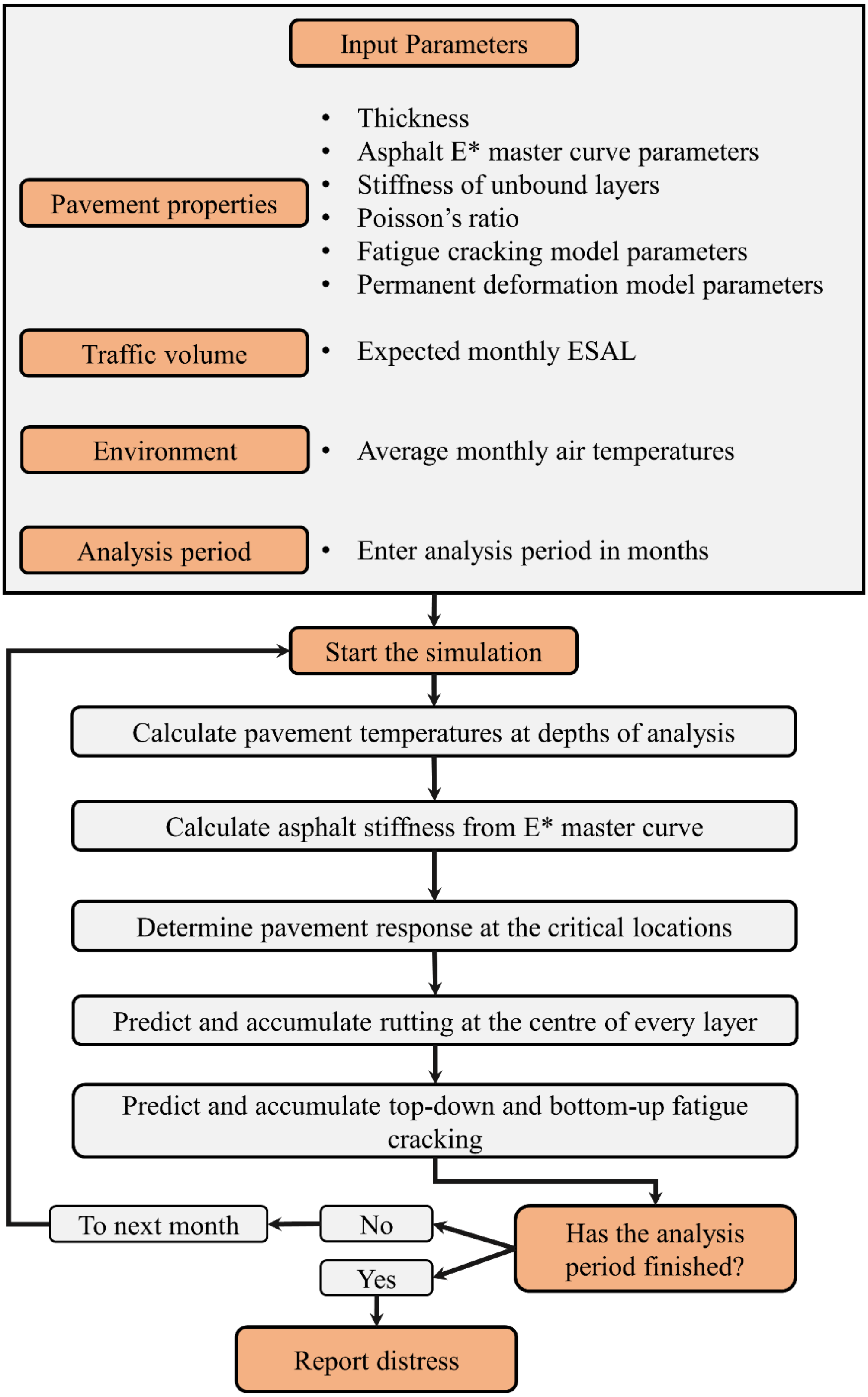


Figure 7-12. A flowchart explaining the method followed to predict pavement performance and its service life

This result may be explained by the decrease in WMRA stiffness at high temperatures where the soft binder starts to approach its critical high temperature. On the other hand, Figure 7-15 and Figure 7-16 present the compressive strain levels in both cold and hot weather. These figures show the same trend as the tensile strain results, low strain levels in the case of SWMA, comparable levels in the case of HMA and CWMA, and high levels in the case of WMRA. Regarding the tensile strain at the pavement surface, the results are shown in Figure 7-17 and Figure 7-18. These figures show the same trend of material response again.

These results suggest that increasing asphalt stiffness can improve pavement performance by reducing strain levels. A careful balance is required, however, because increasing stiffness can increase asphalt susceptibility to fatigue cracking. Furthermore, these results suggest that Sasobit can improve pavement performance due to reducing pavement strain levels, whereas WMRA may exhibit the worst performance due to the high strain levels exhibited by this material. However, these conclusions are based on material response to the applied loads; the way that the material responds to the developed strain should be considered before drawing a final conclusion about the performance of these mixtures.

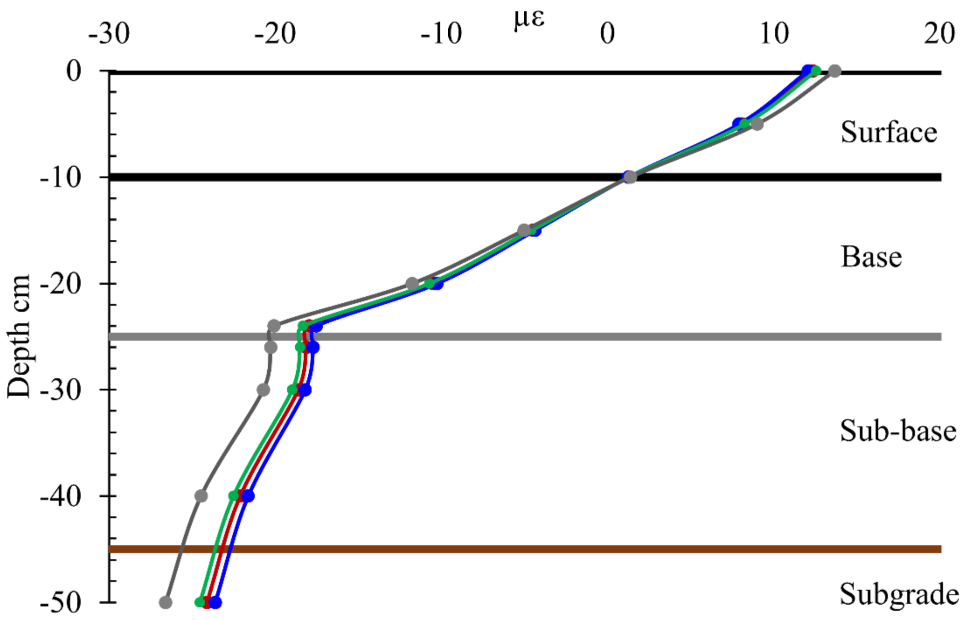


Figure 7-13. Horizontal tensile strain levels in the cold weather

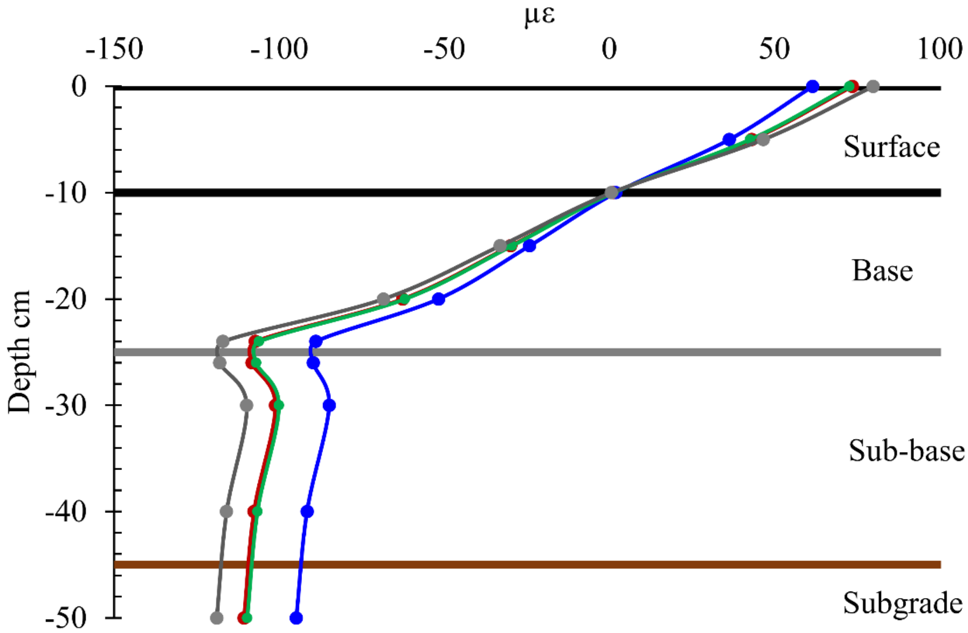


Figure 7-14. Horizontal tensile strain levels in the hot weather

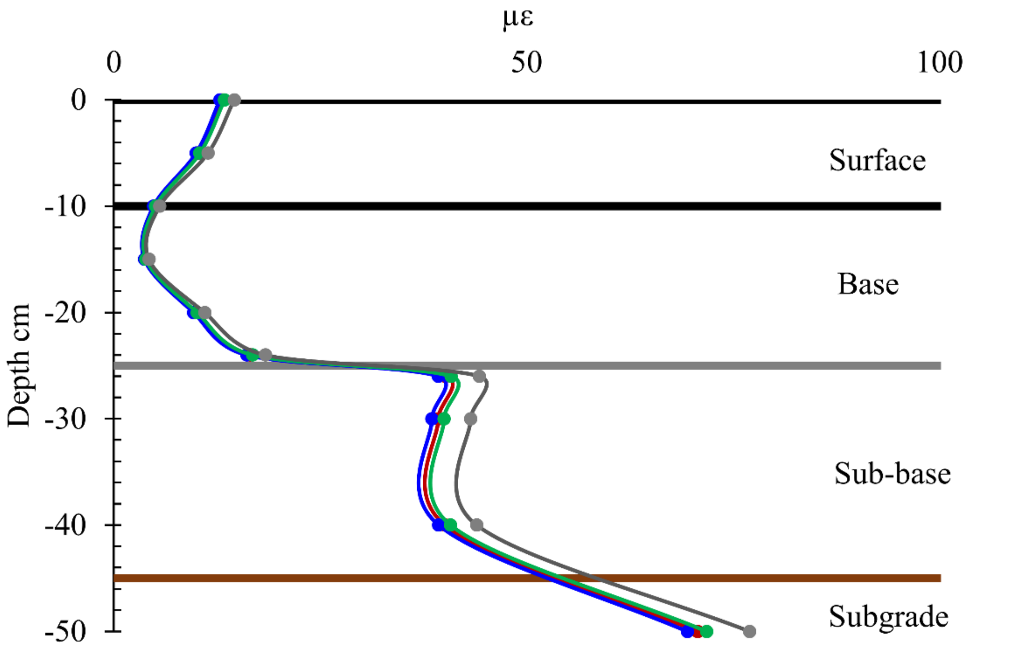


Figure 7-15. Compressive strain levels in the cold weather

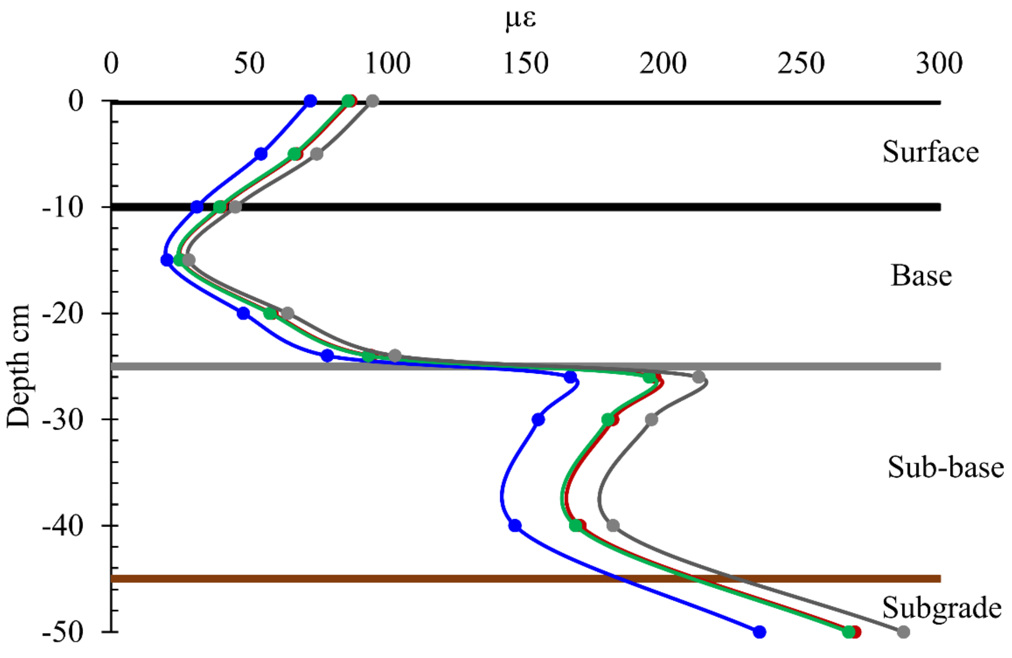


Figure 7-16. Compressive strain levels in the hot weather

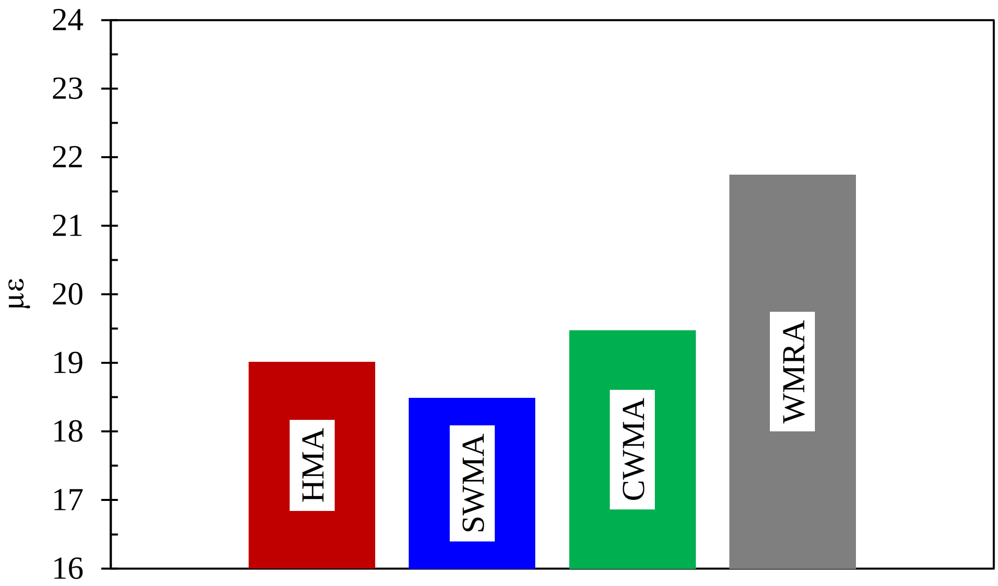


Figure 7-17. Estimated tensile strain at pavement surface in the cold weather

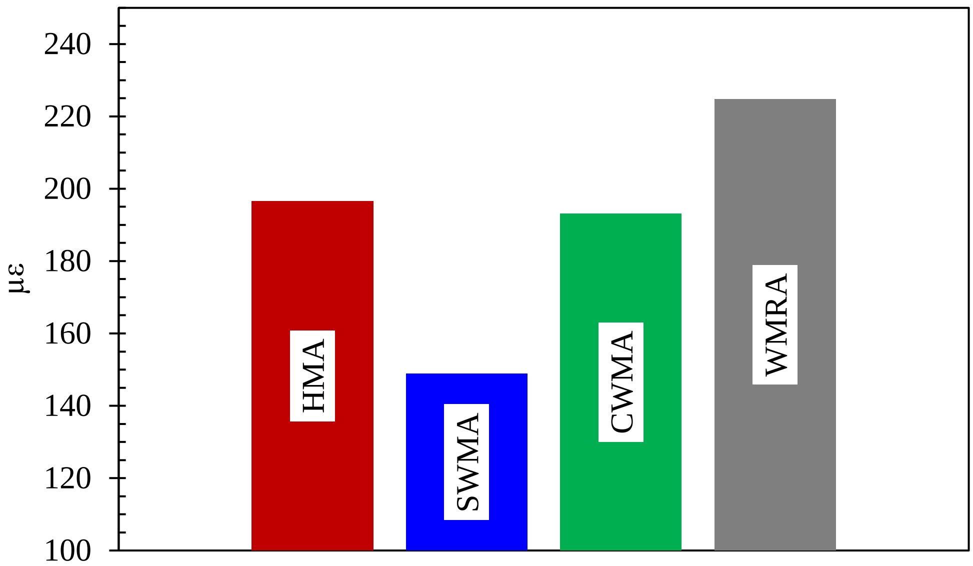


Figure 7-18. Estimated Tensile strain at pavement surface in the hot weather

7.3.2.2 Analysis of Pavement Performance

Based on the prescribed performance prediction approach, the studied mixture performance in terms of bottom-up cracking, top-down cracking, and asphalt rutting was predicted, as shown in Figure 7-19, Figure 7-20, Figure 7-21. The first figure shows the predicted bottom-up fatigue cracking percentage of the studied mixtures after twenty years of service. This figure indicates that SWMA and

WMRA outperformed HMA by reducing the percentage of cracking. This result can be explained by the reduced strain levels in the case of SWMA and by the improved fatigue life in the case of WMRA. CWMA, however, exhibited a higher percentage of cracking due to the reduced fatigue life of this mix. Nevertheless, the threshold value for this kind of distress is 20% in the MEPDG. This means that all of the studied mixtures perform acceptably with respect to this performance measure with this pavement structure. Also, this type of failure is not common in thick pavements (Thom et al., 2002). Top-down cracking, however, is an important measure that causes immediate degradation in the pavement serviceability as the cracks appear at the pavement surface. The predicted top-down cracking results are presented in Figure 7-20. This figure shows that there are two possible effects that can increase the top-down cracking amount in asphalt. The first is by reducing fatigue life of asphalt such as in the case of CWMA; the second is by using an asphalt mix with reduced stiffness which leads to increasing tensile strain levels at the pavement surface, such as in the case of WMRA. Despite WMRA exhibiting excellent fatigue cracking performance, the reduced stiffness of this material led to increasing tensile strain levels at the pavement surface, as shown in Figure 7-18, which resulted in increased top-down fatigue cracking. On the other hand, SWMA clearly improved pavement performance and reduced the levels of this distress type mainly by reducing the tensile strain at the pavement surface.

The permanent deformation results of asphalt are shown in Figure 7-21. This figure indicates that SWMA outperformed other mixtures. This was an expected result due to the increased stiffness and the improved rutting performance of this mix. The control mix performed similarly to the SWMA with relatively increased rutting, about 20%. The CWMA mix exhibited a clearly increased plastic deformation in comparison with HMA, about 1.8 times the HMA rutting; this result is consistent with RLAT and MSCR results which showed that Cecabase increases asphalt susceptibility to plastic deformation. Lastly, the WMRA mix exhibited significantly higher rutting than other mixtures. This result can be explained by two reasons; firstly, by the reduced stiffness of this mix, which leads to increasing the compressive strain in the mix, eventually increasing the plastic deformation. Secondly, to a lesser extent, by the poor rutting performance of this mix, especially at high temperatures.

Furthermore, all of the predicted permanent deformation results were consistent with the RLAT test results. This indicates that the developed rutting predicting process can be considered valid for the studied materials and can be used in predicting permanent deformation of asphalt pavements. The need for calibration, however, prevents the direct implementation of these models. Accordingly, it is recommended to calibrate these models before any usage in predicting rutting of real asphalt roads.

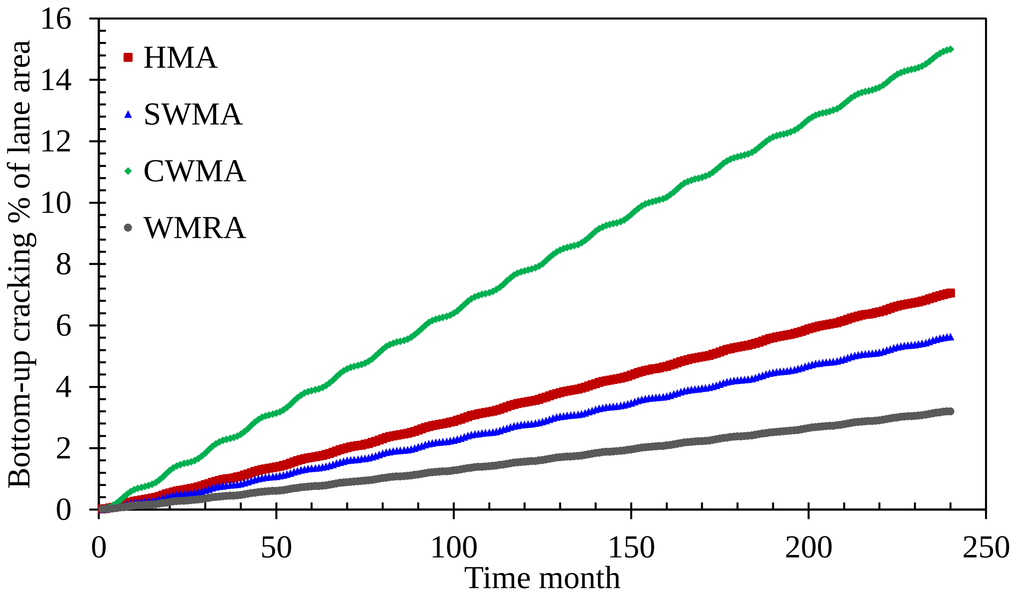


Figure 7-19. Predicted bottom-up fatigue cracking

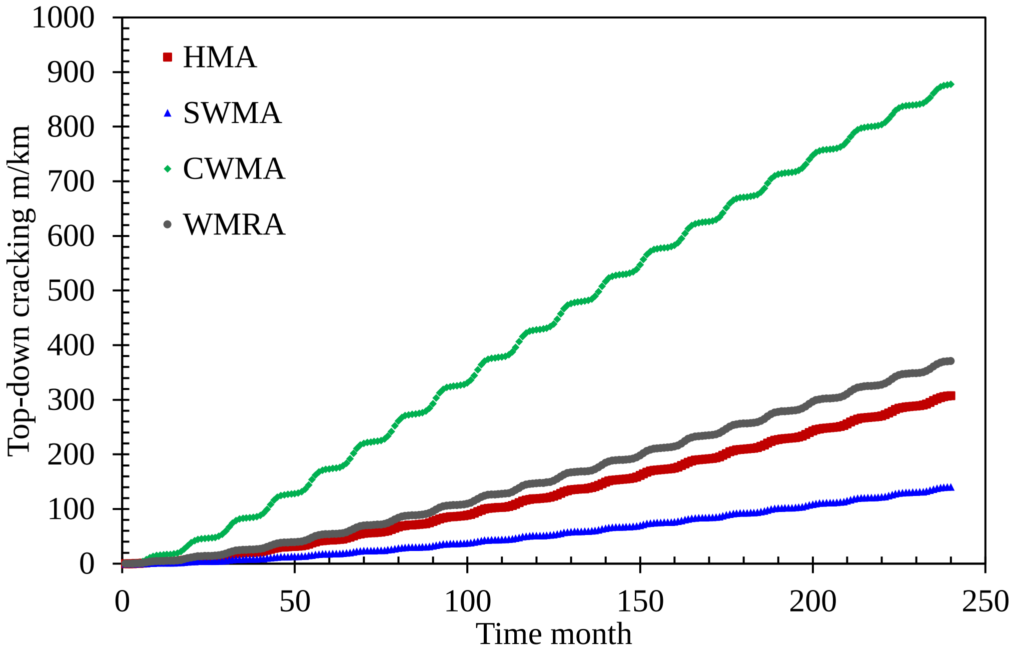


Figure 7-20. Predicted top-down fatigue cracking

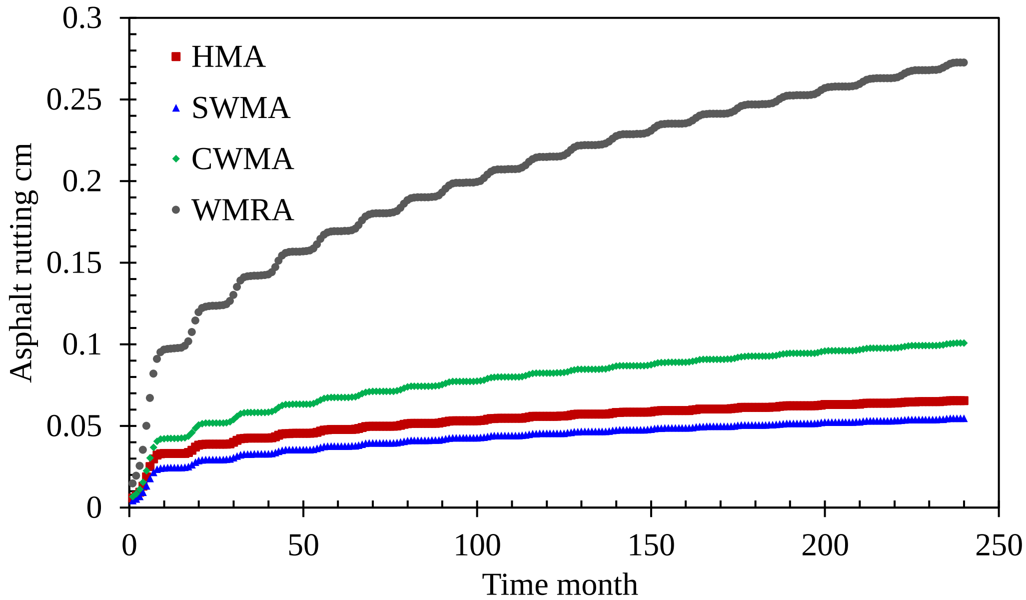


Figure 7-21. Predicted rutting in asphaltic layers

7.4 Summary and Conclusions

A mechanistic-empirical pavement performance prediction method has been suggested in this chapter to predict the performance of the studied mixtures. The method utilises the MEPDG approach to predict pavement performance but using the properties of the studied mixtures. The properties used in the analysis were dynamic modulus, permanent deformation, and fatigue cracking life. E* model parameters were calculated from the dynamic modulus testing using the two-point bending method; rutting model parameters were calculated from the RLAT test results at different temperatures by an innovative method; fatigue cracking life model parameters were determined from fatigue cracking tests at different temperatures using the two-point bending method. These data were integrated into a Matlab code that predicts performance measures over time. A typical pavement structure was considered in the analysis, the structure consisting of sub-base and subgrade layers and four alternatives of asphaltic base and surface layers that correspond to the four studied mixes. The code basically sends the required inputs to Kenlayer software to determine pavement response at predetermined locations in the pavement, then it predicts and accumulates distress levels in a timewise process.

Within the scope of this chapter, the following conclusions can be drawn:

1- The results demonstrated that pavement performance can be affected by two factors: first, by changing pavement response due to changing stiffness of pavement layers; second, by the way that the material responds to the applied loads, which depends on the material properties such as rutting resistance and fatigue cracking resistance.

2- The predicted bottom-up fatigue cracking results indicated that WMRA outperformed other mixes due to the improved fatigue life of this mix; SWMA performed better than the control mix by reducing tensile strain levels at the bottom of the base layer; CWMA showed the highest level of cracking due to the poorer fatigue characteristic of this mix.

3- Concerning top-down fatigue cracking performance, the results revealed that CWMA is the most susceptible mix to this distress, followed by WMRA. This result could be explained by the poorer fatigue characteristic in the case of CWMA and the reduced dynamic stiffness in the case of WMRA. SWMA, however, outperformed other mixes due to its increased stiffness, which reduced the tensile strain levels at the pavement surface.

4- The permanent deformation performance results indicated that SWMA outperformed other mixtures due to two factors, first by reducing the vertical compressive strain in asphalt, second by the improved rutting performance of this mix. CWMA exhibited increased permanent deformation due to the poorer rutting resistance properties of this mix. Lastly, WMRA exhibited the highest rutting among all the mixtures due to the increased vertical compressive strain of this mix and the poorer rutting resistance properties of this mix at high temperatures.

5- The rutting prediction model used fitted efficiently the performance of mixtures that are sensitive to testing temperatures, such as CWMA and WMRA. However, if the model is used to fit performance of a mix that is barely sensitive to the temperature, such as SWMA, then the model may not fit the data very well. This point highlights the need to develop different rutting prediction methods to predict the permanent deformation of that kind of asphalt mixture.

6- Based on these results, it can be concluded that all of the studied mixtures can perform acceptably as asphaltic base materials with very good rutting and bottom-up cracking performance. As surface materials, however, the control mix and SWMA are better alternatives than other mixes due to the degraded performance of the other mixes with respect to the top-down fatigue cracking measure, which is one of the dominant distress types in thick pavements.

Chapter 8: Summary, Conclusions and Recommendations

8.1 Summary

WMA and WMRA mixtures have received a great deal of attention from highway agencies and asphalt organisations due to the ever-increasing concerns of environmental pollution and economic saving. Several studies have indicated that the use of WMA additives could reduce GHG emissions and energy consumption during the production phase of asphalt. Furthermore, incorporation of RAP with WMA has been reported to be one of the most sustainable alternatives to enhance the sustainability of the asphalt industry. Despite the benefits of these asphalt types, there is significant uncertainty regarding the production conditions, mechanical performance, and durability of these mixtures, which hinders the implementation of these types especially when high dosages of RAP are incorporated.

In this study, different WMA mixes with and without a high dosage of RAP (50%) were comprehensively investigated. Two of the commonly used WMA additives, namely Sasobit and Cecabase, were used and researched at binder and mixture levels. At the binder level, different dosages of these additives were mixed with the base bitumen and the empirical and rheological properties of the resulting binders were tested and monitored at different ageing levels. At the mixture level, the production temperatures of WMA were first optimised, then the mechanical performance in terms of rutting, fatigue cracking, and durability was investigated.

Regarding the WMRA mix, this mix was designed in two stages. Firstly, the penetration grade of the required virgin binder was determined based on the properties of the RAP binder and logarithmic relationship between binder proportions and penetration grades. Secondly, the production conditions, more specifically mixing time and temperature of this mix, were optimised based on the volumetric and mechanical performance characteristics of this mix compared to the control mix. After optimising these conditions, mechanical performance in terms of rutting, fatigue cracking, and durability was also investigated.

Moreover, based on rutting and fatigue cracking properties of the designed mixtures, the performance of these mixes was predicted by implementing the MEPDG method. The RLAT results at different high temperatures were utilised to optimise the permanent deformation model parameters. The two-point bending fatigue cracking results were utilised to optimise the fatigue life model parameters. Finally, these models were implemented in predicting rutting, top-down cracking, and bottom-up cracking. The prediction process involved modelling the pavement as a multilayer system where the unbound layers act as an elastic medium and the asphalt layers act as a viscoelastic medium. Based on this assumption pavement response was calculated by Kenlayer software and used in the derived rutting and fatigue life models to predict permanent deformation and fatigue cracking distress of a typical asphalt pavement in the UK.

8.2 Conclusions

The principal aim of this research was to investigate the effects of production temperatures on WMA and WMRA performance and to optimise these conditions in order to produce mixtures with equivalent or improved performance to a control mix. Reducing production temperatures of asphalt without compromising the performance would definitely reduce GHG emissions and reduce the energy required to produce asphalt. Incorporating RAP into the mix would maximise these benefits by saving the fuel required to produce bitumen and aggregate raw materials and saving GHG emissions produced during this process. To achieve these aims, comprehensive experimental work and fundamental analysis were carried out; the main conclusions that can be drawn from this research are addressed as follows:

8.2.1 Binder Level Conclusions

1. The Sasobit reduces the penetration, increases the softening point, and increases the complex shear modulus of the base binder. Also, this effect is directly related to the additive dosage; the higher the additive concentration, the larger the effect and vice versa.

2. The Sasobit increases the complex shear modulus and reduces the phase angle, which leads to increasing the elastic behaviour of bitumen.

3. MSCR and LAS test results indicated that the Sasobit can enhance rutting resistance and can improve fatigue cracking resistance.

4. Although the Sasobit increases the shear modulus, after the combined short-term and long-term bitumen ageing stages, the G* of the base binder was comparable to the binder modified with Sasobit indicating this additive can retard the bitumen ageing process.

5. The Cecabase did not show an apparent effect on bitumen empirical and rheological properties. However, this point may be valid for the dosages used for this additive; increasing the additive concentration beyond the used levels may show a different conclusion.

6. The Cecabase can make bitumen relatively susceptible to rutting and fatigue cracking, as indicated by MSCR and LAS test results.

7. The uncoupling analysis of the creep element from the recovery element of the MSCR test results revealed that the Sasobit significantly reduced bitumen creep and improved bitumen recovery, whereas the Cecabase slightly increased bitumen creep and reduced bitumen recovery. This indicates that Sasobit can improve rutting resistance by reducing the creep and improving the creep recovery, whereas the Cecabase can increase rutting protentional due to its negative impact on bitumen creep and creep recovery properties. Furthermore, this conclusion shows the necessity of uncoupling creep and creep recovery elements to derive a better conclusion from MSCR results.

8. Although the Cecabase did not change bitumen properties, it significantly retarded the G* increase and phase angle decrease after the combined ageing process of bitumen, which indicates that the additive retards bitumen ageing.

9. Although Sasobit is supposed to be a viscosity reducer, the rotational viscosity results indicated that this additive only slightly reduced bitumen viscosity, which leads to the conclusion that this WMA additive is not only a viscosity reducer.

10. The Cecabase did not alter bitumen rotational viscosity, which was an expected result since this additive is a chemical WMA additive.

11. The disagreement in the ranking of bitumen fatigue performance between Superpave fatigue cracking parameter and LAS results indicated the unsuitability of the Superpave parameter to rank bitumen fatigue cracking performance for WMA modified binders.

12. All bitumen characterisation results indicated that the impact of WMA additives on bitumen properties should be studied beyond the limit of linear viscoelasticity since neither empirical nor rheological properties were able to correctly capture the effect of the additives on bitumen performance.

13. Binder design based on blending laws is quite a critical step in the mix design of WMRA. In this regard, the penetration linear, logarithmic relationship showed very good results in calculating the required virgin binder grade based on the target binder grade and mixing proportions. Satisfying this relationship can guarantee comparable viscosity and Superpave parameter properties to the target binder grade. However, this result is valid only at the binder blending level; at the mixture level, the results of this equation may or may not be suitable depending on RAP binder percentage and properties.

14. No binder performance evaluation tests were conducted on the RAP binder and virgin binder blends because these tests will not reflect the actual performance of RAP containing asphalt due to the DoB problem between the two binders. This makes assessing performance of asphalt type from binder testing inaccurate or even invalid.

8.2.2 Mixture Level Conclusions

8.2.2.1 WMA

1. Aggregate coating can be studied and quantified at different mixing temperatures using image processing techniques.

2. The developed image processing method is able to capture the impact of WMA additives on aggregate coating and is capable of distinguishing the effect of Sasobit from Cecabase on aggregate coating results.

3. Reducing mixing temperatures of asphalt requires additional mixing time to achieve full coating regardless of whether using WMA additives. In the case of using the additives, however, the additional mixing time was less than the case of not using the additives, indicating an improved coating when the additives were incorporated.

4. Based on aggregate coating results, both the additives reduced mixing temperature of asphalt from 155°C, the standard mixing temperature of 40-60 pen binder, to 145°C. This reduction can be further increased by mixing for a longer time; in this study, one minute was suggested, which resulted in mixing CWMA at 130° and SWMA at 135°C. However, this conclusion is based on mixing in a bucket mixer, mixing at an asphalt plant may require different mixing times.

5. By comparing the bitumen viscosity results with aggregate coating results, it can be concluded that the equi-viscous approach was not valid in designing mixing temperatures of WMA.

6. Since binder viscosity results of Sasobit modified binders, in particular, the 2%SMB were quite comparable to the base binder, then it can be concluded that the effect of this additive can be a viscosity reducer and binder-aggregate friction reducer as well rather than a pure viscosity reducer. This can explain the improved aggregate coating of SWMA mix.

7. ITSM and ITS results confirmed that SWMA performance is significantly correlated with mixing temperature, the higher the mixing temperature, the higher these measures and vice versa. This point indicates that designing the mixing temperature of a WMA mix should be conducted based on multi-criteria such as aggregate coating and performance rather than equi-viscous, equi-torque, or even aggregate coating alone.

8. Integrating ITSM and ITS with aggregate coating results showed that SWMA can be mixed at 135°C with equivalent ITS and increased ITSM and sufficient aggregate coating. On the other hand, CWMA performance was independent of mixing temperatures, with an insignificant reduction in ITSM and ITS indicators; therefore, mixing temperature of this mix could be decided based on an aggregate coating criterion which resulted in mixing at 130°C to achieve acceptable aggregate coating.

9. By comparing the effect of WMA additives on mixing temperatures and compaction temperatures, it can be concluded that the additives are more beneficial in enhancing the compaction process than the mixing process of asphalt.

10. The dynamic modulus test results indicated that Sasobit increases E* at intermediate and high temperatures, whereas Cecabase does not significantly change E* of asphalt in that range of temperatures. At low temperatures, E* results of all mixes were comparable due to the high stiffness of bitumen at low temperatures.

11. RLAT test results showed that Sasobit improves rutting resistance of asphalt, especially at high operating temperatures such as 50 and 60°C. This

point also indicates that the reduced mixing temperature of SWMA does not compromise the permanent deformation performance of this mix. On the other hand, Cecabase makes asphalt slightly susceptible to rutting, and this effect could be due to the negative impact of this additive on bitumen resistance to rutting as indicated by MSCR test results or could be because the additive retards bitumen ageing. The combined effect of these reasons can explain the increased rutting susceptibility of CWMA mix.

12. Four fatigue cracking analysis methods were applied in this study to examine the applicability of these methods to characterise fatigue life on nonstandard asphalt mixtures such as WMA and WMRA. The traditional 50% reduction in modulus method was the best method in capturing the zone of sharp crack formation; therefore, this method was implemented in this research.

13. Sasobit's general impact on fatigue behaviour was the sudden formation of a sharp crack when failure takes place, indicating an increased tendency of asphalt towards brittle fatigue failure. In contrast, Cecabase flattens the fatigue degradation curves indicating an increased tendency of asphalt towards ductile fatigue failure.

14. Generally, there was no clear trend of the used additive's impact on asphalt fatigue performance. At 20 °C Cecabase clearly reduced fatigue life whereas Sasobit slightly reduced it when the strain level was below 180 με; above this level, both additives seemed to increase fatigue life. Conversely, at 0 °C, both additives improved fatigue life to a certain extent when the strain level was below 180 με; above that level, both additives slightly reduced

fatigue life. At 10 °C, however, fatigue performance of CWMA and SWMA was comparable to HMA when tested below a strain level of approximately 150 με; above this level, both additives reduced fatigue life. These results demonstrate that these additives have changed asphalt sensitivity to the applied strain and testing temperature; therefore, both factors should be included in the analysis and prediction of fatigue performance of WMA.

15. The impact of the additives on asphalt ageing confirmed the conclusion that these additives retard binder ageing. That was evident in the ageing index based on E* results before and after the long-term oven ageing process. This point indicates that both additives can improve asphalt fatigue performance in the field.

16. The LTOA impact on fatigue performance of the studied mixtures showed that the control mix fatigue life slightly dropped after ageing at low strain levels, and it was comparable at high strain levels. SWMA fatigue life increased at strain levels lower than 180 με and reduced above that level, which indicates an increased brittle behaviour at high strain levels. CWMA fatigue performance did not change after ageing, which can be attributed to the retarded asphalt ageing due to the presence of Cecabase.

17. SWMA moisture damage resistance can be considered satisfactory since this mix passed the 70% ITSR requirement and the 0.45 MPa NCHRP suggested threshold. This mix also passed the NCHRP strength threshold 0.793 MPa, which should be achieved after LTOA and one MCC. Bearing in mind that in this research, three MCCs were applied after the LTOA

process instead of one; this gives an additional indication that this mix can perform acceptably in terms of durability measurements.

18. On the other hand, CWMA can also perform sufficiently concerning the durability of this mix, although this mix showed the lowest ITSR, 70.2%. This mix also passed the 0.45 MPa ITS limit without ageing or conditioning and the 0.793 MPa after the combined LTOA and MCC. Moreover, in the three MCCs, it was observed that the stiffness of this mix nearly stabilised after the first MCC whereas it kept dropping in the other mixes, which also indicates that this mix can perform acceptably in terms of moisture damage resistance.

8.2.2.2 WMA Containing RAP

1. Performance of WMA containing RAP depends to a significant extent on DoB which is found to be a function of mixing time and mixing temperature in addition to the virgin binder, RAP percentage and properties; therefore, these parameters should be optimised and accurately designed to assure acceptable quality and performance of that kind of asphalt.

2. Based on the developed DoB quantification method, WMRA with 50% RAP with comparable volumetric properties, ITSM, and ITS to a control mix can be produced when mixing at 135°C for five minutes. This means that RAP binder viscosity should be less than about 1000 cP to allow for an acceptable blending with the virgin binder to take place, although this high viscosity should become lower when mixing with a soft binder.

3. The mechanical properties of this asphalt depend to a critical extent on the grade of the virgin binder. This is because any incomplete binder blending

makes the mechanical response of the mix dependent on three individual binders rather one homogeneous binder; these are the RAP binder, the virgin binder, and the blended binder. In this situation, the first binder that fails under loading and environmental conditions may cause the failure of the entire mix.

4. Although the 189-pen binder resulted in comparable air voids, stiffness, and strength, further investigation showed that this mix was susceptible to permanent deformation when tested at a temperature higher than the critical high temperature of this binder; more specifically higher than 42°C. Lower than this temperature, rutting performance was slightly better than the control mix. This means that if the DoB is not exactly 100%, there is a risk of rutting failure due to the failure of the remaining unblended soft binder under operational temperatures higher than its critical high temperature.

5. Despite the 189-pen binder compromising rutting performance at high temperatures, this binder reduced E* of asphalt at low and intermediate temperatures which significantly improved fatigue life of asphalt. This means that WMRA performance can be improved against specific distress by accurately selecting a virgin binder grade and suitable production conditions.

6. There is limited evidence that the WMA additives used can enhance RAP binder and virgin binder blending. This appeared in RLAT results of SWMA-188 and CWMRA-188 mixtures when tested at 60°C; the results showed that the presence of the additives reduced the plastic strain by about

10-15% in the case of Cecabase and Sasobit respectively in comparison with WMRA without any additives.

7. Using the 93-pen binder significantly improved the rutting performance of WMRA mixtures, even at high operating temperatures. This was evident in RLAT results of SWMRA-93 and CWMRA93 mixtures at temperatures of 50 and 60°C. This improved performance can be attributed to the grade of the rejuvenator, which has a critical high temperature of 52°C, which means this binder would start to fail under operating temperatures higher than this temperature.

8. Despite the 93-pen binder increasing WMRA stiffness and improving its rutting performance, fatigue life of SWMRA-93 and CWMRA93 mixtures was still significantly better than the control mix. This result can be attributed to the grade of this binder, which was sufficiently stiff to resist permanent deformation and reasonably soft to enhance fatigue performance of this mix. This point also means that an accurate selection of the rejuvenator grade can significantly improve the mechanical performance of the designed mix.

9. As a rule of thumb in selecting the grade of a soft binder as a rejuvenator in asphalt mixtures containing a high percentage of RAP (i.e. >30%), the softest binder that has a critical high temperature higher than the annual highest pavement temperature can be selected as a rejuvenator. Obviously, this rule can be applied in countries with mild to cold weather with annual highest pavement temperature lower than 40°C.

10. Investigating ageing resistance of WMRA revealed that this mix can have a good ageing resistance. A possible reason for this conclusion is that once the soft binder is blended and surrounded by the aged binder, the aged binder tends to protect the soft binder from the ageing and oxidation process. Another possible reason could be the aggregate gradation of WMRA, which generally has more passing in the zone 0-3 mm than the control mix. Despite the macro density matching the target density, the spatial density could be different, especially in the mastic zone, which may prevent the oxidation process and retard bitumen ageing.

11. The LTOF process improved fatigue performance of WMRA, but at the same time, it became more sensitive to strain magnitude, similarly to SWMRA fatigue behaviour: increased fatigue life when the strain level is below 180-200 με and decreased fatigue life at higher strain levels.

12. Moisture resistance of WMRA was satisfactory as the ITSR of this mix was 81.3%. This mix also passed the NCHRP strength requirements, 0.45 MPa after moisture conditioning, and 0.793 MPa after ageing and moisture conditioning; which indicates that this mix is not susceptible to moisture damage. These conclusions can probably be explained by two factors. Firstly, some studies have mentioned that asphalt ageing can enhance adhesion forces between aggregate and binder, which makes it more moisture damage resistant as the improved adhesion prevents the intrusion of water. Secondly, if the assumption of the increased asphalt density (low air voids content) in the 0-3 mm aggregate zone is valid, then this factor can improve water damage resistance by preventing the water intrusion into this zone.

13. The variability in the results of rutting and fatigue cracking of WMA containing RAP was obviously higher than WMA and HMA. This observation can be attributed to the variability of RAP properties and blending / mixing quality between RAP and virgin materials. Accordingly, further research may be required to improve the quality of this mix and reduce its heterogeneity.

8.2.3 Performance Modelling Conclusions

1. A primary conclusion from the performance modelling results is that the response of every WMA and WMRA mixture is different from conventional HMA due to factors such as effects of additives on the rheology of bitumen, additive impact on ageing of binder and mixture, and the altered strain sensitivity of these mixes. This means that most of the currently available performance prediction models developed for conventional asphalt may not be suitable to simulate and predict these mixtures.

2. The mechanical performance of asphalt depends on two primary elements. Firstly, pavement response which depends on pavement structure and elastic or viscoelastic properties of pavement layer properties. Secondly, the way that each asphalt type responds to the applied loads, which makes the performance of every asphalt type perform differently.

3. Asphalt rutting can be predicted from RLAT results by utilising the MEPDG rutting model. The model parameters can be determined by an optimisation process to fit model predictions to RLAT results. But it was necessary to include a correction function to account for lateral confining stress since RLAT is an unconfined test. This method reasonably discriminated rutting

performance of the studied mixtures, but it must be calibrated before it can be applied in real-life permanent deformation prediction.

4. SWMA outperformed other mixtures in terms of all implemented performance indicators. One fundamental reason for this result is that the Sasobit increases asphalt stiffness, which reduces strain levels in this material and this point would definitely improve the mechanical performance of this mix.

5. Performance prediction of CWMA revealed that this mix slightly increased rutting, which can be explained by the effect of Cecabase on bitumen creep and creep recovery. Nevertheless, the total permanent deformation of this mix was about 1 mm after twenty years of simulation, which is sufficiently minor to be neglected.

6. WMRA rutting prediction showed that this mix can exhibit significantly higher rutting than other mixtures. A possible reason for this result was the sensitivity of the mix to temperature and number of load applications since the weights of these factors in the model (a2 and a3) were the highest of all the mixes. Another reason was the high vertical strain level calculated by pavement response, which generally increases permanent deformation.

7. Fatigue cracking performance prediction showed that fatigue behaviour of asphalt depends on two factors. Firstly, material behaviour such as CWMA which showed higher top-down and bottom-up cracking due to the relatively poor fatigue life of this mix. Secondly, by mix stiffness; the stiffer the material, the lower the strain level, which reduces fatigue damage; SWMA is an example of this kind of response.

8. Although WMRA fatigue performance was efficient, the reduced stiffness of this mix caused excessive tensile strain levels at the pavement surface which caused slightly increased top-down cracking; whereas at the bottom of the base layer, the difference in the horizontal tensile strain was not significant. Therefore, the bottom-up fatigue cracking performance of WMRA was superior. Accordingly, this mix can be used as a base or surface layer, but it best fits an asphalt base layer. Also, improving fatigue life of an asphalt mix may not guarantee fatigue cracking performance of an asphalt pavement; fatigue life should be improved but not at the expense of material stiffness.

9. SWMRA-93 and CWMRA-93 exhibited promising performance in terms of fatigue cracking, but their performance could not be simulated because of the limited performance results of these mixes, which were not sufficient to build sufficient performance prediction models.

8.3 Recommendations for Future Research

This study has covered different fundamental aspects of WMA and WMRA, such as production conditions, mechanical performance, and performance modelling. However, further research is required to cover essential areas that have not been covered in this study or even to provide a better understanding of the behaviour and characteristics of WMA and WMRA mixtures. The areas that are suggested for further investigations are as follows:

1. In this study, an image processing method was developed and applied to quantify aggregate coating during the asphalt mixing process. However, this method was applied to two WMA additives only. A further study to improve

the accuracy and sensitivity of the image processing method covering a wider range of WMA additives would be very beneficial. Because in this case, the mixing temperatures of these materials will be designed based on the main objective of using these additives, which is aggregate coating. Also, applying this method to the third kind of WMA, which is by foaming, could be interesting, and it could widen the application of the image processing method. However, the foaming time of the bitumen is quite limited, so it is expected to be challenging to apply this method when no foaming additives are used.

2. The compaction temperatures of asphalt were not covered thoroughly in this study. The limited compactability results presented in chapter four showed that workability evaluation of asphalt by the roller compaction method can be an effective approach. In contrast, this compaction method resulted in quite rational compaction temperatures for SWMA and CWMA; but it could not be applied in the case of WMRA due to the differences in the aggregate gradations especially the sizes 0-2mm which can significantly alter asphalt workability. Accordingly, this compaction method is recommended for further research to evaluate asphalt workability at different temperatures, and the following modifications are suggested to improve the outcome of this method further:

- Divide aggregates into portions more finely such as eight sizes rather than four (for 0-14 mm AC); in this case, matching the target aggregate gradation can be more accurately achieved, and this factor can be controlled.

- Change the compaction effort process from four loads into two loads. This step is required to simulate the compaction process in the field; the first effort is five passes at 172 kPa to simulate compaction of asphalt by the paving machine; the second effort is at 379 or 483 kPa to simulate asphalt compaction by roller compactors. In this way, the compaction effort of different mixes can be compared directly without converting compaction effort from one load into another.

- Store asphalt slabs in an oven at the designated compaction temperature prior to the compaction process. This step is required to homogenise the temperature variation of asphalt in the compaction moulds, and that should improve the reliability of the compactability results.

3. The incorporation of rubber (rubberised bitumen) or polymers (polymer modified bitumen) has shown some essential benefits such as improving rutting and fatigue characteristics of asphalt. The high production temperatures of these kinds of asphalt, however, is one of the main factors that hinders the application of this mixture. The high production temperatures are necessary to reduce the viscosity of the modified bitumen to ensure sufficient mixing and compaction. Accordingly, investigating the applicability of using WMA additives to reduce production temperatures of these mixtures would be extremely important since it may achieve significant economic and environmental benefits in addition to the performance advantages.

4. The RLAT discriminated the rutting performance of different WMRA mixtures sensitively. Accordingly, it would be interesting to investigate the

capability of this test to quantify the DoB between RAP binder and virgin bitumen of different RAP containing asphalt mixtures (hot or warm). But it is extremely important to control aggregate gradation and volumetric properties of the studied mixtures in order to make DoB as the sole contributing factor affecting RLAT results.

5. WMA mixtures containing RAP showed high variability in mechanical performance results. Investigating new methods to reduce RAP heterogeneity and control the quality and homogeneity of WMA-RAP mixtures will be valuable research. This is because it will increase the acceptability of this mix and eliminate one of the main factors that hinders the implementation of this kind of asphalt.

6. Calibrating the developed rutting and fatigue cracking models in this study could be quite useful in pavement design and management projects as these mixes would probably be more implementable in the future and so far, there is no published study in the UK regarding this topic.

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